CAMDEN COUNCIL

NEPEAN RIVER FLOOD STUDY

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4	Final Report	NM N Martin	WJH W Honour	C Thomas	15/04/2015

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1. INTRODUCTION

The Nepean River enters the Camden LGA by passing through its southern border just southeast of the township of Camden (*refer* **Figure 1.1**). From there, the river meanders some 30 kilometres to the northwest before exiting the Camden LGA and entering the Bents Basin State Conservation Area.

The Nepean River then turns to the north and flows a further 15 kilometres before reaching the township of Wallacia. North of Wallacia the river turns markedly to the west before finding its confluence with the Warragamba River a further 3 kilometres downstream.

The catchment of the Upper Nepean River within the LGA (*including the Narellan Creek catchment*) covers an area of some 14,345 hectares. The study area for this Flood Study encompasses the catchment of the Upper Nepean River within the Camden LGA (*excluding the Narellan Creek subcatchment, modelled separately*) which covers a total area of some 10,930 hectares. A number of tributaries flow into the main stream in the vicinity of Camden. The most notable of these are Navigation Creek, Narellan Creek, Matahil Creek, Sickles Creek, Cobbitty Creek and Bringelly Creek.

The Nepean River flows along the northern and eastern boundaries of the Camden township. The subcatchment area of Navigation Creek drains into the river downstream of Camden, while the subcatchment areas of Matahil Creek, Narellan Creek, Sickles Creek, Cobbitty Creek and Bringelly Creek drain into the river downstream of Camden (*refer* **Figure 1.1**).

The total catchment of the Upper Nepean River considered within this study covers an approximate area of 1,800 square kilometres and comprises the Nepean River (*and its tributaries*) upstream of its confluence with the Warragamba River. Menangle Weir and Cowpasture Bridge / Camden Weir are the key reference locations within the Camden LGA used in this study. The catchment areas upstream of these locations are approximately 1,280 and 1,380 square kilometres respectively. Although not within the LGA of Camden Council, Wallacia is the key reference location at the downstream extent of the study and is located 15 kilometres downstream of the northern LGA boundary and approximately 2 kilometres east of the Nepean / Warragamba confluence. The catchment area upstream of this location is 1,795 square kilometres.

Three major population centres exist within the LGA of Camden; Camden / South Camden (*on the southern side of the Nepean River*), Elderslie (*on the northern side of the river adjacent to Camden*) and Narellan further to the east (*refer* **Figure 1.1**). It is understood that the suburbs of Harrington Park, Currans Hill and Mount Annan, in the vicinity of Narellan, have all expanded rapidly since the completion of the previous Upper Nepean Flood Study in 1995. Several areas in the vicinity of Camden itself have also expanded significantly since the previous study was completed, including Cobbitty, Ellis Lane, Grasmere and Kirkham. Furthermore, a new subdivision known as Spring Farm has introduced approximately 4,000 new lots to the northern side of the Nepean River.

Flooding events are rare within the study area. However, when such events do occur, flows escaping from the Nepean River are known to inundate the low lying areas of Camden and certain sections within South Camden and Elderslie. Floodplain areas along many of the tributaries of the river (*particularly Narellan Creek and Matahil Creek*) are also known to be affected by backwater flooding from the Nepean River during flood events. Although the current study focuses on flooding from the Nepean River, flooding from these and other tributary streams are thought to represent potential flooding sources in their own right during extreme rainfall events.

The modelling undertaken during the study aims to provide a more robust and detailed definition of flooding behaviour within the study area and produce a series of maps and datasets that summarise the outputs of the models. It is intended that this information and the associated models will then be used in the development of flood management measures as part of the subsequent Floodplain Risk Management Study and Plan.

2. STUDY METHODOLOGY

2.1 STUDY AREA

The study area defined by Council is as follows (*refer* **Figure 1.1**):

- The Nepean River from Menangle Weir (*4.5 kilometres upstream of Camden Council's LGA boundary*) to the river's confluence with the Warragamba River near Warragamba Park; and
- Five tributaries of the Nepean River within Camden Council's LGA boundary, namely Navigation Creek, Matahil Creek, Sickles Creek, Cobbitty Creek and Bringelly Creek.

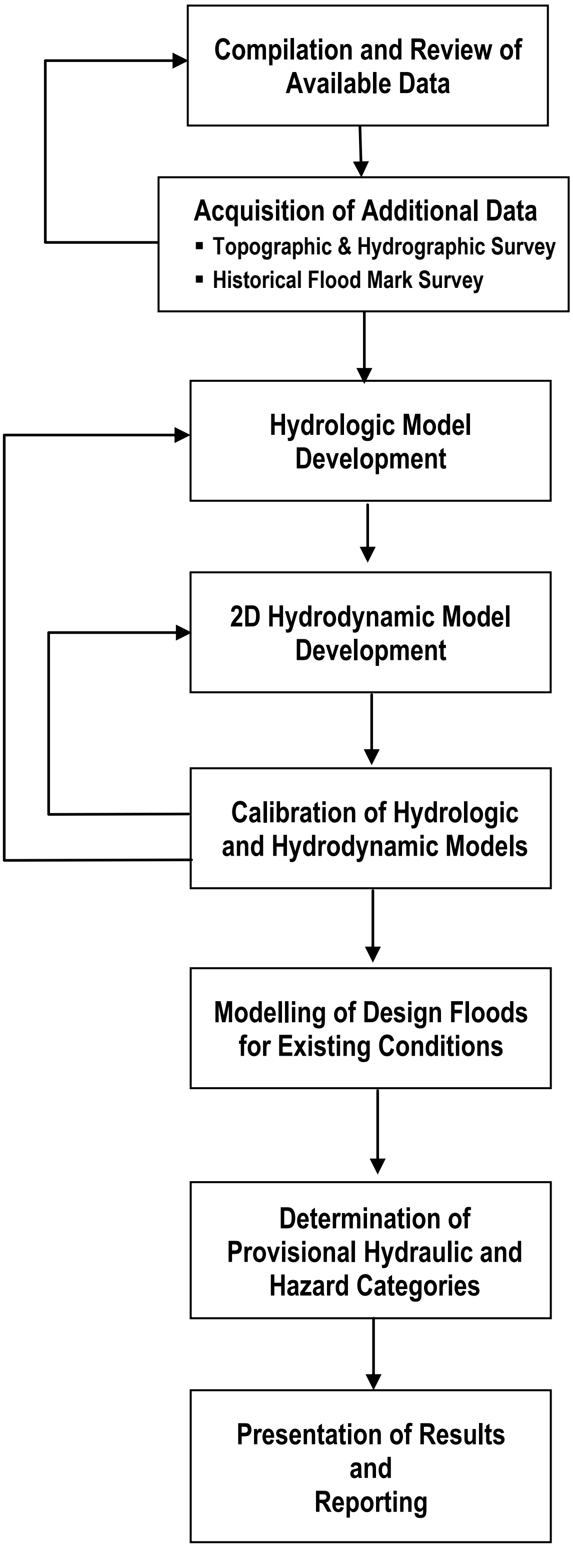
It should be noted that the Narellan Creek catchment is being modelled as a separate project and hence has not been specifically incorporated into the Nepean River study area.

2.2 ADOPTED APPROACH

The general approach and methodology employed to achieve the study objectives involved:

- Compilation and review of available information, including previously completed flood studies, streamflow gauge records, rainfall records, topographic mapping of the floodplain and details of bridge crossings and other structures;
- Site inspections and interrogation of aerial photography and other geographical data in order to establish catchment roughness, slope and land-use attributes;
- The collection of historical flood information, including records of peak flood levels for historical floods;
- The development of a computer based hydrologic model to simulate the transfer of rainfall into runoff and its concentration in local streams during flood events;
- The development of a computer based hydrodynamic model to simulate the movement of floodwaters through the reaches of the Nepean River floodplain that lie both within and downstream of the boundaries of the Camden Shire Council LGA area;
- Calibration and verification of the models; and
- The determination of peak flood levels and flow velocities at selected locations along the selected reach of the Nepean River for the predicted 500, 200, 100, 50, 20, 10, 5 and 2 year Average Recurrence Interval (*ARI*) floods and the Probable Maximum Flood (*PMF*).

The flow chart shown overleaf outlines the key steps and the sequence of work that has been undertaken in preparing this Flood Study.


Compilation and Review of Available Data
Acquisition of Additional Data
▪ Topographic & Hydrographic Survey
▪ Historical Flood Mark Survey
Hydrologic Model Development
2D Hydrodynamic Model Development
Calibration of Hydrologic and Hydrodynamic Models
Modelling of Design Floods for Existing Conditions
Determination of Provisional Hydraulic and Hazard Categories
Presentation of Results and Reporting

2.3 COMPUTER MODELS

Computer models are the most reliable and cost-effective tools available to simulate flood behaviour in rivers and streams. Two types of computer models were developed as part of the Flood Study for use in assessing and quantifying flooding characteristics within the Nepean River catchment. These are:

- A hydrologic model, covering the Nepean River and its tributaries upstream of and within the study area; and
- A hydrodynamic model, extending along the Nepean River between Menangle Weir and its confluence with Warragamba River.

The **hydrologic model** simulates catchment runoff following a particular rainfall event. The main outputs from the hydrologic model are discharge hydrographs which define the quantity of runoff as well as the rate of rise, timing and magnitude of peak discharges resulting from the rainfall event. The discharge hydrographs are utilised as inputs into the hydrodynamic model.

The **hydrodynamic model** simulates the passage of floodwater along waterway reaches and across floodplain areas. The hydrodynamic model calculates key flooding characteristics such as flood levels, flow velocities, floodwater depths and flood hazard at selected points of interest throughout the study area.

3. REVIEW OF AVAILABLE DATA

3.1 HISTORY OF FLOODING

Flooding events have proven to be rare within the study area since European settlement of the area. However, when such events do occur, flows escaping from the Nepean River are known to inundate the low lying areas of Camden itself and certain sections within South Camden and Elderslie. Floodplain areas along many of the minor tributaries of the river (*particularly Narellan Creek and Matahil Creek*) are also known to be affected by backwater flooding from the Nepean River during flood events. Flooding from these and other tributary streams also represent potential flooding sources in their own right during extreme rainfall events.

Reliable streamflow records have been recorded in Camden since 1949. Several large floods have occurred during this time, notably in June 1964, June 1975, March 1978, April 1988 and August 1990. Significant flooding also occurred in 1873. However, data relating to this event is limited to peak flood levels at isolated locations along the river.

The June 1964 flood is the largest to have occurred since records at Camden commenced. Despite limited data, the 1964 flood was estimated to have an Average Recurrence Interval (*ARI*) of 15 years in the '*Upper Nepean River Flood Study*'. The second largest flood on record is the March 1978 event.

During large flood events, residential and commercial areas in Camden are inundated. A number of bridges over the Nepean River upstream of Theresa Park are inundated during relatively minor flood events resulting in potential isolation of local residents.

Table 3.1 provides a list of the major floods and the estimated peak heights during these events at Cowpasture Bridge and Camden Weir (*in Camden*) and at Wallacia (*refer* **Figure 1.1**).

Table 3.1 RECORDED PEAK FLOOD HEIGHTS

FLOOD EVENT	PEAK FLOOD HEIGHT (*mAHD*)		
	Cowpasture Bridge	Camden Weir	Wallacia Weir
June 1964	69.75	-	43.93
March 1978	69.12	-	42.24
April 1988	68.45	-	40.81
August 1990	66.30	66.11	39.21

3.2 PREVIOUS INVESTIGATIONS

A number of previous flood studies have been undertaken that relate to flooding within the study area. A synopsis of those investigations considered relevant to this study is provided in the following sections.

3.2.1 Camden Floodplain Management Strategy Study (1985)

This study, completed by Lyall Macoun and Joy, assessed the 100 year ARI flood extent for Camden (*generally northern areas*) as well as the damages across residential, industrial and commercial areas during an event of this magnitude. Mitigation options were also established, with a final proposed option strategy considering a combination of property purchases, development controls, zoning changes and flood proofing.

3.2.2 Camden Flood Study (1986)

A flood study was undertaken by the Water Resources Commission for Camden in 1986 which used a HEC-2 steady state analysis in order to determine water surface profiles for both the 20 and 100 year ARI flood events. Cross-sections were derived from orthophotomaps and incorporated into the hydraulic model.

The primary aims of the study were to delineate floodways and inundation limits within the study area. The study assessed flood damages, flood hazard and identified possible flood mitigation measures. Model calibration utilised data collected from the June 1964 and March 1978 flood event. A simple flood frequency analysis was also undertaken using the existing streamflow data for Cowpasture Bridge.

3.2.3 Macarthur South Water Quality, Drainage and Flood Study (1990)

This study, undertaken by Sinclair Knight & Partners, involved the development of a RORB hydrologic model for the entire Upper Nepean River catchment. The model was calibrated for the March 1978 flood event and generated design flows for the 5, 20 and 100 year ARI flood events. As part of the calibration, pluviograph data was obtained from the Water Board (*now Sydney Water*).

A MIKE11 hydrodynamic model was constructed using 30 cross-sections of the river between Maldon Weir and Wallacia.

3.2.4 Warragamba Flood Mitigation Dam Environmental Impact Statement Flood Study (1995)

The Upper Nepean River was modelled in 1995 as part of the Warragamba Flood Mitigation Dam EIS by Webb McKeown and Associates. The investigation involved the construction of a RUBICON hydraulic model extending as far upstream as Camden in order to derive flow hydrographs at the Nepean River / Warragamba River confluence. As such, only the peak water surface profiles between Wallacia and the Nepean River / Warragamba River confluence derived in this study hold relevance to the Upper Nepean River as the majority of the recorded data was collected downstream of the confluence.

3.2.5 Upper Nepean River Flood Study (1995)

This study was undertaken by Lyall and Macoun Consulting Engineers (*LMCE*). The objective of this study was to define flood behaviour in the Upper Nepean River in terms of flooding behaviour for flood frequencies between the 0.5% and 20% AEP, as well as the Probable Maximum Flood (*PMF*) extreme event.

A RORB hydrologic model was produced and was calibrated to the June 1964, June 1975, March 1978, April 1988 and August 1990 historical flood events.

MIKE11 hydrodynamic modelling was undertaken in order to generate peak flood level results for the design runs utilising the hydrographic cross-sections of the Nepean River completed as part of the Camden Flood Study by the Water Resources Commission in 1986.

One model was used to determine small to medium flood events (*5 to 20 year ARI*), while another, shorter model was used for major flooding (*100 year ARI and larger*) where out-of-channel flows effectively 'shortened' the relative channel lengths along the river. This study found that backwater flooding from the Nepean River controlled flood levels for a considerable distance upstream along the tributaries.

3.2.6 Upper Nepean River – Tributary Flood Studies Volume 1 (1997 & 1998)

Following the Upper Nepean River Flood Study (*1995*), Lyall and Macoun Consulting Engineers (*LMCE*) completed a staged approach in defining flood characteristics of Upper Nepean River tributaries located in the vicinity of Camden. Stage 1 involved defining flooding in Navigation, Sickles, Matahil (*East and West*) and Cobbitty Creeks, while Stage 2 investigated flooding on Bringelly Creek and the lower reaches of the Narellan Creek catchment.

Both stages involved creating separate RORB hydrologic models for each of the tributaries (*except Narellan Creek*). These models were calibrated to peak discharges obtained by the probabilistic rational method (*PRM*) for the 10 and 100 year ARI events. An XP-RAFTS model of Narellan Creek, provided by Council and adjusted by LMCE, was used to derive flows generated from the Narellan Creek catchment.

The construction of the hydraulic model involved refining the existing MIKE11 model used for the Upper Nepean River Flood Study (*1995*). The updated model contained 70 cross-sections along the main river channel but also included 50 sections along the tributaries in the vicinity of Camden. During this stage of the overall series of studies undertaken by LMCE, additional sections were added in order to refine localised conditions (*i.e. bridges, culvert, and changes in channel dimensions*) that were not deemed critical in the original, broader Upper Nepean River Flood Study.

3.2.7 Upper Nepean River – Tributary Flood Studies Volume 2 (1999)

The second volume of the Upper Nepean River Tributary Flood Studies, completed by Lyall and Macoun Consulting Engineers (*LMCE*) involved determining the Probable Maximum

Flood (*PMF*) flood levels along Narellan Creek upstream of Northern Road and its six tributaries. The study involved adjustments and extensions to the existing XP-RAFTS model as well as a PMP estimation based on the Generalised Short Duration Method.

The existing MIKE11 hydraulic model was initially to be utilised for the downstream extent of the catchment and extended upstream. However, instabilities at drop structure locations meant that a HEC-RAS hydraulic model was used to model flows in the upper reaches of the Narellan Creek catchment. The depth, velocity and peak discharges during the PMF were then examined in detail at various road crossings. Sensitivities to culvert blockages were also examined in the study.

