CAMDEN COUNCIL

UPPER SOUTH CREEK FLOOD STUDY

FINAL REPORT 2011 REVISION 1

MAY 2012

Level 2, 160 Clarence Street
Sydney, NSW, 2000

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

UPPER SOUTH CREEK FLOOD STUDY

FINAL REPORT 2011 REVISION 1
MAY, 2012

Project UPPER SOUTH CREEK FLOOD STUDY	**Project Number** 29024
Client Camden Council	**Client's Representative** Maria Pinto
Authors Stephen Gray Ivan Varga	**Prepared by** Ivan Varga
Date 18 May 2012	**Verified by**

Revision	Description	Date
6	Final Report 2011 Revision 1	May 2012
5	Final Report Revision 1	February 2012
4	Final	November 2011
3	Final Draft	October 2011
2	Draft Final Revision #2	September 2011
1	Draft Final	February, 2011

UPPER SOUTH CREEK FLOOD STUDY

TABLE OF CONTENTS

PAGE

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS USED IN THIS REPORT

- AEP: Annual Exceedance Probability
- ALS: Airborne Laser Survey
- ARI: Average Recurrence Interval
- ARR: Australian Rainfall and Runoff
- BOM: Bureau of Meteorology
- CDIRS: Computerised Design IFD Rainfall System
- OEH: Office of Environment and Heritage
- DEM: Digital Elevation Model
- DWR: Department of Water Resources
- FMC: Floodplain Management Committee
- FPRM: Floodplain Risk Management
- GIS: Geographical Information System
- IFD: Intensity-Frequency-Duration
- LCC: Liverpool City Council
- LGA: Local Government Area
- UNSW: University of New South Wales
- UTS: University of Technology Sydney

Please note that a glossary is also provided at the rear of this document in Appendix A.

EXECUTIVE SUMMARY

The Upper South Creek Catchment lies within Camden City Council's Local Government Area. Within the defined study area (i.e. from the head waters to as far north as Bringelly Road) the South Creek catchment includes (from east to west) Bonds Creek, Kemps Creek, Rileys Creek, South Creek, Lowes Creek and Thompsons Creek.

The Upper South Creek area is currently undergoing limited urban development with significantly more of the same planned for the future and as such landscapes which are currently mainly rural residential, will, over the course of ensuing years, become relatively urbanised. In order to plan development and ensure that its responsibilities regarding planning and risk management are met, Camden Council has engaged in the State Government's Floodplain Risk Management Planning process. The first step of this process is the carrying out of a Flood Study in order to define design flood behaviour and also to create a model suitable for testing mitigation works in ensuing stages of work.

As part of the Study a hydraulic model has been built. In order to establish the accuracy of the model for describing design flood behaviour it has been calibrated and validated. Results of the model calibration/validation work are presented herein, including a review of relevant previous studies and a discussion on the data collected for constructing and calibrating the hydraulic models used in the study. Note that previously Council and OEH have confirmed the suitability of the modelling system developed, on the basis of calibration/validation work presented herein. Calibration was carried out for the 1988 event which was a large event (between a 30 and 70 year ARI event based on References 1 and Reference 2, respectively) and for which 15 flood marks were available. Two smaller events (~ 10 year ARI) were used for validation purposes (1991 and 1992 events). The match between the model and the observed data for these events indicates that the model is able to emulate observed behaviour.

Design flood modelling has been undertaken for the 5%, 2%, 1%, 0.5% and 0.2% AEP events as well as the PMF. Critical durations identified are the two hour (Kemps and Bonds Creeks) and the nine hour (South Creek). The provisional hazard and hydraulic categories have also been defined and mapped for all of the events.

Features of flooding within the study area are;

- Limited variability between events with respect to flood level and extent; and
- Numerous roads overtopped for all events modelled.

Note that the Flood Study produces results on the basis of ground survey completed in 2008. As such no results are provided within the extents of proposed Oran Park and Turner Road developments as here the ground is in flux.

Generally speaking flood liability in the Upper South Creek catchment is limited to low lying areas and floodplain. Flood levels do not increase markedly for rarer events and flood extent does not vary significantly between smaller more frequent events and the larger rarer events and this characteristic is due to the well defined floodplain. Low slopes in the catchment and wide floodplains also lead to less hazardous flood flows than might otherwise be expected.