3.3 AVAILABLE DATA

A range of data is required in order to develop a hydrodynamic flood model and for that model to be applied to simulate flood behaviour. Typically, contours of the land surface and cross-sections of the river and creek system are required to represent the floodplain topography and channel bathymetry. Details of critical hydraulic controls such as bridges and roadway embankments also need to be defined as they can influence flooding characteristics. In addition, surface roughness parameters are required to reflect the influence that land features and vegetation may have on the way floodwaters travel overland. These are usually based on consideration of vegetation density and floodplain geomorphology with reference to published recommendations (*e.g., Chow, 1959*).

Streamflow data and historical flood level information are needed for calibration and verification of a flood model. Streamflow data is typically available from gauges where flows or water levels have been recorded over time. Historical flood levels can also be established by field survey after a flood or from anecdotal information provided by those who witnessed or experienced the flood. This data is extremely valuable and can be used to calibrate and verify the flood model.

The data for this study has been obtained from a number of sources including Camden Council, the Sydney Catchment Authority and the Office of Environment & Heritage (*formerly the Department of Environment, Climate Change & Water* (*DECCW*)).

Historical flood information was gathered from previously published flood level and stream flow records, most of which was obtained from government agency archives. This was supplemented by data contained in previous investigations and anecdotal recollections and photographs gathered from community members as part of consultation activities for this study.

3.3.1 Topographic Data

Between 25th February 2011 and 23rd March 2011, an aerial laser survey was undertaken across a large area of land in that encompasses Camden Shire and the study area upstream and downstream of Council's boundaries. This data has been made available by the New South Wales Government Department of Land and Property Information (*LPI*).

The survey generated *Light Detection and Ranging* (*LiDAR*) data for the study area. The processed LiDAR data was provided as spot elevations in a grid with a spacing of one metre across all terrestrial sections of the study area. Available documentation from those responsible for procurement of the data indicates that spot elevations have a vertical accuracy of 0.3 metres and a horizontal accuracy of 0.8 metres.

The LiDAR data is considered to provide the most reliable and extensive data-set defining the topography of the Nepean River floodplain and has, therefore, been utilised as the primary source of topographic data used in the study.

Aerial laser survey techniques are unable to penetrate through water. Therefore, the LiDAR data does not include hydrographic features that are often important for flood modelling, most notably the bathymetry of streams that carry water under normal flow conditions. That is, the LiDAR data does not typically include data defining the bed and lower banks of the river and creek channels.

Given the size and channel characteristics of the main Nepean River channel, aerial surveys are likely to have been conducted during times when the volumes of water within the river were considerable. However, trapezoidal extrapolations of the LiDAR data according to known river depths revealed that the volumes of water present in the channel are considered to only represent a small percentage of the total bank-to-bank river channel volume in many locations and a significantly smaller percentage of the wider floodplain volume. Despite this, in a number of locations the apparent volumes present in the channel at the time of the LiDAR capture was significant.

Accordingly, it is considered inappropriate to assume that the LiDAR data adequately represents the channel dimensions and correct conveyance capacity of the Nepean River channel. Thus an additional source of topographic data was required in order to adequately capture the bathymetry of the Nepean River appropriately for use in a two-dimensional hydrodynamic flood model.

With this in mind, Camden Council was able to acquire additional data in the form of a detailed hydrographic survey of the river channel captured by the Sydney Water Monitoring Services Hydrometric Services Group (*refer below*).

Analysis of the tributaries of the Nepean River indicated that the channels contained low volumes of water during the capture period of the aerial laser survey data. As such, it was deemed appropriate to assume that the LiDAR data could be used to adequately represent the hydrographic topography along these tributaries.

3.3.2 Hydrographic Data

Hydrographic survey of the Hawkesbury Nepean Valley was undertaken by Sydney Water throughout 2011 and 2012. This survey encompassed the majority of the study area, with cross-sections beginning just downstream of Menangle Bridge (*approximately 500 metres downstream of Menangle Weir*) and ending at Wallacia Weir.

Utilising the data set available, a hydrographic surface of the main Nepean River channel was able to be created through the use of data point interpolation. This surface was used to complement the LiDAR within the model's topographic database.

It should be noted that a 3 kilometre section of the Nepean River as it flows through the gorge between Theresa Park and Bents Basin was deemed inaccessible by Sydney Water and thus, data for this area has not been recorded. However, a suitable interpolation of the stream channel geometry within the gorge was obtained using the upstream and downstream cross sections and the nearby LiDAR topographic data.

3.3.3 Hydrologic Data

Extensive searches were undertaken to obtain as much hydrologic data as possible for the Nepean River catchment within and in the vicinity of the study area. Hydrologic data usually exists in two forms, namely:

- Pluviometer and daily read rain gauges rainfall records; and
- Stream discharge records.

A summary of the data that was obtained is outlined in the following sections.

Historical Rainfall Data

Continuous rainfall data for specific storms is required for the calibration and verification of hydrologic computer models. This data is usually obtained from pluviometers located within or in the immediate vicinity of the catchment being modelled. Pluviometers generate plots of the instantaneous variation in rainfall with time.

An investigation was carried out to determine the location and details of rainfall gauges within the catchment of the Nepean River upstream and within the study area. The investigation included a search of the Bureau of Meteorology's (*BOM*) Water Resources Station Catalogue and New South Wales' Government's Water Information Database. Rainfall data utilised within the previous modelling of the catchment (*in particular, the Upper Nepean River Flood Study, 1995*) was also used to guide the availability and suitability of the various sources of rainfall data.

The investigation determined that a total of four pluviometer stations are located within the study area (*including Narellan*). Moreover, a further four pluviometer stations were identified in nearby catchments immediately east and west of the catchment boundary in the vicinity of the study area.

The distribution of the pluviometer stations investigated for the catchment is shown in **Figure 3.1**. A summary of the gauges and the recorded rainfall data available for each of these gauges is listed in **Table 3.2**.

Table 3.2 SUMMARY OF INVESTIGATED PLUVIOMETER RAINFALL GAUGES

RAINFALL GAUGE NAME	LOCATION *[lat/long]*
Camden Park Reservoir	-34.1164, 150.716
West Camden STP	-34.0590, 150.681
Camden Airport[1]	-34.0400, 150.690
Narellan	-34.0333, 150.783
Brownlow Hill	-34.0254, 150.646
Oakdale	-34.0694, 150.441
Campbelltown (Mt Annan)[2]	-34.0600, 150.770
Pondicherry	-33.9876, 150.736
Badgerys Creek	-33.8833, 150.750
Warragamba Met Station	-33.8902, 150.593

1 Not required as data available during calibration periods for nearby gauges at Camden Park Reservoir, West Camden STP and Narellan.

2 Data not available during calibration periods.

Streamflow Data

Streamflow data is generated from rating curves for gauging stations that are located along streams and rivers. A time series of flood level over the duration of a flood is recorded at the gauging station and the corresponding rating table is used to generate a discharge hydrograph. The discharge hydrograph provides a measure of the rate of flow at any particular time during the flood (*i.e., the number of megalitres per day or cubic metres per second*).

The NSW Office of Water's PINNEENA database (*2001*), NSW Government's Water Information Database and the Sydney Catchment Authority's stream database was interrogated in order to identify the location of all stream flow gauges within and upstream of the study area. The search identified that records exist for three gauges on the Nepean River within the study area, in addition to a gauging station at the upstream boundary of the study area at Menangle Weir. The locations of these gauges are shown in **Figure 3.2**. A summary of the gauges and the extent of the streamflow record at each gauge is provided in **Table 3.3**.

Table 3.3 SUMMARY OF INVESTIGATED STREAM GAUGING STATIONS

GAUGE No	GAUGING STATION NAME	LOCATION *[lat/long]*	RECORD AVAILABLE
212238	Menangle Weir	-34.1183, 150.7430	1968 to present
212900	Cowpasture Bridge	-34.0544, 150.7047	1949 to present
212216	Camden Weir	-34.0479, 150.7035	1989 to present
212202	Wallacia	-33.8611, 150.6302	1908 to present

3.3.4 Historical Flood Levels

A number of details from historical flood events collected as part of the Nepean River Flood Study (*1995*) were obtained as part of the study. This data was in the form of surveyed flood marks, as reported by local residents and businesses relating to past flood events. The levels (*in metres above Ordnance Datum*), river chainages and / or locations were included within Appendix C of the final Nepean River Flood Study (*1995*).

Further information relating to historical recollections was also obtained as part of the community consultation process of this study. A total of 126 questionnaires were completed and returned by local residents, primarily within the town centre of Camden. Of these, nine contained location and peak water level information that was specific enough to be utilised as historical flood level data.

3.3.5 Other Geographic Data

Various other sets of geographic data relating to the study area were also obtained from Camden Shire Council and other public domain / government sources. This includes aerial photography, cadastre, roadway, drainage and watercourse information.

4. FLOOD FREQUENCY ANALYSIS

Flood frequency analysis enables the magnitude of floods of a selected Annual Exceedance Probability (*AEP*) to be estimated by statistical analysis of recorded floods. Methods have been developed that allow a probability distribution to be fitted mathematically to observed data so that flood magnitudes of required probabilities can be calculated. These procedures are outlined in Chapter 10 of the 1987 edition of *'Australian Rainfall and Runoff'*, and in Book IV of the revised edition published in 1998.

The procedures apply primarily to peak discharges at a site and generally should not be applied to peak water levels. This is because the distribution of water level at a site can include discontinuities due to sudden changes in cross-sectional area as discharges increase. Furthermore, the relationship between flood stage and discharge may vary throughout the period of record due to changes in the river channel geometry caused by scouring or bank erosion.

Notwithstanding, it needs to be recognised that flood discharges are typically determined at streamflow gauge locations from recorded water levels. This is because it is not practical to constantly gauge stream velocities and flow rates at all locations where flow estimates are required. Therefore, by relating flow rate to a level or stage at a certain point on a stream, and developing a rating curve similar to that used for measuring flumes or weirs, flows can be estimated continuously by simply monitoring stream stages. Most stream gauges are 'rated' and the rating curve is used to determine the flow that corresponds to the recorded flood level.

To undertake a valid flood frequency analysis at least 10 to 15 years of streamflow data is required. The data should constitute a random sample of independent values from a homogenous population (*Institution of Engineers Australia, 1987*). If the flow records are too short the calculated probabilities cannot be expected to be reliable.

4.1 METHODOLOGY

4.1.1 Choice of Series

Flood frequency analysis can be based on either an annual or a partial series. The annual series is the most common analysis method. For annual series analysis, missing record periods are of no consequence and can be included in the period of record, provided it can be determined that no major floods occurred during years for which data was unavailable.

The annual series approach uses the maximum instantaneous discharge in each year of record. The year may be a calendar year, or if there is seasonality in flow, a water year commencing at the end of the low flow period. For N years of data there will be N values in the annual flood series.

A partial series analysis uses discharges for all floods above a specified minimum discharge, irrespective of the number that occur in a given year (*although the events should be independent*). There may be more than one flood in the analysed record for some years and none for others.

Either approach is acceptable, but can result in a different flood frequency distribution. ARR 98 suggests a general preference for the annual series if floods rarer than the 10 year ARI flood event are of primary interest (*as they are in this study*). This is because annual and partial series analyses give similar results in this probability range, and the annual series also has the advantages of likely independence of annual peaks (*this can be checked by examination of the record at the end of each year*), unambiguous extraction of records, and conformity of the annual flood frequency distribution to many theoretical distributions.

4.1.2 Frequency Distribution

Several types of probability distributions are available for flood frequency analysis, including:

- The generalised extreme value distribution (*also known as the Gumbel distribution*);
- The three parameter log-normal distribution;
- The log Pearson III distribution; and,
- The Wakeby distribution.

In flood frequency analysis, discharges in the series are plotted on a frequency diagram. This has discharge as the ordinate (*linear or log scale*) plotted against Annual Exceedance Probability (*AEP*) or Average Recurrence Interval (*ARI*) as the abscissa (*probability scale*). For the abscissa (*x axis*) Normal, Exponential or Gumbel (*Extreme Value Type I*) probability scales are most commonly used, corresponding to the commercially available graph papers of these types.

The type of plot chosen is generally a convenience, and also allows the presentation of data in such a way that deviations from the distribution assumed by the axes can be judged.

Each discharge in the annual series was given a "plotting position" (*PP*); that is, an AEP for plotting purposes, using the recommended formula in ARR 98, namely:

$$PP = \frac{m - 0.4}{N + 0.2}$$

where m is the rank in the flood series (*the highest flood in the series having rank m = 1*), and N is the number of peaks in the record (*in this case 20 years*). As the number of peaks in the data series used for the flood frequency analysis is different from the number of years, this plotting position is modified using the formula:

$$P = PP \cdot \frac{N}{n}$$

where *n* is the total length of record represented by the series.

ARR 98 recommends fitting a log Pearson III distribution for an annual series. The methodology presented in ARR 98 is based on the method of moments, preserving the moments of the logarithms of flows. The log Pearson III distribution was fitted to the partial series presented in **Table 4.1** and plotted on log normal probability scales is shown in **Appendix A**.

4.2 FLOOD FREQUENCY ANALYSIS FOR NEPEAN RIVER

Stream discharge records for three stations along the Nepean River were obtained from the Sydney Catchment Authority's stream database. In order to conduct the analysis on a longer period of data, additional historical data was also obtained from Appendix B of the Upper Nepean River Flood Study (*1995*). The data obtained is summarised in **Table 4.1**.

Table 4.1 SUMMARY OF AVAILABLE STREAM DISCHARGE DATA

LOCATION	SCA RECORD	ADDITIONAL RECORD[1]
Menangle Road Bridge / Weir	07/1990 to 12/2012	10/1860 to 02/1991
Cowpasture Bridge / Camden Weir	08/1989 to 12/2012	10/1860 to 02/1992
Wallacia Weir	12/1975 to 12/2012	10/1860 to 02/1992

1 Upper Nepean River Flood Study (DLWC 1995).

4.2.1 Nepean River at Menangle

Stream level records were recorded at a location on Menangle Road Bridge (*212904*) as far back as 1891. However, the station did not regularly begin to record day-to-day readings until 1963. A continuous recording station at nearby Menangle Weir (*212238*) has been operated by Sydney Water since July 1990.

According to the analysis of recorded data at Menangle Road Bridge undertaken during the Upper Nepean River Flood Study (*1995*), the river in this location is sandy in nature and subject to river morphology issues which would necessitate regular assessment and updates of rating curves. Similarly, the report states that an historical lack of reliable rating curves for the location appears to have resulted in data that consistently underestimates discharges when compared to discharge estimates made upstream and downstream of the station during high flow events. As such, the report concluded that data from this station is not reliable enough to be used in a flood frequency analysis.

The data taken at Menangle Weir is thought to be more reliable. However, the station stopped recording midway through the August 1990 flood event in the area and no major events have since been experienced. As such, the record at Menangle Weir is not suitable for a standalone flood frequency analysis.

Accordingly, as no truly suitable data could be obtained, flood frequency analysis was not undertaken for Menangle Road Bridge or Menangle Weir.

4.2.2 Nepean River at Camden

Informal stream level records have been recorded at Cowpasture Bridge in Camden (*212900*) since 1860, although records prior to 1949 were only taken during flood events. Whilst undertaking the Camden Flood Study (*1986*), the Water Resources Commission was able to derive a series of annual peak discharges using records from the station itself and estimations derived from nearby stations. The three highest flows assumed in the study were from 1860, 1873 and 1898 (*4860, 10060 and 7170 m^3/s, respectively*). The fourth highest discharge was the first to be experienced after the introduction of formal recordings at Cowpasture Bridge in 1949 and occurred in June 1964.

A continuous recording station at nearby Camden Weir (*212216*) has been operated by Sydney Water since August 1989. Records from this location were used in conjunction with the data from Cowpasture Bridge to create a list of the largest events in the Camden area and were used as the basis of the flood frequency analysis.

Excluding all flow events with a peak discharge less than 100 m^3/s resulted in a total of 60 records being available for inclusion in the analysis. A partial series flood frequency analysis was undertaken based on this data, which is listed in **Appendix A**. The analysis was undertaken using a log Pearson Type III distribution.

Considering that the three highest flow events assumed to have occurred were based on estimates taken up to 150 years ago, investigations were conducted with *and* without this older / higher data. The results of the analysis are presented in **Table 4.2**.

Table 4.2 SUMMARYOF FLOOD FREQUENCY ANALYSIS FOR CAMDEN

	PEAK DISCHARGE [m^3/s]						
	2yr	**5yr**	**10yr**	**20yr**	**50yr**	**100yr**	**200yr**
1860 – 2012	788	1968	3219	4865	7799	10735	14424
1964 – 2012	733	1631	2454	3423	4952	6309	7868

The results of the individual analyses suggest that the three floods that occurred in the late 19th century have a heavy influence on the overall results of the calculations, particularly for the larger events. The results of the analyses when undertaken purely on the recordings taken since 1949 (*of which the first major flood was in 1964*) are markedly lower. It is difficult to assess the accuracy or validity of the three highest flows as they relate to the flood frequency analysis results. However, in reality it appears likely that the "true" discharges for the various design flood events would lie somewhere between the two scenarios tested.

These values are generally within the range of values for Camden reported in the Upper Nepean River Flood Study (*1995*), which also investigated cases with and without the earlier, higher data. The partial series results for the dataset including the events from 1860, 1873 and 1898 (*a peak 100 year ARI discharge of 10700 m^3/s*) is similar to the equivalent annual series analysis conducted as part of the Nepean River Flood Study (*1995*). Similarly, the partial series result for the data recorded between 1950 and the present day reflects the corresponding annual series result reported in the 1995 report. The previous study also investigated additional flood frequency analysis trials. However, it is unclear what methodology was adopted during these calculations.

4.2.3 Nepean River at Wallacia

Stream level records have been recorded at Wallacia Weir (*212202*) since 1908 and continuous records commenced in 1962. As with the stream records at Camden, the three highest flows assumed at this location in the study were from 1860, 1873 and 1898 (*5090, 7080 and 5900 m^3/s, respectively*) and are based on less reliable information from the nearby area. The fourth highest discharge was the first to be experienced after the introduction of formal recordings at Wallacia commenced in 1908 also occurred in June 1964. A number of other large events were observed, starting in 1917.

Once again, the total number of records for inclusion in the analysis was restricted to 60 records, meaning that all flow events with a peak discharge less than 400 m^3/s were excluded. A partial series flood frequency analysis was undertaken based on this data, which is listed in **Appendix A**. The analysis was undertaken using a log Pearson Type III distribution.

Considering that the three highest flow events assumed to have occurred were based on estimates taken up to 150 years ago, investigations for Wallacia were also conducted with *and* without the pre-1917 data. The results of the analysis are presented in **Table 4.3**.

Table 4.3 SUMMARYOF FLOOD FREQUENCY ANALYSIS FOR WALLACIA

	PEAK DISCHARGE [m^3/s]						
	2yr	**5yr**	**10yr**	**20yr**	**50yr**	**100yr**	**200yr**
1860 – 2012	881	1737	2643	3870	6177	8635	11931
1917 – 2012	833	1469	2053	2767	3960	5101	6494

The results of the individual analyses again suggest that the three floods that occurred in the late 19th century had a heavy influence on the overall results of the calculations. The results of the analyses when undertaken purely on the recordings taken since 1917 (*of which the first major flood was in 1964*) are markedly lower. It is interesting to note that the peaks assumed at Wallacia for the 1873 and 1898 flood events were calculated as being lower at Wallacia than at Camden, suggesting that a significant degree of uncertainty is inherent in the discharge calculations undertaken for these three events. This confirms yet again that the, while it is likely that the three pre-1917 discharges do have some validity, it is likely that the “true” discharges for the various design flood events would lie somewhere between the two scenarios tested.

The values listed in **Table 4.3** are also generally in agreement with the values for Wallacia reported in the Upper Nepean River Flood Study (*1995*), which also investigated cases with and without the earlier, higher data.

5. HYDROLOGIC MODELLING

5.1 HYDROLOGIC MODEL DEVELOPMENT

A hydrologic model of the Nepean River Catchment within the study area was developed to simulate rainfall and runoff processes in the catchment and produce the river / creek flows (*discharges*) that are required to determine flood levels in the subsequent hydrodynamic model. The Runoff Analysis and Flow Training Simulation (*XP-RAFTS*) software package was used with the objective of quantifying design flood discharges from the catchments.

XP-RAFTS can be used to develop a deterministic runoff routing model for the simulation of catchment runoff processes. It incorporates a range of common catchment parameters into its calculation procedures and is recognised in *'Australian Rainfall and Runoff – A guide to Flood Estimation'* (*1987*) as a suitable tool for use in flood routing within Australian catchments.

XP-RAFTS was chosen for this investigation because it has the following attributes:

- It can account for spatial and temporal variations in storm rainfalls across a catchment;
- It can accommodate variations in catchment characteristics;
- It can be used to estimate discharge hydrographs at any location within a catchment; and,
- It has been widely used across eastern NSW and therefore, where suitable calibration data is not available, the results from modelling of other similar catchments can be used as a guide in the determination of model parameters.

The XP-RAFTS model was developed using a range of physical characteristics of the catchment. These include subcatchment area, average slope, percentage impervious area and roughness. The model accounts for rainfall losses and routes rainfall excess through the catchment of interest.

The model was used to estimate subcatchment runoff peaks and to generate discharge hydrographs which can be compared with historical, recorded flows during the calibration process and to generate design inflow hydrographs in the subsequent hydrodynamic model simulations.

5.1.1 Subcatchment Details

Although LiDAR terrain data was provided for the areas of the catchment within the Camden Shire LGA area, high resolution drainage line data of these areas was also used to define the subcatchment areas where LiDAR data was not provided. LIDAR data and aerial photography were then used to refine the subcatchment subdivisions within the LGA boundary.

As a detailed and calibrated RORB model of the Upper Nepean River had already been completed as part of the Upper Nepean Flood Study (*1995*) it was considered appropriate to only complete a detailed XP-RAFTS model of the immediate study area (*i.e., downstream of Menangle Weir*). Therefore, the pre-existing RORB model was obtained and manipulated in order to provide inflow hydrographs at Menangle Weir for all design and calibration events for use in the current study. The extent of the RORB model used to provide inflows for the catchment upstream of Menangle Weir (*i.e., for upstream catchment areas*) is shown in **Figure 5.1**.

It should be noted that, although stream level information was recorded at Menangle Weir during the 1964, 1978 and 1988 calibration events, the location was not used within the hydrologic model calibration process during the previous study. This relates to stream morphology issues at the weir and the resulting lack of reliable rating curves to convert the recorded levels into discharges (*refer* **Section 4.2.1**).

The catchment definition process resulted in the creation of 52 subcatchments within the study area downstream of Menangle Weir. As flooding in the Camden area was the primary focus of the study, the resolution of subcatchments was reduced slightly downstream of the LGA boundary at Bents Basin. Three to five subcatchments were typically used to define inflows to each of the tributaries of the Nepean River.

Subcatchments were differentiated on the basis of the alignment of major tributary flow paths and watershed boundaries, as well as the homogeneity of land use, vegetation and ground slope. Parameters such as catchment area, slope, roughness and percentage impervious area were established from the available data and assigned to each subcatchment accordingly.

Initially, subcatchments were formed by delineation of subcatchment boundaries from the intersections of major streams. Subcatchments were also defined at the locations of existing streamflow gauges so that XP-RAFTS hydrographs at these locations could be compared to recorded hydrographs to aid in calibration. Finally, subcatchment boundaries were inserted at the locations where input hydrographs were to be supplied during the hydrodynamic modelling stage.

The adopted subcatchment break-up is shown in **Figure 5.2**. A summary of the adopted sub catchment parameters is provided in **Appendix B**.

5.1.2 Adopted XP-RAFTS Model Structure

The XP-RAFTS model was developed by superimposing the model over the subcatchment break-up shown in **Figure 5.2**. The node and link arrangement was created to provide the pathways for rainfall excess to be “routed” through each of the tributary subcatchments. Details of the parameters adopted for each model node, including lag times for floodwater distribution between nodes, is contained in **Appendix B**.

5.1.3 Rainfall Loss Model

In a typical rainfall event, not all of the rainfall that falls onto the catchment is converted to runoff. Depending on the prevailing ‘wetness conditions’ of the catchment at the commencement of the storm (*i.e., the antecedent wetness conditions*), some of the rainfall may be lost to the groundwater system through infiltration into the soil, or may be intercepted by vegetation and stored. This component of the overall rainfall is considered to be 'lost' from the system and does not contribute to the catchment runoff.

To account for rainfall losses of this nature, a rainfall loss model can be incorporated within the XP-RAFTS hydrologic model. For this study, the ***Initial-Continuing Loss Model*** was employed to simulate rainfall losses across the catchment. This model assumes that a specified amount of rainfall (*e.g., 10 millimetres*) is lost from the system to simulate initial catchment wetting when no runoff is produced, and that further losses occur at a specified rate per hour (*e.g., 1.5 millimetres per hour*). These further losses are referred to as continuing losses which aim to account for infiltration once the catchment is saturated. Both the initial and continuing losses are effectively deducted from the total rainfall over the catchment, thereby leaving the remaining rainfall to be distributed through the watershed as runoff.

As no definitive loss rate data is available for the Nepean River catchment, initial estimates of rainfall loss rates were based on data contained in previous studies and on recommendations outlined in the XP-RAFTS User Manual and documented in *'Australian Rainfall and Runoff'* (*1987 and 1998*). The loss rates were further refined during calibration of the hydrologic model using available rainfall and streamflow data. The adopted loss rates are listed in **Appendix B**.

5.2 HYDROLOGIC MODEL CALIBRATION

Flood routing models such as XP-RAFTS should be calibrated and verified using rainfall and streamflow data from specific historic flood events. Rainfall records from a major storm that caused flooding can be used as rainfall input into the model to reflect the variability of rainfall over the catchment through the course of the storm.

For model calibration, the rainfall excess is routed through the model and discharge hydrographs are generated at locations where streamflow records for the flood corresponding to the storm have been gathered. Calibration is completed by refining hydrologic model parameters (*within reasonable limits*) to achieve the best match between recorded and simulated hydrographs.

Calibration typically involves adjustment of model loss rates, roughness coefficients and lag times until a good match is obtained between the computer-generated hydrograph and the hydrograph recorded at streamflow gauging locations. A good calibration is considered to be achieved when the peak discharges, the relative timing of the peak discharges and the total volume of runoff (*hydrograph shape*) derived from simulation of an historical event "agree" with data recorded for that historic event.

Calibration is a complex process and is often hindered by hidden or intangible factors, such as geomorphic changes to the catchment over time and equipment or datum changes at gauging stations.

5.2.1 Model Calibration Data

Continuous rainfall data for specific storms is required for the calibration and verification of hydrologic computer models. This data is usually obtained from pluviometers located within the catchment that is to be modelled.

Pluviometers generate plots of the instantaneous variation in rainfall over time. Proper calibration of hydrologic models without pluviograph records is generally not achievable.

As discussed above, eight pluviometer stations are located within or in the vicinity of the study area (*refer* **Figure 3.1**). The details of these pluviometers are listed in **Table 3.2**.

As outlined in **Section 3**, a number of major floods have occurred in the study area in the last 50 years. These include floods in June 1964, June 1975, March 1978, April 1988, and August 1990. A summary of these events, as recorded at the Cowpasture Bridge stream gauging station, is listed in **Table 5.1**.

Table 5.1 SUMMARY OF HISTORIC FLOODS AT COWPASTURE BRIDGE

FLOOD EVENT	PEAK FLOOD HEIGHT[1] *[metres gauge]*	GAUGE ZERO *[mAHD]*	PEAK FLOOD LEVEL *[mAHD]*	PEAK DISCHARGE[1] *[m³/s]*	APPROX. RECURRENCE INTERVAL[2] *[years]*
1964	14.08	55.71	69.79	4500	15
1975	12.70	55.71	68.41	3030	7
1978	13.45	55.71	69.16	3580	12
1988	12.80	55.71	68.51	3050	7
1990	10.63	55.71	66.34	1470	3

1 Directly from gauge record for station 212900.

2 Based on Flood Frequency Analysis contained in 1995 Upper Nepean Flood Study (LCME), which included the three larger events reported to have occurred prior to 1917.

Aside from the three large events that were reported to have occurred prior to 1917, the June 1964 flood is the largest event to have been recorded in the study area and would ideally be the most suited for hydrologic and hydraulic model calibration. However, minimal suitable pluviograph rainfall data is available for this event in the downstream extents of the catchment. Similarly, recorded pluviograph rainfall data was not extensive in the catchment during the June 1975 event, nor did hydrographs include the recession limbs experienced at Camden and Wallacia.

However, good quality, continuous pluviograph data was recorded at various locations in the catchment during the three remaining flood events (*1978, 1988 and 1990*). A discussion of the suitability of the various recorded datasets for the hydrologic model calibration process is provided in the following sections.

1978 Storm Event

The 1978 flood event was the result of a continuous period of heavy rainfall that occurred over a three day period between Saturday 18th March 1978 and Monday 20th March 1978. Pluviograph rainfall data was recorded during this period at Badgerys Creek, Narellan, Pondicherry, Warragamba Met Station and Oakdale. As such, a good range of data is available for the Nepean River catchment between Menangle Weir and Wallacia and can be used to reliably represent the rainfall that occurred within the Nepean River catchment downstream of Menangle Weir during the event.

The period of heavy rainfall was experienced throughout the catchment on the morning of Saturday 18th March 1978 and ceased late on Monday 20th March 1978. Falls of between 200 and 330 millimetres were recorded at a variety of stations in the vicinity of the study area during this period. A graph displaying the cumulative rainfall recorded at the various stations during this period are presented in **Appendix C**. The steepest sections of the graphs indicate intense rainfall over a relatively short period (*i.e., a rainfall burst*).

Stream levels throughout the study area were recorded to have risen consistently across a two day period as a result of the continuous nature of this rainfall, which is likely to have occurred over a similar period in the upper catchment areas upstream of Menangle Weir. As such, a single peak level was reached at each gauge location (*Cowpasture Bridge, Wallacia*) during the event. Flood levels peaked at Cowpasture Bridge during the afternoon of Monday 20th March 1978 and then at Wallacia early on Tuesday 21st March 1978 as flows progressed downstream. At Cowpasture Bridge the peak flood level was just below the “Major” flood classification level, while at Wallacia the peak flood level was just above the “Major” flood classification level.

1988 Storm Event

The 1988 flood event was the result of a period of heavy rainfall that occurred over a three day period between Thursday 28th April 1988 and Saturday 30th April 1988.

Pluviograph rainfall data was recorded during this period at Badgery’s Creek, Brownlow Hill, Camden Park Reservoir, Narellan, Pondicherry, Warragamba Met Station and West Camden STP in the vicinity of the study area. As such, a good range of data is available for the Nepean River catchment between Menangle Weir and the Warragamba River confluence.

Moderate rainfall commenced in the study area early on Thursday 28th April 1988. Rainfall intensity remained relatively consistent in the days that followed, before increasing in intensity on Saturday 30th April 1988. Rainfall ceased in the vicinity of the study area later that evening. The observed temporal patterns across all pluviometer stations were relatively consistent; between 200 and 260 millimetres of accumulated rainfall was recorded across the three day period. This includes the high intensity period recorded during the final 10 hours of the rainfall event where between 80 and 100 millimetres of rainfall was recorded.

A graph showing the cumulative rainfall recorded during this period is presented in **Appendix C**.

Stream levels throughout the study area were recorded to have risen steeply as a result of the initial burst of rainfall on Thursday 28th April 1988. Peak levels were recorded at Cowpasture Bridge during the afternoon of Saturday 30th April 1988. Peak river levels were subsequently recorded at Wallacia during the morning of Sunday 1st May 1988 as flows progressed towards the downstream extent of the study area. At Cowpasture Bridge the peak flood level was just below the “Major” flood classification level, while at Wallacia the peak flood level was just above the “Major” flood classification level.

1990 Storm Event

The 1990 flood event was the result of a period of heavy rainfall that occurred between the Tuesday 31st July 1990 and Thursday 2nd August 1990. This event represents both the most current and most minor flood event of the three selected.

Low intensity rainfall commenced in the study area on the afternoon of Tuesday 31st July 1990. Rainfalls then increased in intensity at around 21:00 that night delivering between 70 and 100 millimetres of rainfall over the 22 hour period that followed. Another shorter burst of rainfall contributed a further 20 to 30 millimetre over the two hour period between 06:00 and 08:00 on Thursday 2nd August 1990.

A graph showing the cumulative rainfall recorded during this period is presented in **Appendix C**.

Stream levels throughout the study area began to rise on Wednesday 1st August 1990 as a result of local rainfall and rainfall from the upper catchment areas above Menangle Weir. Peak levels were recorded at Camden Weir during the evening on Thursday 2nd August 1990. Peak levels were recorded at Wallacia later that night. At Camden the peak flood level was above the “Moderate” flood classification level, while at Wallacia the peak flood level was just above the “Major” flood classification level. It should be noted that in this case the hydrograph at Wallacia is thought to have given an overestimate of flow due to backwater effects from flows in the Warragamba River system.

Adopted Calibration Events

An appropriate ‘calibration event’ requires the availability of both time and spatially varying pluviograph rainfall and time varying records of flood discharge at key points along the primary catchment streams.

The data presented in the preceding sections indicates that streamflow data has only been available in the Camden area on a regular basis since 1949, with the installation of a streamflow gauge at Cowpasture Bridge. A streamflow gauge at Camden Weir was installed in 1989. The earliest significant flood events to have occurred within the study area during the lifetime of these stations were the events experienced in 1964 and 1975. However, streamflow gauging data is incomplete for both the 1964 and 1975 events. For this reason, and the shortage of good quality pluviometric data, it is not possible to reliably calibrate the hydrologic model to these historic events.

Data searches established that time and spatially varying pluviometer rainfall data is available for the flood events that occurred in March 1978, April 1988 and August 1990. Similarly, a substantial amount of streamflow data is available at the three stream gauging stations located in the study area during these three events (*refer* **Table 3.3**). Accordingly, based on consideration of the availability, quality and extent of hydrologic and flood level data, it was determined that the XP-RAFTS hydrologic model should be calibrated to data recorded during these three events. A matrix summarising the rainfall and streamflow datasets used during the calibration of the events is presented in **Table 5.2**.

Table 5.2 SUMMARY OF RAINFALL & STREAMFLOW DATA USED FOR CALIBRATION

GAUGE No	STREAMFLOW GAUGE NAME	March 1978	April 1988	August 1990
212202	Wallacia	✔	✔	✔
212900	Cowpasture Bridge	✔	✔	
212216	Camden Weir			✔
GAUGE No	**RAINFALL GAUGE NAME**	**March 1978**	**April 1988**	**August 1990**
067108	Badgerys Creek	✔	✔	✔
-	Narellan	✔	✔	✔
068007	Brownlow Hill		✔	✔
-	Pondicherry	✔	✔	✔
067027	Warragamba Met Station	✔	✔	✔
-	Camden Park Reservoir		✔	
068125	Oakdale	✔		
-	West Camden STP		✔	✔

5.2.2 Calibration Simulations and Results

The temporal rainfall patterns for the March 1978, April 1988, August 1990 events were extracted and entered into the hydrologic model. The rainfall records applied for the calibration and verification of the XP-RAFTS model for the three chosen historical events are shown in **Appendix C**. The relevant rainfall record location used to define rainfall during the calibration events at each subcatchment node is tabulated in **Appendix D**.

Upstream inflow hydrographs at Menangle Weir were extracted from the previously calibrated RORB model constructed as part of the Upper Nepean River Flood Study (*1995*) for each event and incorporated directly into the XP-RAFTS model. The development and calibration of this model, for the significantly larger upstream catchment areas, was deemed to be rigorous for the three chosen events. Therefore, in this study, calibration related purely to adjusting the model parameters for the subcatchment areas located within the study area (*i.e., downstream of Menangle Weir*). It was expected that direct inflow hydrographs at Menangle Bridge would significantly contribute to flows observed within the modelled study area as the catchment upstream of Menangle Weir covers some 1,200 square kilometres. By comparison, the local subcatchment areas between Menangle Weir and Wallacia Weir only account for approximately 500 square kilometres of the total catchment (*refer* **Figure 5.1**).

In order to obtain an acceptable match between simulated and recorded flood hydrographs for the March 1978, April 1988, August 1990 floods, the parameters within the XP-RAFTS hydrologic model that define initial loss, continuing loss, average catchment roughness (*Manning's 'n' co-efficient*) and lag time between catchments were adjusted until a suitable "fit" was achieved. Although loss parameters for a given subcatchment are often varied between calibration events to obtain the best possible fit, lag parameters for a given location are assumed to be constant across all calibrated events, as are roughness parameters.

For the initial calibration stage of the hydrologic modelling, the lag times had been estimated by taking the average of the lag times calculated by the AR&R and the Bransby-William methods (*refer* **Appendix B**). These lag times were then adjusted when calibrating the model through several iterations to obtain a good "fit" to the historical hydrographs.

Initial and continuing rainfall loss rates generally affect the shape of the rising limb of the hydrograph and the flow magnitudes. At the commencement of each calibration the calibrated model assumed initial losses of 15 millimetres and continuing losses of 2.5 millimetres per hour (*absolute*). These parameters generally produced simulated hydrograph peaks close to the historical peaks, largely due to the goodness of fit inherent in the previously calibrated hydrographs at Menangle Weir. However, some variation of the loss parameters was necessary to better match the peaks across all of the historical hydrographs. Variation of the rainfall loss rates between events is considered acceptable since the pre-flood catchment conditions (*wet vs. dry*) would realistically differ depending on the historic event.

A summary of the parameters used to calibrate the model during the three chosen events is presented in **Appendix D**, while **Table 5.3** provides a summary of the recorded and modelled peak flows at key locations during each calibration event. A series of plots comparing the modelled discharge hydrograph with respect to the recorded discharge hydrograph at each stream gauging station during each calibration event is also shown in **Appendix D**.

Table 5.3 COMPARISON OF RECORDED AND MODELLED PEAK DISCHARGES FOR HISTORIC FLOOD EVENTS

GAUGING STATION	PEAK DISCHARGE [m^3/s]					
	1978		1988		1990	
	Recorded	Modelled	Recorded	Modelled	Recorded	Modelled
Nepean River at Menangle Weir	-	3568	-	3423	-	2167
Nepean River at Cowpasture Bridge	3650	3572	3050	3503	-	-
Nepean River at Camden Weir	-	-	-	-	1502	2170
Nepean River at Wallacia	3626	3642	2925	3644	2277	2285

1978 Storm Event

Overall, the applicable rainfall data and model parameters were selected such that the best balance of fit was achieved across the event, and suitable fits were achieved at the two key downstream locations along the Nepean River (*Cowpasture Bridge and Wallacia*) (*refer* **Appendix D)**. This did result in a minor "shortfall" in peak discharge of 78 m^3/s (2*%*) at Cowpasture Bridge and an "overestimation" of 16 m^3/s (*0.3%*) at Wallacia during the peak event on Monday 20th March 1978 and Tuesday 21st March 1978, respectively.

While the majority of the characteristics of the calibrated model hydrographs were governed by the significant inflow at Menangle Weir, adjustments to losses, roughness and lag time were made to subcatchments located within the study area. The final adopted initial loss, continuing loss, roughness and lag time parameters for each subcatchment and figures comparing the recorded and modelled hydrographs for the 1978 event are presented in **Appendix D**.

It was noted that the peak discharge recorded at Cowpasture Bridge during this event was, in fact, slightly higher than the peak discharge recorded downstream at Wallacia (*refer* **Table 5.3**). This would suggest that either the peak discharge calculated for Cowpasture Bridge is overestimated for this event or that the peak flow was somehow attenuated between Menangle Weir and Wallacia during the event, perhaps due to the relative timing of local versus upstream inflows.

The originally modelled loss rates of 15 mm (*initial*) and 2.5 mm/hr (*continuing*) resulted in yet a higher peak discharge at Wallacia of 3842 m^3/s (*refer sensitivity test for low losses hydrograph in* **Appendix D**).

Accordingly, higher loss parameters of up to 60 mm (*initial*) and 10 mm/hr (*continuing*) were trialled in order to account for the apparent attenuation/storage within the Camden area between Menangle and Wallacia and thereby reduce the discharge at Wallacia Weir as close as possible to the recorded value of 3626 m^3/s. The sensitivity tests suggested that the peak discharge at Wallacia was somewhat sensitive to variation in the loss parameters during this particular event. The higher loss values provided a more suitable fit for this event and were used in the final calibration simulation (*refer 1978 event hydrographs in* **Appendix D**).

It was also noted that a minor secondary peak was recorded at Cowpasture Weir on Tuesday 21st of March 1978. This was not captured in the falling limb of the hydrograph as the modelled rainfall pattern upstream of the study area did not account for an additional minor rainfall burst that appeared to occur in the mid to upper catchment area. At Wallacia, the single peak discharge estimation closely matched the shape of the recorded hydrograph.

The overall fits achieved at these locations are considered to be appropriate and are typically equivalent to or better than the corresponding fits achieved for this event as part of the Upper Nepean Flood Study (*LMCE, 1995*).

1988 Storm Event

Overall, the applicable rainfall data and model parameters were selected such that the best balance of fit was achieved across the event, and suitable fits were achieved at the two key downstream locations along the Nepean River (*Cowpasture Bridge and Wallacia*) (*refer* **Appendix D)**. This resulted in an "overestimation" in peak discharge of 453 m^3/s (15*%*) at Cowpasture Bridge and an "overestimation" of 719 m^3/s (*25%*) at Wallacia during the peak event on Saturday 30th April 1988 and Sunday 1st May 1988, respectively.

As indicated above, while the majority of the characteristics of the calibrated model hydrographs were governed by the inflows at Menangle Weir, adjustments to losses, roughness and lag time were made to subcatchments located within the study area in order to attain a better "fit" at the two local gauging stations. The final, adopted initial loss, continuing loss, roughness and lag time parameters for each subcatchment and figures comparing the recorded and modelled hydrographs for the 1988 event are presented in **Appendix D**.

While the peak discharge levels are overstated at both locations within the model, the overall timing and shape are reliably represented. However, it was again noted that the peak discharge recorded at Cowpasture Bridge during this event was slightly higher than the peak discharge recorded downstream at Wallacia (*refer* **Table 5.3**). This again suggests that either the peak discharge calculated for Cowpasture Bridge is overestimated during the event or that the peak flow is attenuated between Menangle Weir and Wallacia.

Further to this, the peak discharge of the hydrograph imported from the previous study (*at Menangle Weir*) was significantly higher than the recorded value at Cowpasture Bridge (*refer* **Table 5.3**). Accordingly, initial loss parameters as high as 50 mm were tested during the calibration process in an attempt to achieve a discharge at Wallacia Weir as close to the recorded value as possible. However, the peak discharge at Wallacia proved to be insensitive to the loss parameter values tested (*refer 1988 event hydrographs* **Appendix D**). Accordingly, more "typical" loss rates of 15 mm (*initial*) and 2.5 mm/hr (*continuing*) were again used in the final calibration simulation.

The overall fits achieved at these locations are considered appropriate and are typically equivalent to or better than the corresponding fits achieved for this event as part of the Upper Nepean Flood Study (*LMCE, 1995*).

1990 Storm Event

The August 1990 flood event differed from the two previous calibration events in that the event was primarily the result of lower rainfall totals overall. An additional burst of rainfall specific only to the lower catchment was also modelled.

Overall, the applicable rainfall data and model parameters were selected such that the best balance of fit was achieved across the event, and suitable fits were achieved at the two key downstream locations along the Nepean River (*Camden and Wallacia*) (*refer* **Appendix D)**. This resulted in an "overestimation" in peak discharge of 673 m^3/s (*45%*) at Camden Weir, but only 8 m^3/s (*0.4%*) at Wallacia during the peak of the event on Thursday 2nd August 1990 and Friday 3rd August 1990, respectively.

Of particular note in this case are the peak discharge values observed at Camden Weir, which appear to be inconsistent with the recorded and modelled totals at the gauging station location downstream at Wallacia and with the modelled totals further upstream at Menangle Weir (*refer* **Table 5.3**). This would seem to suggest an error in the recordings or in the rating curve used to derive discharge values during this event.

Aside from this discrepancy, the overall fits achieved at these locations are considered to be appropriate and are typically equivalent to or better than the corresponding fits achieved for this event as part of the Upper Nepean Flood Study (*LMCE, 1995*). It should again be noted that the model results proved to be largely unaffected by any variance in loss parameters (*refer 1990 event hydrographs* **Appendix D**) and so typical acceptable values of initial and continuing loss were again used in the final calibration simulations.

5.2.3 Adopted Hydrologic Model Parameters

The values of initial loss, continuing loss, roughness and lag time parameters that are to be adopted for each subcatchment in the later design simulations are also presented in **Appendix D**.

As outlined above, sensitivity testing undertaken during the calibration process revealed that the selection of loss parameters in the subcatchment areas between Menangle Weir and Wallacia Weir generally did little to impact the peak discharge values the calibration locations (*Cowpasture Bridge, Camden and Wallacia Weir*) during the events calibrated. The exception to this was observed at Wallacia Weir during the 1978 calibration event, when higher parameters of 60 mm initial loss and 10 mm/hr continuing loss provided a better fit than the more “typical” values of 15 mm initial loss and 2.5 mm/hr continuing loss.

As no streamflow data exists along the tributaries within this area, direct calibration of these subcatchments is not possible. Accordingly, guidance contained within Australian Rainfall and Runoff (*1987*) was used in order to estimate more conservative “typical” values of initial and continuing losses; an initial loss value of 15 mm and a continuing loss value of 2.5 mm/hr were selected (*refer* **Appendix D**).

6. HYDRODYNAMIC MODELLING

6.1 GENERAL

One of the most important outcomes from the study is the determination of peak flood levels and velocities for a range of design floods. This information can be used to determine hydraulic categories (*floodway, flood storage and flood fringe*) and the variability in flood hazard across the floodplain. It will assist Camden Council in future land use planning and in the assessment of development proposals.

As outlined in **Section 2.3**, computer models can be used to simulate flood behaviour and quantify key flood characteristics such as flood levels, flow velocities, floodwater depths and flood hazard at selected points of interest. The TUFLOW hydraulic modelling software package was chosen as the tool for this purpose and was applied to develop a computer model for the Nepean River and its tributaries within the study area.

TUFLOW utilises a finite difference, two-dimensional (*2D*) approach based around a regular grid. This grid serves as the basis on which the continuity and conservation of momentum equations (*Saint Venant equations*) are solved. Constraints to the equations, such as the catchment topography, catchment roughness, inflow and outflow boundaries or links to one-dimensional (*1D*) elements are all included in the model as exports from a GIS database. This database approach makes TUFLOW a powerful tool for developing complex one-dimensional (*1D*), two-dimensional (*2D*) and linked 1D / 2D flood models.

The upstream end of the area to be modelled is defined by the location of Menangle Weir, approximately 4.7 kilometres upstream of the location where the Nepean River crosses the LGA boundary of Camden Council. The size and shape of the study area is such that it is practical to create a single two-dimensional model network that incorporates the entire area at a suitable level of resolution (*refer* **Figure 6.1**).

Development of the computer flood model was carried out over several stages that addressed the different processes of flood hydrology (*conversion of rainfall to runoff*) and flood hydraulics (*the routing of runoff*). The methodology that was employed to develop the flood model involved the following:

- Collate all available topographic data and develop a digital terrain model of the area that is to be covered by the flood model. This data is converted to a grid with specified dimensions within the 2D network of the TUFLOW model simulations.

- Define Nepean River channel using surveyed cross-section data.

- Augment the digital terrain model to include important topographical features that may not have been included in the aerial survey data or that may be too fine to be easily “picked up” by the size of the grids within the TUFLOW network. For example, levees or flood walls with widths of less than 5 metres are typically not adequately recognised within model networks with grid sizes of between 5 and 10 metres. Therefore, topographic information needs to be manually defined to override the default elevations at vital locations.

- Use aerial photography and other GIS data to define land-use areas for definition of hydraulic roughness throughout model area.
- Calibrate and verify the flood model to historic flood events such as the March 1978, April 1988 and August 1990 flood events.

6.2 HYDRODYNAMIC MODEL DEVELOPMENT

6.2.1 Available Hydrographic and Topographic Survey Data

A hydrodynamic model is developed from data that defines the topography of the waterways and floodplain areas. It also needs to incorporate critical hydraulic controls such as bridges, culverts and roadway embankments that may influence the downstream movement of floodwaters. Accordingly, the Light Detection and Ranging (*LiDAR*) data recorded in the study area between 25th February 2011 and 23rd March 2011 and hydrographic survey of the Hawkesbury Nepean Valley undertaken in the study area throughout 2011 and 2012 (*refer* **Section 3.3.2**) was used to generate the topographic data used within the hydrodynamic model network.

6.2.2 2D Model Network Development

The hydrodynamic was required to simulate flows within the Nepean River from Menangle Weir (*4.5 kilometres upstream of Camden Council's LGA boundary*) to the river's confluence with the Warragamba River near Warragamba Park. Therefore, the model network was defined to incorporate the Nepean River and all potential floodplain areas within this reach of the river. Furthermore, the five major tributaries of the river that exist within Council's LGA boundary were modelled within agreed extents (*refer* **Figure 6.1**), viz.:

- Navigation Creek near Macarthur Circuit (*LGA boundary*);
- Sickles Creek near Benwerrin Crescent (*LGA boundary*);
- Matahil Creek East at Wire Lane (*LGA boundary*);
- Matahil Creek West at Westbrook Road (*LGA boundary*);
- Narellan Creek at Kirkham Lane (*upstream from this point to be modelled separately*);
- Cobbitty Creek at The Northern Road; and
- Bringelly Creek at the dam adjacent to the LGA boundary.

Two additional tributaries within the study area (*but outside of the LGA boundary for Camden Council*) were also specifically defined within the model network, viz.:

- Foot Onslow Creek (*500 metres upstream of confluence with Nepean River*) and;
- Mount Hunter Rivulet (*downstream of Downes Bridge*).

It should be noted that the above extents were used as the minimum required modelled extents for these watercourses. Where it was more convenient or otherwise necessary to extend the model further upstream, the extent of the hydrodynamic model was modified to accommodate the overall requirements of the model in achieving its task of modelling the floodplain of the Nepean River. For instance, the modelled extent of Narellan Creek was extended upstream of Kirkham Lane to The Northern Road in the TUFLOW model, as flooding is likely to back up towards the elevated ground along the road during the design flood events. In such cases, the modelled inflows from the hydrologic model at the required boundary locations remained the same and the variations to the hydrodynamic network merely reflected the minor adjustments required to properly distribute floodwaters from the greater Nepean River system.

Model Topography & Bathymetry

The floodplain topography within the study area is generally well represented by the terrain model constructed using the available LiDAR data. Accordingly, the two-dimensional model network was primarily developed using this data-set. As discussed in **Section 3.3**, levels recorded within the streams in the model network were assumed to inadequately represent the stream bed levels as it was demonstrated that the volumes of water present within the streams were significant at the time the data was recorded. Accordingly, a hydrographic surface of the main Nepean River channel needed to be created through the use of data point interpolation using the surveyed hydrographic cross-section data for the river. This surface was used to complement the LiDAR within the river channel areas (*refer* **Figure 6.2**).

Analysis of the LiDAR data within the seven tributaries listed above also revealed that small areas of ponding and inadequate definition existed within the LiDAR data-set along these stream channels. In such cases, the topography of the channels could be adjusted by defining the banks of the channel and interpolating creek bed levels along the channel using nearby data.

These modifications were generally minor and adjustments were made in the interests of completeness. However, the channel of Narellan Creek downstream of The Northern Road appeared to contain several sections of notably deeper water in the LiDAR dataset. This was confirmed by analysis of the available aerial photography data. Although the water within the channel appeared to be no more than a metre deep in most of the affected locations, an alternative source of data was required to better estimate the topography of the creek bed in these locations. This was achieved by interpolating cross-section data for the stream that had previously been used in the MIKE11 one-dimensional model created as part of the Upper Nepean River – Tributary Flood Studies (*1997 & 1998*).

The combined topographic data was then used to define a grid within the 2D network of the model which consisted of square grids with a width of 8 metres. TUFLOW further divides each grid into four smaller squares in order to perform its hydraulic calculations. Therefore, the topography is eventually sampled at 4 metre intervals. Trial simulations of a preliminary model network setup indicated that the definition of flowpaths and terrain objects was generally adequate at this resolution.

The extent of the two-dimensional section of the TUFLOW model network is shown in **Figure 6.1**.

Floodplain Roughness

Main channel and overbank roughnesses were estimated for the study area from aerial photograph analysis and field observations of channel and floodplain vegetation density. The initial roughness values adopted were determined by comparing vegetation density and soil types observed in the field, with standard photographic records of stream and floodplain condition for which roughness values are documented. A total of seven roughness categories were defined throughout the model network. The final selection of these parameters is discussed in **Section 6.3**.

Hydraulic Features

Further augmentation of the topographic data was also required in order to ensure that any important topographical features that may not have been captured in the LiDAR data-set or that may have been too fine to be “picked up” within the 8 metre grid size were adequately defined. It is also important that the crest levels are used as the elevation value for the representative grid cell for such features. Therefore, locations where such adjustment of topographic information was required were defined during the TUFLOW network setup. Locations that received particular attention within the study area included the tributaries within Camden Council’s LGA area (*Navigation Creek, Narellan Creek, Matahil Creek, Sickles Creek, Cobbitty Creek and Bringelly Creek catchments*), weir structures along the main Nepean River channel, the railway and the Former National Equestrian Sports Centre near Menangle Park and abutments to the north of Camden. The topography incorporated into the TUFLOW model network is shown in **Figure 6.2**.

The locations of all bridge crossings, culverts and other structures across watercourses and flowpaths within the model were determined using the available LiDAR data, aerial photography and online topographic sources such as Bing Maps, Google Street View and the New South Wales Government’s Six Maps database. A total of 27 potential structures were identified, as listed in **Table 6.1** and shown in **Figure 6.3**.

The Camden Valley Way / Camden Bypass crossing of the Nepean River in Camden, known as the Macarthur Bridge, consists of an elevated bridge structure which covers some one kilometre of the river channel and floodplain. As the bridge was found to have its deck levels above the predicted peak modelled flood levels, it was not deemed necessary to model constrictions through the bridge pylons as a specific “hydraulic structures” in the model.

Table 6.1 IDENTIFIED STRUCTURES WITHIN HYDRODYNAMIC MODEL EXTENT

WATERCOURSE	TYPE	NAME / LOCATION	SUBURB	MODELLED?
Bringelly Creek	Bridge	Cut Hill Road	Cobbitty	Yes
Nepean River	Bridge	Cobbitty Bridge	Brownlow Hill	Yes
Nepean River	Bridge	Macquarie Grove Road	Camden	Yes
Nepean River	Bridge	Cowpasture Bridge (Argyle Street)	Camden	Yes
Nepean River	Bridge	Macarthur Bridge (Camden Bypass)	Camden	*No*
Nepean River	Footbridge	(adjacent to) Chellaston Street	Camden	Yes
Mt Hunter Rivulet	Bridge	Lower Mt Hunter Bridge (Werombi Road)	Brownlow Hill	Yes
Sickles Creek	Culvert	Werombi Road	Ellis Lane	Yes
Sickles Creek	Culvert	Smalls Road	Grasmere	*No*
Matahil Creek East	Culverts	(adjacent to) Onslow Park	Camden	*No*
Matahil Creek East	Bridge	Cawdor Road	Camden	Yes
Wetland adjacent to Matahil Creek East	Bridge	Burragarong Road	Mt Hunter	Yes
Matahil Creek East	Culvert	Burragorang Road	Mt Hunter	Yes
Matahil Creek East	Bridge	(adjacent to) Ron Dine Memorial Reserve	Camden South	Yes
Cobbitty Creek	Culvert	Cut Hill Road	Cobbitty	Yes
Cobbitty Creek	Culvert	Chittick Lane	Cobbitty	*No*
Cobbitty Creek	Culvert	Cobbitty Road	Oran Park	Yes
	Railway underpass	Racecourse Avenue	Menangle	Yes
	Culvert	Springs Road	Menangle	Yes
Matahil Creek West	Bridge	Burragarong Road	Bickley Vale	Yes
Matahil Creek West	Culvert	Sheathers Lane	Grasmere	Yes
Foot Onslow Creek	Bridge	Macarthur Bridge (Woodbridge Road)	Menangle	Yes
Narellan Creek	Bridge	Kirkham Lane	Kirkham	Yes
Mt Hunter Rivulet	Culvert	Downes Bridge (Brownlow Hill Loop Road)	Brownlow Hill	Yes
Nepean River	Bridge	Menangle Bridge (Menangle Road)	Menangle	Yes
Nepean River	Bridge	Blaxland Crossing Bridge (Silverdale Road)	Wallacia	Yes
Navigation Creek	Bridge	Elizabeth Macarthur Avenue	Camden Park	*No*

However, the abutments / pylons of the structure were included in the model network by increasing the hydraulic roughness value of the stream in the vicinity of the crossing, in addition to their representation in LiDAR topographic data.

Three small culvert structures at Smalls Road in Grasmere, Onslow Park in Camden and Chittick Lane in Cobbitty and a small bridge structure crossing Navigation Creek along Elizabeth Macarthur Avenue in Camden Park were identified to occur within low lying embankments in the network. These structures were all deemed to be designed to convey only small volumes during low flow conditions. The associated embankments are likely to be readily overtopped during flood events and the culverts themselves are unlikely to provide any significant conveyance capacity. Accordingly, these culvert structures were not specifically modelled within the hydrodynamic network.

The remaining 22 structures were predicted to be submerged during some or all of the design flood events and it was necessary to specifically include the hydraulic characteristics of these structures in the final hydrodynamic model network. Data was entered into the TUFLOW network that defined the dimensions of the structure (*e.g., bridge deck level or culvert dimensions*) as well as a corresponding rise in the hydraulic roughness value of the stream in the vicinity of the crossing, where applicable.

Six weir structures were also identified along the Nepean River as it passes through the model network, as listed in **Table 6.2**. The crest levels of these structures was approximated using the available LiDAR data in conjunction with the aerial photography; the crests were generally visible in the photography, allowing their levels to be estimated using the corresponding bank levels. The weirs were then modelled within the TUFLOW network by simply raising the topography across the stream (*by one grid width*) at this location.

Table 6.2 IDENTIFIED WEIRS WITHIN HYDRODYNAMIC MODEL EXTENT

WATERCOURSE	NAME / LOCATION	CREST LEVEL *(mAHD)*	MODELLED?
Nepean River	Camden Weir	57.0	Yes
Nepean River	Sharpes Weir	54.1	Yes
Nepean River	Cobbitty Weir	50.5	Yes
Nepean River	Mt Hunter Rivulet Weir	49.5	Yes
Nepean River	Brownlow Hill Weir	48.0	Yes
Nepean River	Theresa Park Weir	46.5	Yes
Nepean River	Wallacia Weir	26.5	Yes

6.2.3 Model Boundary Conditions

Flood models require upstream inflow boundaries to define stream discharges entering the model at the upstream limits, local inflows to define runoff that enters the model from subcatchments located within the network itself and a relationship to define the hydraulic conditions at the downstream limit. The locations of these boundaries are shown in **Figure 6.4**.

Upstream Inflow Boundaries

As discussed in **Section 5**, the upstream boundary conditions for the hydrodynamic model are provided by discharge hydrographs generated from hydrologic modelling of the upstream subcatchments. Discharge hydrographs were generated at each of the upstream inflow locations identified in **Figure 6.4** from the results of the hydrologic modelling that was undertaken using the XP-RAFTS model. These hydrographs generally enter the model network along boundary lines that are defined across the watercourse channels at the required locations. This method has been employed at ten of the upstream inflow locations.

At locations where the subcatchment definition or local topography determine that the use of boundary lines to define upstream inflows is difficult or inappropriate, inflow hydrographs can be directed to enter the model network via small local inflow polygons. This method has been employed at five upstream inflow locations.

The most notable of these locations is the downstream inflow from Narellan Creek. The Narellan Creek subcatchment is highly urbanised and is being modelled as part of a separate study. Prior to commencement of both projects it was agreed that the interface between the two models would be located along Kirkham Road and that inflows from the downstream end of the Narellan Creek XP-RAFTS model would enter the larger Nepean River model along this interface. However, initial testing of the TUFLOW hydrodynamic model revealed that flooding from the Nepean River is likely to would overtop Kirkham Lane during flood events and inundate much of the floodplain area between Kirkham Lane and The Northern Road to the northeast. Accordingly, the TUFLOW model network was extended to include the area between Kirkham Lane and The Northern Road, and a local inflow polygon was used to input the discharge hydrograph from the Narellan Creek catchment into the creek within this area.

The upstream inflow locations and the method employed to introduce the corresponding inflow hydrographs into the model network at these locations is summarised in **Table 6.3**. The XP-RAFTS model nodes corresponding to each of these inflow locations are also listed.

Local Inflow Boundaries

Runoff that is generated within subcatchments downstream of the upstream boundaries enter the model as local inflows directly into the network. The local inflow polygons are shown in in **Figure 6.4**, which represent those parts of the corresponding subcatchment polygon that intersect with the TUFLOW model network. The discharge hydrograph calculated for each subcatchment is distributed directly into watercourses within the inflow polygons.

Table 6.3 UPSTREAM BOUNDARY CONDITIONS FOR TUFLOW MODEL

WATERCOURSE	LOCATION *(refer Figure 5.2)*	XP-RAFTS MODEL NODE *(refer Figure 4.1)*	INFLOW METHODOLOGY
Nepean River	Menangle Weir	(MENANGLEIN)	Boundary Line
Navigation Creek	Near Macarthur Circuit	NAVI B	Boundary Line
Matahil Creek East	Wire Lane	MAT E DMY	Boundary Line
Matahil Creek West	Westbrook Road	MAT W A	Boundary Line
Sickles Creek	Benwerrin Crescent to Smalls Road	SICKLES A	Local Polygon
Narellan Creek	The Northern Road to Kirkham Lane	(NARELL IN)	Local Polygon
Mt Hunter Rivulet	West of Silverwood Road	MT H C DMY	Boundary Line
Cobbitty Creek	The Northern Road	COBBITTY A	Boundary Line
Wattle Creek	North of Taylor Place	UNNMD1 DMY	Boundary Line
Unnamed Creek 2	Coates Park Road	UNNAMED2	Local Polygon
Eagle Creek	West of McKee Road	EAGLE	Boundary Line
Forest Hill Creek	Nepean River	FOREST	Boundary Line
Bringelly Creek	LGA boundary	BRI C1 DMY	Boundary Line
Duncans Creek	Silverwood Avenue	DUNCANS A	Local Polygon
Jerrys Creek	Nepean River	JERRYS	Local Polygon

Downstream Boundary

For hydrodynamic models located within inland areas, the downstream boundary of the model is required to create an interface within a flowing river system that can effectively remove flows from the model while not impacting on upstream hydraulic behaviour or on the overall stability of the model itself. This is typically achieved by specifying a data table that defines the relationship between river level and discharge at that location. Recorded streamflows or rating curve information can be used. However, no such information is available at the downstream end of the study area.

Accordingly, a normal-depth stage-discharge relationship was derived using a sample cross-section of the river at the downstream boundary location using the Manning Formula, viz:

$$Q = A\,R^{2/3}\,S^{1/2}\,n^{-1}$$

where Q = discharge [m^3/s]

A = cross-sectional area [m^2]

n = Manning's roughness co-efficient

S = slope of channel [m/m]

R = hydraulic radius [m]

6.3 TUFLOW MODEL CALIBRATION

Calibration and verification of the hydrodynamic flood model is an important step in the model development process. If an acceptable calibration of the model to recorded events can be achieved, it ensures the reliability of the results of the subsequent design flood simulations.

In order to maintain consistency with the hydrologic modelling, calibration of the hydrodynamic model was undertaken for the March 1978, April 1988 and August 1990 historical floods. Streamflows derived from hydrologic modelling of rainfall-runoff processes across the upper and local subcatchment areas for each of these historic storm events were simulated in the hydrodynamic flood model.

Flood levels generated from the simulations were then compared to recorded gauge levels and historical flood recollections. Calibration of the model was achieved by adjusting floodplain roughness parameters within acceptable limits to obtain the best 'fit' between simulated and recorded peak flood levels.

6.3.1 Available Historic Flood Level Information

Recorded stream level information was available for the Nepean River at the Cowpasture Bridge, Camden Weir and Wallacia gauging stations (*refer* **Table 3.3**) for the March 1978, April 1988 and August 1990 historical floods.

As part of the study, a flooding questionnaire was distributed to residents within the study area. A total of 127 responses were received to the questionnaire, which requested respondents provide details of any specific recollections they may have with respect to observed peak flood levels or known debris marks. From the received responses, eight observations were provided that could reliably be used to determine peak flood levels or extents during the calibration process for the three chosen historical flood events.

Similar data was also obtained during the Upper Nepean River Flood Study (*1995*). However, on inspection of the data, many of the readings appeared to be inaccurate. For example, differences of several metres were often reported at similar locations during the same event. Notwithstanding this, a selection of the most representative data was chosen from this set in order to complement the data within areas of interest.

The historical flood level data chosen for calibration to the three events is summarised in **Table 6.4**.

Table 6.4 HISTORIC FLOOD LEVEL INFORMATION USED IN MODEL CALIBRATION

EVENT	LOCATION	PEAK FLOOD LEVEL *[mAHD]*	SOURCE OF INFORMATION
March 1978	U/S Macarthur Bridge, Camden	69.9	Previous study[1]
March 1978	5 Wilkinson Street, Camden	69.2	Community questionnaire
March 1978	Cowpasture Bridge	69.2	Stream level records
March 1978	44 Engesta Avenue, Camden	68.8	Community questionnaire
March 1978	161 Cut Hill Road, Cobbitty	62.9	Previous study[1]
March 1978	Cut Hill Reserve, Cobbitty	62.5	Previous study[1]
March 1978	Wallacia Weir	42.2	Stream level records
April 1988	Arndell Street, Camden South	69.7	Previous study[1]
April 1988	22 Lerida Avenue, Camden	69.0	Community questionnaire
April 1988	1 Belgenny Avenue, Camden	68.9	Community questionnaire
April 1988	11 Pindari Avenue, Camden	68.4	Community questionnaire
April 1988	19 Pindari Avenue, Camden	68.2	Community questionnaire
April 1988	Cowpasture Bridge	68.5	Stream level records
April 1988	8 Edward Street, Camden	68.3	Community questionnaire
April 1988	17 Elizabeth Street, Camden	68.2	Community questionnaire
April 1988	300 McKee Road, Theresa Park	60.4	Previous study[1]
April 1988	340 McKee Road, Theresa Park	60.2	Previous study[1]
April 1988	409 McKee Road, Theresa Park	59.6	Previous study[1]
April 1988	Wallacia Weir	40.8	Stream level records
August 1990	Arndell Street, Camden South	68.4	Previous study[1]
August 1990	Cowpasture Bridge	66.3	Stream level records
August 1990	Mt Hunter Rivulet Weir	62.1	Previous study[1]
August 1990	Wallacia Weir	39.2	Stream level records

1 Upper Nepean River Flood Study (DLWC 1995)

6.3.2 Boundary Condition Data

The available historic pluviograph rainfall data was routed through the XP-RAFTS hydrologic model as part of the model's calibration process (*refer* **Section 5.2**). The final, calibrated version of the XP-RAFTS hydrologic model was then used to generate inflow hydrographs for the March 1978, April 1988 and August 1990 at the appropriate upstream and local inflow boundary locations to the TUFLOW model.

6.3.3 Calibration Simulations

The TUFLOW flood model was used to simulate the March 1978, April 1988 and August 1990 flood events.

During the calibration process, modifications were made to the model network, primarily the adjustment of roughness parameter values. These changes were made to achieve an improved "fit" between simulated and recorded flood levels throughout the study area.

It was noted during the initial model simulations that the resulting peak flood levels in the vicinity of Wallacia were highly sensitive to the tailwater levels assumed at the downstream limit of the model (*i.e., at the confluence of the Nepean River and the Warragamba River just downstream of Wallacia*). Comprehensive testing of the hydraulic mechanisms of the model suggested that the tailwater level impacted the peak flood levels in this area significantly more than variation in roughness.

Peak level information during the three historical events was available for the Warragamba River confluence (*Upper Nepean River Flood Study, 1995*). However, the timing of the rise and fall of river levels in the Warragamba River during these events was not available. Accordingly, in the absence of accurate time-varying level data at the confluence of the two rivers, a series of sensitivity tests were undertaken whereby the tailwater levels in the Warragamba River were varied in relation to the recorded levels in the Nepean River at Wallacia.

A number of peak levels were tested (*using the peak levels from the previous study as a guide*) in conjunction with the overall hydrograph shape recorded at Wallacia to define the assumed variation of the tailwater level throughout the course of the event. The final calibration simulations utilised stage-time relationships at the downstream end of the model that resulted in the most suitable peak flood levels being achieved at the Wallacia gauging station location. The final peak tailwater levels were in general agreement with the levels assumed in the Upper Nepean Flood Study (*1995*).

A comparison of the final calibrated model results and the recorded flood levels at the Cowpasture Bridge, Camden Weir and Wallacia Weir gauging station locations is provided in **Table 6.5**. Comparisons between the recorded and simulated flood levels at the locations listed in **Table 6.4** for the three events are shown in the figures contained in **Appendix E**.

Table 6.5 COMPARISON OF RECORDED AND MODELLED PEAK LEVELS FOR HISTORICAL FLOOD EVENTS

GAUGING STATION	PEAK FLOOD LEVEL *[mAHD]*					
	1978		1988		1990	
	Recorded	Modelled	Recorded	Modelled	Recorded	Modelled
Nepean River at Cowpasture Bridge	69.2	69.0	68.5	68.5	66.3	66.5
Nepean River at Camden Weir	-	-	-	-	66.1	66.3
Nepean River at Wallacia Weir	42.2	42.1	40.8	40.7	39.2	39.2

6.3.4 Discussion

The modelling results for all three calibration events (*refer* **Appendix E**) indicate that a generally good agreement between recorded and simulated flood levels was obtained using the TUFLOW model. The differences between recorded gauge levels and simulated levels at Cowpasture Bridge, Camden Weir and Wallacia Weir were within 200 millimetres for all three events simulated.

As discussed in **Section 6.3.3**, levels in the Wallacia area were highly influenced by the adopted tailwater levels at the confluence of the Nepean River and the Warragamba River. The tailwater level patterns applied at the downstream boundary resulted in peak flood levels at the Wallacia Weir gauging station that closely matched those recorded for all three events.

In order to ensure uniformity between the three events, a single set of final roughness parameter values was chosen to represent the modelled area for the three selected events. Accordingly, it was necessary to select parameters that resulted in the best overall “fit” for the modelled results in relation to the observed data across the three events. As a result, the agreement between observed and modelled peak flood levels was poor in a small number of locations for each event. However, these instances mostly related to data that was less reliable or based on estimated levels or flood extents. Recorded gauge levels are assumed to provide more reliable indications of peak flood levels than other sources of observed data, and decent matches were achieved in the modelled results at these locations for all three events.

In conclusion, it is considered that appropriate fits were achieved for the majority of the observations utilised in the calibration process. Therefore, it is considered that the TUFLOW model provides a reliable and suitably calibrated tool for the simulation of design floods for the study area. The final roughness parameter values applied to all three calibrated events are listed in **Table 6.6**. These values are in general agreement with the calibrated roughness parameters derived during the Upper Nepean River Flood Study (*1995*), where values for the one-dimensional cross-sections along the modelled reaches of the river were generally between 0.050 and 0.065.

Table 6.6 FINAL ROUGHNESS PARAMETER VALUES USED IN TUFLOW MODEL

CATEGORY	ROUGHNESS PARAMETER *[Manning's 'n']*
Urban Areas	0.08
Open Watercourses	0.04
Heavily Vegetated Creeks	0.06
Grass / Pasture / Brush	0.06
Forested Areas	0.10
Roads	0.02
Bridge Abutment Areas	0.08

The distribution of these roughness parameter values within the TUFLOW model network are shown in **Figures 6.5** and **6.6**.

7. DESIGN FLOOD ESTIMATION

7.1 GENERAL

Design floods are hypothetical floods that are commonly used for planning and floodplain risk management investigations. Design floods are based on statistical analysis of rainfall and flood records and are defined by their probability of occurrence. For example, the 100 year recurrence flood can also be expressed as the 1% Annual Exceedance Probability (*AEP*) flood. That is, there is a 1% chance of the 100 year recurrence flood occurring in any given year. The same flood probability may also be expressed as the 100 year Average Recurrence Interval (*ARI*) flood.

The X% AEP terminology has been adopted for the design flood mapping prepared as part of this flood study, as per the following:

- 50% AEP refers to the 2 year ARI event;
- 20% AEP refers to the 5 year ARI event;
- 5% AEP refers to the 20 year ARI event;
- 1% AEP refers to the 100 year ARI event;
- 0.5% AEP refers to the 200 year ARI event; and
- 0.2% AEP refers to the 500 year ARI event.

It should be noted that there is no guarantee that the design 1% AEP event will occur just once in a one hundred year period. It may occur more than once, or at no time at all in a given one hundred year period. This is because the design floods are based upon a statistical 'average'. Further notes regarding the adopted terminology in relation to a recent discussion paper prepared as part of the AR&R revision project (*Engineers Australia, 2013*) is provided in **Appendix F**.

The Probable Maximum Flood (*PMF*) has also been modelled as an extreme event.

7.2 HYDROLOGIC MODELLING

7.2.1 Design Flood Simulations

The XP-RAFTS hydrologic model described in **Section 5** was used to simulate runoff from the subcatchments downstream of Menangle for design storm conditions. The design storm conditions were based on rainfall intensities and temporal patterns for the study area, which were derived using standard procedures outlined in *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (*1987*) (*ARR 87*). The design storm rainfall data was generated by applying the principles of rainfall intensity estimation described in Chapter 2 of ARR 87. Upstream inflows at Menangle Weir were again derived from the RORB model developed during the Upper Nepean Flood Study (*1995*).

It should be noted that Engineers Australia and the Bureau of Meteorology published updated intensity-frequency-duration (*IFD*) data and recommendations in relation to their implementation (*Engineers Australia, 2013*) during the course of the study. This included additional information in relation to the derivation and implementation of areal reduction factors (*ARFs*) (*Engineers Australia, 2013*).

However, following discussions with the OEH and Engineers Australia it was recommended that the latest information was not suitable for the study. This was primarily because the temporal patterns within the XP-RAFTS model for the catchment are based on the ARR 87 methods and are considered incompatible with the 2013 IFD data. Furthermore, the latest methods for deriving ARFs are untested and it is not recommended to integrate such ARFs with ARR 87 data. Accordingly, it was decided that the most conservative approach was to continue to adopt the IFD data and temporal patterns from the ARR 87 documentation and adopt the same ARF factors as those used in the RORB model developed during the Upper Nepean Flood Study (*1995*) and used to define flows upstream of Menangle Weir.

An estimate of Probable Maximum Precipitation (*PMP*) rainfall was also derived for the subcatchments downstream of Menangle Weir. The PMP is defined as the greatest depth of precipitation that is meteorologically possible for a given duration at a specific location. The PMP can be routed through the hydrologic model to provide discharge hydrographs for the Probable Maximum Flood (*PMF*) at specified locations throughout the subcatchments.

According to guidance contained within the '*Guidebook to the Estimation of Probable Maximum Precipitation: Generalised Southeast Australia Method*', the catchment lies within the 'GSAM Zone'. Accordingly, the GSAM methodology was employed in the study.

Again, as the upstream inflows at Menangle Weir were derived from the RORB model developed during the Upper Nepean Flood Study (*1995*), a 12 hour critical storm duration was assumed in order to achieve consistency between the previous methodology and the latest study. Likewise, the GSAM PMP rainfall depth for the catchment was assumed to be 550 millimetres in accordance with Figure 5.6 of Upper Nepean Flood Study (*1995*). The spatial distribution of the rainfall was then determined using the procedure outlined in Section 3 of the guidebook using a reference Terrain Adjustment Factor (*TAF*) of 1.3.

Critical Storm Duration

A range of storm durations were first considered to establish the critical storm duration for the locations within the study area. The critical storm duration was assumed to correspond to the duration that generated the maximum peak discharges along the Nepean River at Menangle and Camden (*the focus of interest for Council*). Both locations were tested for a range of durations for all design events. A critical storm duration of 48 hours was determined to be critical at both locations for the 50%, 20% and 5% AEP design events, while 36 hours was deemed to be critical for the 1%, 0.5% and 0.2% AEP design events.

The hydrology of the tributaries feeding to the Nepean River has been incorporated into the flood modelling for the 36 and 48 hour duration storms so that the critical flood behaviour can be determined for the Nepean River. However, it should be noted that for areas upstream of the backwater influence of the Nepean River the peak level of flooding along the tributaries will be the result of a shorter critical duration storm (*e.g., the 2 hour duration storm*). Council intends to assess the critical case for local catchment flooding as part of separate future investigations. Accordingly, the design event model results and mapping for this study have been 'clipped' to remove the sections of local tributary flooding located upstream from the Nepean River backwater.

Areal Reduction Factors

Design rainfall information (*IFD data*) is provided in *ARR 87* in the form of point rainfall intensity. Such data reflects the peak rainfall intensity during a specified design event at a single point location. However, such rainfall events are unlikely to be experienced simultaneously across large catchment areas.

An areal reduction factor (*ARF*) is the ratio between the areal average rainfall across a catchment and the point rainfall values provided within IFD charts. Applying an ARF to the point rainfall intensities provides an "adjusted" rainfall intensity to apply across the catchment when deriving design event information.

An updated methodology for deriving ARFs for Australian catchments was published during the period of the study *(Engineers Australia, 2013*). However, OEH and Engineers Australia recommendations suggested that the latest information was not suitable for use until the full revision of ARR has been completed. Furthermore, as the dominant flows in the Nepean River in the study area were derived from the RORB model developed during the Upper Nepean Flood Study (*1995*), it was decided that the areal reduction factors used during the former study should be incorporated into the current XP-RAFTS model. These factors are listed in **Table 7.1**.

Table 7.1 AREAL REDUCTION FACTORS USED IN XP-RAFTS MODEL

AEP *[%]*	**ARF** *[%]*
50	96
20	96
5	94
1	90
0.5	88
0.2	88

Discharge hydrographs were then generated throughout the catchment for all design events using the appropriate critical storm duration and the appropriate rainfall intensities, design temporal patterns and areal reduction factors.

7.2.2 Hydrologic Modelling Results

Design discharge hydrographs determined using the XP-RAFTS hydrologic model and based on the derived critical durations were used to define inflows into the TUFLOW hydrodynamic model. A summary of the peak discharges for each upstream model boundary location is provided in **Table 7.2**.

Table 7.2 PEAK UPSTREAM INFLOWS USED IN TUFLOW MODEL

LOCATION (*refer* Figure 6.4)	XP-RAFTS MODEL NODE	PEAK DISCHARGE[1] (m^3/s)						
		PMF	0.2%	0.5%	1%	5%	20%	50%
		12hr	36hr	36hr	36hr	48hr	48hr	48hr
Nepean River	(MENANGLEIN)	18421	11048	9469	8314	5220	2447	1074
Navigation Creek	NAVI B	327	143	121	108	86.0	58.4	36.3
Matahil Creek East	MAT E DMY	467	204	173	155	124	84.6	53.3
Matahil Creek West	MAT W A	80.4	33.0	28.0	25.1	21.3	14.6	9.4
Mt Hunter Rivulet	MT H C DMY	1202	447	379	339	256	169	103
Cobbitty Creek	COBBITTY A	9.1	4.8	4.1	3.7	3.1	2.3	1.6
Wattle Creek	UNNMD1 DMY	333	140	118	105	86.3	56.7	34.9
Eagle Creek	EAGLE	408	156	132	118	90.5	57.8	34.5
Forest Hill Creek	FOREST	286	111	93.1	82.9	68.8	45.6	28.0
Bringelly Creek	BRI C1 DMY	132	55.3	47.3	42.7	35.3	24.4	15.9

1. *Peak discharges listed do not necessarily occur simultaneously.*

7.3 HYDRODYNAMIC MODELLING

7.3.1 Design Flood Simulations

The TUFLOW hydrodynamic model that was developed for the project was then used to simulate each of the design flood events. However, prior to the final simulations, adjustments were made to the local topography data in order to account for changes that had occurred within the study area in the time since the calibration period. This primarily included changes to the local topography resulting from the Spring Farm development on the northern side of the Nepean River approximately four kilometres upstream of Camden. Details of these works were provided by Council.

Upstream boundary conditions were defined for each design flood based on the inflow hydrographs generated using the XP-RAFTS hydrologic model (*refer* **Table 7.2**). For example, design 1% AEP flood discharge hydrographs for river inflows were extracted from the model output and used to define the rate of flow into the TUFLOW model.

A total of ten upstream boundary inflows were adopted to input flows into the upstream extents of the flood model (*refer* **Table 7.2**). A further 35 local inflows were specified throughout the model allowing localised flows to be input into the hydrodynamic model network at the location of the corresponding subcatchment.

7.3.2 Hydrodynamic Modelling Results

Flood Levels and Extent of Inundation

The predicted extent of inundation across the floodplain for the design events have been extracted from the modelling results and are presented for the model network area in a series of maps within **Appendix G**. The figures also indicate the peak flood levels at each location in the study area via flood level contour lines. Individual figures have been included for two smaller areas within the Camden LGA area for the key design events (*5% and 1% AEP and PMF*).

Flooding in the lower reaches of Narellan Creek has been captured in the flood modelling for the Nepean River (*as a combination of backwater flooding and local flows*) and also separately for Narellan Creek as part of the Narellan Creek Flood Study (*in draft*). In this area an envelope has been taken of the maximum level of flooding between the two sets of model results. The mapping has then been ‘clipped’ along a boundary agreed with Council (*adjacent to the Nepean River*), to provide a set of flood maps for each Flood Study report with no overlap between.

The flood mapping has also been ‘clipped’ along the tributaries of Matahil Creek, Sickles Creek, Cobbitty Creek and Bringelly Creek, to remove those sections of mapping upstream from the limit of the 1% AEP backwater from the Nepean River. As discussed in Section 7.2, the peak level of flooding along tributaries (*according to the critical duration of local catchment rainfall*) will be assessed as part of separate investigations.

All flood mapping in this report has been prepared using this approach, including for flood levels, depths, hazards and hydraulic categories.

Floodwater Depths & Velocities

Peak floodwater depths were also extracted from the modelling results for each of the design flood events and are presented for the model network area in a series of maps within **Appendix H**. Individual figures have been included for two smaller areas within the Camden LGA area for the key design events (*5% and 1% AEP and PMF*). Peak floodwater flow velocities for the adopted design flood events have been superimposed over the floodwater depth plots shown in **Appendix H** as velocity vectors.

Model Validation

A final analysis of the hydrodynamic modelling results was undertaken by comparing the peak flows simulated at Cowpasture Bridge in Camden with the corresponding peak flows derived during the Flood Frequency Analysis and hydrodynamic modelling undertaken during the Upper Nepean Flood Study (*LMCE, 1995*). The results indicate that the flows at Cowpasture Bridge simulated within the derived TUFLOW model are similar to those obtained from the previous study for the 1%, 5% and 20% AEP design flood events (*refer* **Table 7.3**).

Table 7.3 COMPARISON OF MODELLED PEAK DISCHARGES AT COWPASTURE BRIDGE WITH PREVIOUS STUDY

STUDY	PEAK DISCHARGE [m^3/s]		
	1%	5%	20%
1995 – Flood Frequency Analysis[1]	7900	4900	2100
1995 – Hydrodynamic Modelling[2]	7400	4900	2200
Present Study	7550	4820	2160

1 Lyall and Macoun Consulting Engineers (1995), 'Upper Nepean River Flood Study', Table B4.1.

2 Lyall and Macoun Consulting Engineers (1995), 'Upper Nepean River Flood Study', Table 6.3.

8. FLOOD HAZARD AND HYDRAULIC CATEGORIES

8.1 GENERAL

The personal danger and physical property damage caused by a flood varies both in time and place across the floodplain. Accordingly, the variability of flood patterns across the floodplain over the full range of floods needs to be understood by flood prone landholders and by floodplain risk managers.

Representation of the variability of flood hazard across the floodplain provides floodplain risk managers with a tool to assess the existing flood risk and to determine the suitability of land use and future development. The hazard associated with a flood is represented by the static and dynamic energy of the flow, which is in essence, the depth and velocity of the floodwaters. Therefore, the flood hazard at a particular location within the floodplain is a function of the velocity and depth of the floodwaters at that location and is related to the ability to wade or drive a vehicle through the floodwaters.

The NSW Government's *'Floodplain Development Manual'* (*2005*), characterises hazards associated with flooding into a combination of three hydraulic categories and two hazard categories. Hazard categories are broken down into high and low hazard for each hydraulic category as follows:

- Low Hazard – Flood Fringe
- Low Hazard – Flood Storage
- Low Hazard – Floodway
- High Hazard – Flood Fringe
- High Hazard – Flood Storage
- High Hazard - Floodway

As a result, the manual effectively divides hazard into two categories, namely, high and low. An interpretation of the hazard at a particular site can be established from **Figure L1** and **L2** on the following page, which have been taken directly from the manual.

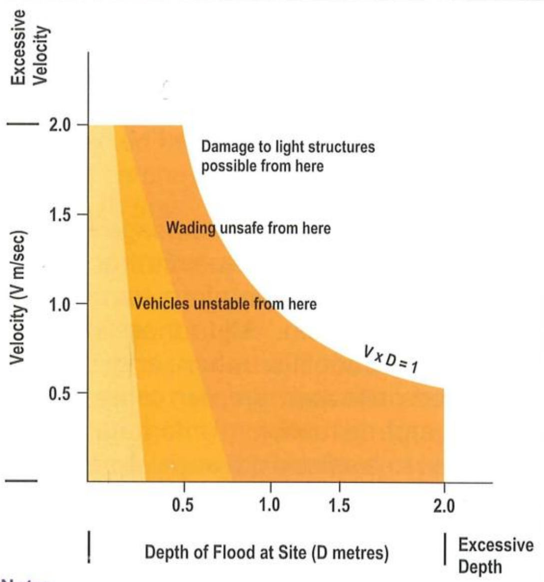
The first of these graphs shows approximate relationships between the depth and velocity of floodwaters and resulting hazard. This relationship has been used to define the provisional low and high hazard categories represented in the second of these plots.

8.2 FLOOD HAZARD

8.2.1 Adopted Provisional Hazard Categorisation

As shown in **Figures L1** and **L2**, flood hazard is a measure of the degree of difficulty that pedestrians, cars and other vehicles will have in egressing flooded areas, and the likely damage to property and infrastructure. At low hazard, passenger cars and pedestrians (*adults*) are able to move out of a flooded area. At high hazard, wading becomes unsafe, cars are immobilised and damage to light timber-framed houses would occur.

Flood hazard is categorised according to a combination of the flow velocity and the depth of floodwater. The categories are defined by lower and upper bound values for the product of flow velocity and floodwater depth.

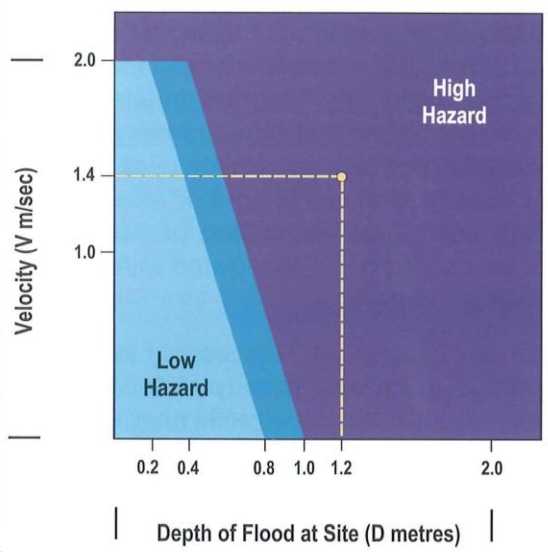


Notes

1. At velocities in excess of 2.0 m/s, the stability of foundations and poles can be affected by scour. Also, grass and earth surfaces begin to scour and can become rough and unstable
2. The velocity of floodwaters passing between buildings can produce a hazard, which may not be apparent if only the average velocity is considered. For instance, the velocity of floodwaters in a model test has risen from an average of 1 m/sec to 3 m/sec between houses.
3. Vehicle instability is initially by buoyancy.
4. At floodwater depths in excess of 2.0 meters and even at low velocities, there can be damage to light-framed buildings from water pressure, flotation and debris impact.

Derived from laboratory testing and flood conditions which caused damage.

FIGURE L1 - Velocity & Depth Relationships



Notes

The degree of hazard may be either -

- reduced by establishment of an effective flood evacuation procedure.
- increased if evacuation difficulties exist.

In the transition zone highlight by the median colour, the degree of hazard is dependant on site conditions and the nature of the proposed development.

Example:

If the depth of flood water is **1.2 m**
and the velocity of floodwater is **1.4 m/sec**
then the provisional hazard is **high**

FIGURE L2 - Provisional Hydraulic Hazard Categories

Spatial and temporal distributions of flow, velocity and water level determined from the computer modelling undertaken as part of this study were used to determine the flood hazard categorisation of the Nepean River floodplain within the study area. Interpretation of this data indicates that for large events such as the 1% AEP flood event, the majority of flooded land would fall within the high hazard category defined in the *'Floodplain Development Manual'* (*2005*).

Hence, for the purpose of understanding how the flood hazard affects existing development and areas of potential future development, it is useful to further subdivide areas falling within the high hazard category, into High Hazard, Very High Hazard and Extreme Hazard.

Similarly, the low hazard category defined in the manual has been subdivided to create a Low Hazard and a Transitional Hazard category.

8.2.2 Provisional Flood Hazard

Provisional flood hazard mapping generated for the study area simulated events is presented in **Appendix I**.

The mapping indicates that a large proportion of the floodplain would be subject to high hazard flooding during events greater than and including the 1% AEP event. This is predominantly a function of the high floodwater depths across much of the floodplain at the peak of the events, although high velocities are experienced in many areas.

8.2.3 Preliminary True Flood Hazard

The hazard represented in this mapping is provisional only. This is because it is based only on an interpretation of the flood hydraulics and does not reflect the effects of other factors that influence hazard (*see clause L6 to Appendix L of the Floodplain Development Manual*). For example, access to an otherwise low hazard area may be through a high hazard area and this may present an unacceptable risk to life and limb and as such the provisional low hazard area may be changed to high hazard.

According to the NSW Government's 'Floodplain Development Manual' (*2005*), the preparation of mapping for the true flood hazard also needs to consider other factors, including:

- The size of the flood;
- Effective warning time;
- Flood readiness of the community;
- The rate of rise of the flood waters;
- Duration of the flooding;
- Any evacuation problems that may be encountered;
- Effective flood access;
- The type of development present;

Evacuation away from areas of immediate hazard is not expected to be problematic at most locations along the tributaries. However, floodwaters are likely to block access along roads that cross these creeks and access from outside the area may not be possible for the duration of the flood event.

The duration of flooding from the Nepean River (including backwater floodwater from the river along the tributaries) is expected to be relatively long; floodwaters are not expected to recede from most properties for two or three days after the commencement of flooding during the 1% AEP event. As a result, any trapped residents may be isolated for extended periods of time and may need to be supplied with food or other provisions.

The mapping for preliminary true flood hazard has been prepared for the design 1% AEP flood event (*refer* **Appendix J**). In most locations, this mapping was derived from the provisional flood hazard mapping by removing islands of lower hazard or other areas that become substantially surrounded by high hazard floodwaters.

8.3 HYDRAULIC CATEGORIES

8.3.1 Adopted Hydraulic Categorisation

The NSW Government's *'Floodplain Development Manual'* (*2005*) also characterises flood prone areas according to the hydraulic categories presented in **Table 8.1**. The hydraulic categories provide an indication of the potential for development across different sections of the floodplain to impact on existing flood behaviour.

Table 8.1 HYDRAULIC CATEGORY CRITERIA

HYDRAULIC CATEGORY	DESCRIPTION
FLOODWAY	• those areas where a significant volume of water flows during floods • often aligned with obvious natural channels • they are areas that, even if only partially blocked, would cause a significant increase in flood levels and/or a significant redistribution of flood flow, which may in turn adversely affect other areas • they are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.
FLOOD STORAGE	• those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood • If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. • Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.
FLOOD FRINGE	▪ the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. ▪ Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

Unlike for the hazard categorisation outlined in **Section 8.2**, the *'Floodplain Development Manual'* (*2005*) does not provide explicit quantitative criteria for defining hydraulic categories. This is because the extent of floodway, flood storage and flood fringe areas is largely dependent on the geomorphic characteristics of the floodplain in question.

Although there are no specific procedures for identifying or determining hydraulic categories, a rigorous methodology involving several stages of analytical analysis in conjunction with flood modelling has been developed by Thomas & Golaszewski (*2012*). This methodology has been applied with success to similar floodplains in NSW and has been shown to provide a robust procedure for defining floodway extent. Most recently, this methodology was successfully applied to the Lower Hastings River floodplain as part of investigations for the *'Hastings Floodplain Risk Management Study*' (*2012*).

The hydraulic category mapping that was prepared for the Nepean River floodplain as part of the investigations for this study is provided in **Appendix K**.

The following sections describe the methodology that was employed to determine the hydraulic category mapping.

8.3.2 Adopted Methodology for Determination of Floodway Corridors

The adopted methodology for determination of hydraulic categories for Nepean River floodplain involved several stages of assessment that relied on analysis of all available hydraulic, topographic, cadastral and geomorphic data-sets.

Once the detailed investigations to determine the extents of floodway corridors were completed, an assessment was also undertaken to determine the extent of flood storage and flood fringe areas. Each of these hydraulic categories was then combined to develop hydraulic category mapping for the study area which can be incorporated into future mapping layers linked to Council's Development Control Plan.

A preliminary floodway extent was firstly determined based on an assessment of aerial photography, topographic data and existing 1% AEP flood modelling results. Determination of this extent or "line" considered the following:

- The location of flood storages that are readily identifiable from aerial photography;
- The location and potential impact of hydraulic controls and geomorphic features that could influence floodwater movement and flood characteristics (*e.g., velocity*);
- Mapping of contours of 'velocity-depth' product (*V x D*); and,
- Mapping of the variation in peak flow velocity.

Because of the complex nature of flooding along the Nepean River within the study area and the varied floodplain types encountered across the study area, establishment of a standard set of criteria was <u>not</u> considered appropriate for the determination of all floodway extents. For example, definition of the floodway extent based on a single target value for velocity or velocity-depth product (*V x D*) would limit the reliability of the investigation findings.

Accordingly, to ensure the assessment of floodway extent was completed reliably, the study area was divided into numerous precincts to enable assessment on a '*local*' scale.

A set of interactive flood maps was produced for each of these precincts to show key hydraulic data including the variation in V x D, peak flow velocities and peak flood depths. The results of modeling of the design 1% AEP flood were used as the benchmark for the analysis.

The interactive flood maps were used to identify areas of the floodplain representing:

- High depth and high velocities; i.e., high V x D (*generally considered floodway*);
- High depth and low velocities (*generally considered flood storage*); and
- Low depth and low velocity (*generally considered flood fringe*).

In this regard, a typical "first pass" assessment of floodway extents was undertaken to identify areas where the velocity-depth product is greater than 3 m^2/s and where flow velocities are greater than 1 m/s. The alignment of significant flow paths across the floodplain (*i.e., potential flood runners*), as inferred by the velocity and V x D contour mapping, was also considered in determining the preliminary floodway extents.

The Preliminary Floodway Extent was further verified by comparison with mapping of the width of the floodplain that would be required to convey 80% of the peak flow. Trial analyses for this project and similar floodplain risk management studies have shown a good correlation between the transitions in velocity-depth product contour mapping, geomorphic characteristics and the width of the floodplain that conveys about 80% of the flood flow. A discussion of this criteria and its appropriateness for defining floodway extent is provided in Thomas & Golaszewski (*2012*).

The width occupied by 80% of the flow was readily determined for any location within the lower reaches of the floodplain using the *Flow Extraction* tool within waterRIDE™. This width was then used to verify and adjust the Preliminary Floodway Extent where appropriate. The resultant floodway mapping is shown in **Appendix K**.

8.3.3 Adopted Methodology for Determining Flood Storage and Flood Fringe

Following determination of those areas of the floodplain categorised as floodway, investigations were focused towards identifying the remaining hydraulic categories, namely flood storage and flood fringe. As outlined in the NSW *'Floodplain Development Manual'* (*2005*), flood storage and flood fringe make up the remainder of the floodplain outside of the floodway corridor.

Flood storage areas are typically defined as those flood prone areas that afford significant temporary storage of floodwaters during a major flood. If filled or obstructed (*through the construction of levees or road embankments*) the reduction in storage would be expected to result in a commensurate increase in flood levels in nearby areas. The remaining flood prone areas not classified as floodway or flood storage are termed flood fringe.

In order to determine the boundary between flood storage and flood fringe, the variation in peak flood depths in areas outside of the floodway extent was mapped to identify areas inundated to depths of approximately 0.5 metres. A depth of 0.5 metres is considered to be upper limit of the transitionary point between flood storage and flood fringe.

In terms of the Nepean River floodplain within the study area, peak depths below 0.5 metres are generally considered to correspond to areas where negligible flow is conveyed and represent a relatively small proportion of storage for floodwaters.

In accordance with the Floodplain Development Manual (*2005*), this represents areas which are unlikely to have any significant impact on the pattern of floodwater distribution through a river and floodplain system and associated flood levels. Accordingly, the boundary between flood storage and flood fringe was defined by a peak 1% AEP flood depth of 0.5 metres.

Flood storage and flood fringe mapping for the floodplain of the Nepean River within the study area is presented within **Appendix K**.

9. PRELIMINARY FLOOD PLANNING AREA

The Preliminary Flood Planning Area within the study area has been determined for the present day situation by mapping the extent of the Flood Planning Levels which were determined according to the peak 1% AEP flood level plus a freeboard of 0.5 metres.

Furthermore, part of the study area is underlain with coal deposits and is within the South Campbelltown Mine Subsidence District. The area of interest, which encompasses an area bordered by Camden Valley Way, Narellan Road and the Nepean River, can be further divided into two subareas as follows:

- Elderslie Urban Release Area (*north of Camden Bypass*) – additional mine subsidence allowance of 1.3 metres; and
- Spring Farm Urban Release Area (*south of Camden Bypass*) – additional mine subsidence allowance of 1.6 metres.

WorleyParsons' *waterRIDE* software has been used to apply the 0.5 metre freeboard and the additional mine subsidence allowances (*where required*) to the peak 1% AEP flood levels in order to determine the Flood Planning Area outlines. The final, composite Flood Planning Area extents are presented in **Appendix L**.

10. FLOOD DAMAGES ANALYSIS

10.1 WHAT ARE FLOOD DAMAGES?

Flood damages are adverse impacts that private and public property owners experience as a consequence of flooding. They can be both tangible and intangible and are usually measured in terms of a dollar cost.

Tangible damages include direct damages such as the damage to property as a consequence of inundation (*e.g. the cost of replacing carpets*). Tangible damages can also be indirect damages such as the cost to the community of individuals being unable to get to work because they are isolated due to flooding. These costs can usually be measured and data has been gathered over many years to provide a reliable indication of the likely damage costs that can be incurred by residential, commercial and industrial property owners.

It is more difficult to quantify intangible damages. Intangible damages include less 'concrete' impacts such as the trauma felt by individuals as a result of a major flood and the associated health related impacts. Only limited data is available, but it has been stated that intangible damages could be as much or more than the tangible damage cost.

As part of the floodplain risk management process, it is necessary to determine the total damages that could be incurred as a consequence of flooding. If the total damage cost is significant, it can be argued that works or planning measures to reduce the cost can be justified.

10.2 FLOOD DAMAGE ASSESSMENT METHOD

10.2.1 Flood Damage Categories

Flood damage costs within the study area were determined based on consideration of the different types of land use within the floodplain. The predominant land uses are classified as:

- Residential;
- Commercial; and
- Industrial.

Residential, commercial and industrial flood damages include damage to structures (*e.g. buildings, houses, factories, offices*) and damage to the items within those structures. They also include damages to outdoor facilities and associated infrastructure, and to the land on which the structures are sited.

Damage to infrastructure as a result of flooding includes losses associated with damage caused by inundation of roads, water supply and sewerage services, and damage to utilities such as electricity, gas and telecommunications systems.

Residential, commercial and industrial damages can be separated into direct and indirect damages. Direct damages are the result of the physical contact of floodwaters with the structure and may include the costs associated with repair, replacement or the loss in value of inundated items. Indirect damages represent all other costs not associated with physical damage to property and typically include the loss of income incurred by residents affected by flooding, as well as flood recovery items such as clean-up costs.

The approach developed to calculate flood damages for the study area is based upon the development of a representative damage curve for typical structures in the floodplain. A damage curve is a numerical relationship that correlates the depth of flooding to the cost of damages that would result from that flooding. The cost of the damages associated with the flooding increases as the depth of flooding increases.

The approach employs the procedures outlined in the DECC (*now OEH*) Floodplain Risk Management Guideline for 'Residential Flood Damages' (*2007*). It involves the application of the damage curves documented in the literature with flood data that has been updated as part of this study. The flood data is compared to floor level data for structures or properties to determine the quantum of damages.

Based on data collected during a site survey conducted by WorleyParsons, residential properties within the study area were classified as either:

- Single storey set directly on the ground;
- Single storey high set (*i.e., on piers*); or
- Double storey building set directly on the ground.

Commercial properties include shops, pubs, offices and large shopping complexes, while industrial premises include metal fabrication works and distribution warehouses. For commercial properties, a distinction was made between stand-alone commercial buildings, and shopping complexes.

An estimate of the direct damages associated with the inundation of commercial and industrial premises was based on recorded damage costs for similar premises reported in the literature. This literature includes a range of previous floodplain management studies and recorded data presented in intergovernmental reports. OEH has advised that this approach is suitable, provided that the damage curve data is updated to reflect current Average Weekly Earnings (*AWE*) and Goods and Services Tax (*GST*), where applicable.

The OEH guidelines for residential properties incorporate some allowance for indirect damages, such as clean-up costs and loss of rental income.

Indirect damages for commercial and industrial premises were assumed to be 50% of the corresponding direct damages which is based on values used previously in the background literature. This accounts for the significant impact of indirect influences, such as the slowdown that a business could experience due to employees being unable to get to work due to inundation of roads.

There is no specific data available to define the extent of the public and corporate infrastructure that could be damaged as a result of flooding. Additional infrastructure damages were applied to reflect 30% of the total direct and indirect costs for residential, commercial and industrial properties. This is consistent with approaches employed for other areas of NSW.

10.2.2 Stage-Damage Relationships

Stage-damage curves reflect the potential flood damage as a function of the depth of over floor flooding of a building. DECC's Floodplain Risk Management Guideline for 'Residential Flood Damages' (*2007*) outlines the method for determining stage-damage curves for residential dwellings. This procedure is recommended as the basis for derivation of average annual damages and net present values of damages to enable the comparison of flood management options.

Standard stage-damage curves have also been developed from records of damages gathered from interviews with residents and landowners in flood affected communities. For example, Smith et al (*1979*) determined stage-damage relationships for different land use types based on data gathered during and following the Lismore floods in 1974. These curves were adopted for analysis of commercial and industrial damages.

The standard stage-damage curves for commercial and industrial properties were scaled to account for indirect damages. Infrastructure costs have been calculated separately.

The adopted stage-damage curves are included within **Appendix M**.

10.2.3 Average Annual Damage

The relative cost of potential flood damages is typically expressed in terms of the Average Annual Damage (*AAD*). The AAD is the average damage per year that would occur from flooding over an extended period of time.

It should be noted that there may be long periods where no floods occur or the floods that do occur are too small to cause significant damage. Conversely, some floods will be large enough to cause extensive damage. Accordingly, the Average Annual Damage is equivalent to the total damage caused by all floods over an extended period of time divided by the number of years within that period (*DECC, 2007*). It provides a quantitative measure for comparing the relative economic benefits of potential flood damage reduction options..

10.3 FLOOD DAMAGES ANALYSIS FOR THE STUDY AREA

In order to calculate the potential flood damages, it is necessary to have data that defines the floor levels of structures and infrastructure that could potentially be flooded and details of the type of structure (*e.g., residential dwelling or commercial premises*). This data can be used with peak flood levels generated from the flood modelling undertaken as part of this study to determine the depth of "over floor" flooding for each residential and commercial property.

Data defining the minimum floor elevations of residential, commercial and industrial buildings was collected by WorleyParsons using on-site observations of the height of floor levels above the adjacent ground elevation. This information was then superimposed over the available terrain data to estimate the finished floor level of each structure. This floor level data was then used alongside the peak flood levels generated from the hydrodynamic modelling study to determine the depth of over floor flooding at each structure.

Estimates of the tangible flood damages associated with the 50%, 20%, 5%, 1%, 0.5% and 0.2% AEP flood events and the Probable Maximum Flood (*PMF*) event are outlined in **Table 10.1**.

The results indicate that the total damage bill is estimated to be approximately $31,660,000 for the design 1% AEP event. The Average Annual Damage for the study area, incorporating all events up to the PMF, is estimated to be **$4,538,000**.

Table 10.1 TANGIBLE FLOOD DAMAGES

FLOOD EVENT	RESIDENTIAL		COMMERCIAL & INDUSTRIAL		INFRASTRUCTURE DAMAGES	TOTAL
	Number[1]	Damages	Number	Damages		
50% AEP	4 (0)	$43,900	-	-	$13,200	$57,000
20% AEP	23 (7)	$837,700	4	$771,800	$482,900	$2,092,400
5% AEP	179 (126)	$14,493,300	24	$3,152,400	$5,293,700	$22,939,300
1% AEP	**333 (222)**	**$30,631,600**	**43**	**$6,841,900**	**$11,242,100**	**$48,715,600**
0.5% AEP	469 (303)	$41,471,400	55	$9,980,424	$15,435,500	$66,887,400
0.2% AEP	632 (467)	$60,552,008	62	$14,705,808	$22,577,300	$97,835,200
Probable Maximum Flood	1609 (1420)	$172,404,926	97	$26,513,686	$59,675,600	$258,594,200

1 Number of residential dwellings subject to over-floor inundation shown in parentheses

The location of properties that are damaged during the design 1% AEP event are shown in **Figure 10.1** and **Figure 10.2**, including those subject to both below-floor and over-floor inundation.

10.4 INTANGIBLE FLOOD DAMAGES

Intangible flood damages are those that are unable to be quantified in monetary terms. These damages are related to the physical and mental health of individuals, environmental concerns, the ability to undertake necessary evacuation measures and disruption to essential community services and operations.

Emotional stress and mental illness can stem from a number of experiences associated with damage to family homes and businesses. These include:

- Destruction of memorabilia (i.e., family photos);
- Death of pets;
- Financing the replacement of damaged property;
- Living in temporary accommodation;
- Children attending a different school;
- Loss of business income and potential clients;
- Loss of wages; and
- Anxiety experienced by young children.

This type of intangible damage to the wellbeing of residents could be significant in the event of a major flood. Accordingly, it is possible that the intangible damage cost could be as high as or higher than the total tangible damage cost.

11. SENSITIVITY TESTING

11.1 IMPACT OF CLIMATE CHANGE

The NSW government has published a number of documents which provide guidance to account for climate change impacts on flooding. The Department of Environment and Climate Change (*DECC*) Floodplain Risk Management Guideline titled: 'Practical Consideration of Climate Change' (*2007*) is most relevant to the present study.

This guideline provides estimates for the change in "Extreme Rainfall" under future climate change conditions for different parts of NSW. The guideline recommends consideration of increased rainfall intensities of between 10% and 30%. Accordingly, it was agreed with Council that the following climate change scenarios would be tested:

- 1% AEP rainfall event with 10% increase in rainfall intensities; and
- 1% AEP rainfall event with 20% increase in rainfall intensities.

11.1.1 Hydrologic Modelling

In order to derive the inflow hydrographs for the two climate change scenarios, the XP-RAFTS hydrologic model was updated to include intensity-frequency-duration (*IFD*) data with the appropriate percentage increases in intensities applied.

Updated inflow hydrographs for all upstream boundary locations and local subcatchment inflows were derived for both climate change scenarios. A summary of the peak discharges for each upstream model boundary location is provided in **Table 11.1**.

11.1.2 Hydrodynamic Modelling

The updated upstream and local subcatchment inflow hydrographs were then used within two further simulations of the TUFLOW hydrodynamic model. The results from these two scenarios provide a suitable indication of the flooding characteristics that could be expected within the study under the assumed climate change conditions.

11.1.3 Observed Impacts on Local Flooding

The predicted impact on peak flood level resulting from the increased rainfall due to climate change have been extracted from the modelling results and are presented in **Figures N.1** *and* **N.4** for the two climate change sensitivity events. The figures display the total increase at each location in the network (*i.e., the peak flood level under climate change conditions minus the corresponding present day peak flood level*). Individual figures have also been included for two sub-areas within the Camden LGA area (*refer* **Figures N.2**, **N.3**, **N.5** *and* **N.6**).

Table 11.1 PEAK UPSTREAM INFLOWS USED IN TUFLOW MODEL (CLIMATE CHANGE)

LOCATION (*refer* Figure 6.4)	XP-RAFTS MODEL NODE	PEAK DISCHARGE[1] (m^3/s)	
		1% AEP+10%	1% AEP+20%
		36hr	36hr
Nepean River	(MENANGLEIN)	9526	10516
Navigation Creek	NAVI B	122	135
Matahil Creek East	MAT E DMY	174	193
Matahil Creek West	MAT W A	28.2	31.4
Mt Hunter Rivulet	MT H C DMY	381	424
Cobbitty Creek	COBBITTY A	4.2	4.6
Wattle Creek	UNNMD1 DMY	118	131
Eagle Creek	EAGLE	132	146
Forest Hill Creek	FOREST	93.2	104
Bringelly Creek	BRI C1 DMY	48.1	53.5

1. Peak discharges listed do not necessarily occur simultaneously.

The observed results generally suggest that impacts are relatively constant from the upstream end of the model (*at Menangle*) to the confluence of the Nepean River with Cobbitty Creek. Further downstream of this location, the influence of the gorge between Theresa Park and Bents Basin becomes more apparent. The flow constriction created by the gorge inlet causes the area immediately upstream of it to accumulate excess flows during flood events and this appears to be exacerbated by the increased flows assumed during climate change scenarios (*refer* **Figures N.1** *and* **N.4**).

The impacts appear to then lessen slightly downstream of the gorge, before rising slightly once again when a similar situation occurs in the river as it approaches the second gorge that links Wallacia with the Nepean River's confluence with the Warragamba River (*refer* **Figures N.1** *and* **N.4**).

Increasing rainfall intensities by 10% resulted in peak flood level increases of between 0.50 and 0.70 metres in the areas upstream of the Cobbitty Creek confluence (*refer* **Figures N.2** *and* **N.3**). Typical impacts as the river passes the urban centre of Camden are between 0.65 and 0.70 metres.

Impacts on levels in the river downstream of Cobbitty Creek increase markedly and are approximately 1.5 metres upstream of the gorge and within the gorge itself (*refer* **Figure N.1**). Although the impacts decrease to around 1.0 metres downstream of the gorge and Bents Basin, they are observed to rise once again in the vicinity of Wallacia and the gorge just upstream of the confluence with the Warragamba River. The observed impacts of increasing rainfall intensities by 10% are also around 1.5 metres in these areas.

A similar pattern of impacts was observed when increasing the rainfall intensities by 20%. Peak flood level increases of between 1.0 and 1.2 metres are predicted in the floodplain areas upstream of the Cobbitty Creek confluence (*refer* **Figures N.5** *and* **N.6**). Typical impacts as the river passes the urban centre of Camden are between 1.1 and 1.2 metres.

Impacts on levels in the river downstream of Cobbitty Creek again increase markedly and are observed to be approximately 2.5 metres upstream of the gorge and slightly lower within the gorge itself (*refer* **Figure N.4**). Although the impacts decrease to around 1.0 metres downstream of the gorge and Bents Basin, they are observed to rise once again in the vicinity of Wallacia and the gorge just upstream of the confluence with the Warragamba River. The observed impacts of increasing rainfall intensities by 20% are around 2.4 metres in these areas.

11.2 IMPACT OF STRUCTURE BLOCKAGE

11.2.1 Hydrodynamic Modelling

In order to simulate the obstruction of flow through structures within the model network as a result of debris accumulation during large flood events, a number of additional model simulations were undertaken that assumed that all structures within the model were blocked for the duration of the design 1% AEP flood event. These simulations assumed a series of blockage levels up to and including 50%. The blockages were applied to all culverts and bridges represented within the model network (*refer* **Table 6.1**).

11.2.2 Observed Impacts on Flooding

As the extent of flooding during the 1%AEP flood event was typically observed to be well in excess of the width of the culverts and bridges within the model, blocking of these structures was generally seen to have a minimal impact on peak flood levels. The pattern of impacts observed during the highest blockage scenario simulated (*50% blockage of all identified structures*) suggests that the blockage of these structures tends to only result in localised impacts immediately upstream and downstream of the structures.

The predicted impact on peak flood level resulting from the assumed structure blockage have been extracted from the modelling results and are presented in **Figures N.7** *and* **N.8**.

The figures display the total increase at each location for two sub-areas within the Camden LGA (*i.e., the peak flood level under blockage conditions minus the corresponding unblocked peak flood level*).

Impacts are observed to be typically less than 0.2 metres and restricted to localised areas in the immediate vicinity of the blockages themselves. The only notable exception to this pattern can be observed as flows within Matahil Creek (*West*) attempt to pass beneath Burragorang Road approximately 2 kilometres west of Camden (*refer* **Figure N.8**). However, the peak flood levels are only predicted to rise by a maximum level of 0.2 metres and are restricted to an area that extends approximately 350 metres upstream of the blockage. The impacts observed elsewhere within the model network are generally much less.

11.3 IMPACT OF LOWER FLOODPLAIN ROUGHNESS

11.3.1 Hydrodynamic Modelling

In order to assess the sensitivity of the peak flood levels within the study area to changes in floodplain roughness a number of additional simulations were performed using a variety of alternative roughness values. The final adopted model simulations assume a Manning's 'n' roughness of 0.060 in accordance with standard values for areas of pasture with light brush cover. The lowest of the roughness values tested during the sensitivity simulations assumed a reduction of this value to 0.045. The impact on peak flood levels in the study area resulting from this assumption were assessed by undertaking a further simulation of the TUFLOW hydrodynamic model for the 1% AEP flood event under the reduced floodplain roughness condition.

11.3.2 Observed Impacts on Flooding

The predicted impact on peak flood level resulting from the assumed lowering of floodplain roughness have been extracted from the modelling results and are presented in **Figures N.9** and **N.10**. The figures display the change in flood levels at two sub-areas within the Camden LGA (*i.e., the peak flood level under reduced roughness conditions minus the adopted roughness peak flood level*).

As with the assessment of climate change impacts (*refer* **Section 11.1**), the observed impacts appear to be largely affected by the flow regime as the Nepean River approaches and enters the gorge that flows between Theresa Park and Bents Basin.

The results observed across the floodplain from the upstream end of the model (*at Menangle*) to the confluence of the Nepean River with Cobbitty Creek are typical of those that would be assumed from a reduction in floodplain roughness; peak flood levels are observed to reduce by as much as 0.35 metres (*refer* **Figures N.9** *and* **N.10**). Accordingly, flows are likely to be flowing at slightly higher velocities as they approach the hydraulic control imposed by the gorge downstream.

As such, a marked "switch" in the impacts in peak flood level is observed downstream of the Cobbitty Creek confluence. The increased flows approaching the entrance to the gorge at

Theresa Park appear to result in a build-up of floodwaters in this area, causing an increase in peak flood levels of up to 0.20 metres during the reduced roughness scenario. These impacts can be observed to dissipate within the gorge and reductions in peak flood levels are again observed by the time the river reaches Bents Basin further downstream.

It is worth noting that this issue was first observed during the calibration of the hydrodynamic model (*refer* **Section 6.3**). Varying and testing the floodplain roughness was undertaken while obtaining a suitable compromise between flood levels in the upstream and downstream sections of the LGA during the model calibration process. The assumed grass/pasture/brush roughness of n=0.060 was found to provide the best overall fit during the model calibration and is considered to offer a conservative approach for design flood level estimation within the urbanised areas of Camden and Elderslie.

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