1. INTRODUCTION

1.1. Study Area

The study area comprises the upper catchment of South Creek, including tributaries, from its headwaters north and east of the Hawkesbury River to 500 m downstream of Bringelly Road. Creeks included from east to west are Rileys Creek, South Creek and Lowes Creek. Bonds Creek, Kemps Creek, and Thompsons Creek are also part of South Creek catchment, however, their confluence is located downstream of Bringelly Road. The catchment area of Thompsons, Kemps and Bonds Creeks within the study area is limited. Flow interaction between the creek systems (upstream of Bringelly Road) does not occur, with significant ridges dividing the catchments. The combined catchment area is approximately 71 km^2 and the distribution of catchment area between creeks is shown below.

Table 1: Study Area Catchment Areas

	Area (km^2)
Thompsons Creek	1.6
Upper South Creek	43.3
Rileys Creek	16.1
Kemps Creek	5.3
Bonds Creek	4.5

The study locality is shown in Figure 1 and the specific study area in Figure 2. The catchment is characterised by undulating hills with elevations ranging from approximately 130 m at the southern catchment boundary to approximately 40 m in the vicinity of Bringelly Road.

Vegetation types represented in the study area mainly comprise pasture and scattered timber. There is significant clearing on most land lots. In some places, particularly near creek lines, there are thick copses of vegetation, including stands of significant timber, but in the main cleared land with an abundance of pasture is observed. A variety of land uses appear to exist on the large land holdings (relative to typical urban lots). Land uses seem to vary from light industrial such as storage and servicing of heavy haulage equipment to hobby farms with very low density stocking rates to relatively intensive market gardening operations.

There appears to have been an amount of landscape alteration throughout the area in order to create small farm dams or to landscape areas adjoining residences. A number of minor and relatively small levee type features are noted near creek lines. These are not considered likely to significantly impact on flow distribution during large flood events, such as the 1% AEP event, for example. Such features have been represented by the use of break lines in the 2D model discretisation and also have been used in determining the sub-catchment layout.

1.2. Background

Extensive urban development in planned for the Camden (LGA) with two sub-division developments currently under construction (Oran Park and Turner Road). As such fractions of the Upper South Creek study area are about to transition from non-intensive applications to full urbanisation. This creates two priorities:

1. Council requires appropriate information on location and extent of flood risk for planning purposes; and
2. Council needs to define the status quo (or 'baseline') in regard to flooding and need a modelling tool that can be used in the future to model the impact of development and assess the efficacy of proposed mitigation works associated with planned sub-division developments.

1.3. The Goal of the Study

The goal of the study is to define design flood behaviour within the study area. This is to be achieved by establishing the flood level, extent and depth. Additionally, provisional hazard and hydraulic categories will be calculated for the 5%, 2%, 1%, 0.5% and 0.2% AEP as well as the PMF events.

The study also provides hydraulic modelling tools that can be used in the next stage of the Floodplain Risk Management Program (FRMP) process as well as, in the future, to assess individual developments within the wider context of the study area.

2. AVAILABLE DATA

A variety of data has been collected in order to facilitate the hydraulic model build process and that data is detailed here. Previous reports have been collected and summarised with a particular emphasis on calibration data and parameter settings used.

2.1. Previous Reports

A number of previous reports pertinent to the study at hand have been reviewed for relevant content. These reports are:

- Reference 1 (South Creek Flood Study, DWR, 1990);
- Reference 2 (South Creek Floodplain Risk Management Study, DWR, Volumes 1 and 2, 1991);
- Reference 3 (Austral Floodplain Risk Management Study and Plan, LCC, 2003); and
- Reference 4 (South Creek Floodplain Risk Management Study and Plan, LCC, 2004).

Various items of useful data have been taken from these reports with the most significant source of information being Reference 1. The following section will briefly summarise each of these reports and detail information salient to the Study. Some particular items of information were relatively important, such as surveyed flood marks, and pages detailing such information have been scanned and included whole in Appendix B. Some report elements particularly references 2, 3 and 4 will be more pertinent at further stages of the floodplain risk management process.

2.1.1. Reference 1 – South Creek Flood Study, DWR, 1990

This study examined flooding in South Creek downstream to Richmond Road encompassing a catchment area of 414 km^2. As part of the study, data was collected and RAFTS and Mike11 models were built. Extensive survey was carried out in order to provide cross-sections for hydraulic modelling. The report states that the 1986 and 1988 flood events are the largest that have occurred since white settlement, with the 1986 being more concentrated toward the east of the catchment (Ropes Creek is specifically mentioned) whilst the April 1988 event is the largest in the entire catchment (that did not involve backwatering from the Hawkesbury River).

Data utilised in the study is as follows:

- 1:10,000 and 1:4,000 maps for defining catchments;
- Structures and cross-sections (480) were surveyed;
- Rainfall data was obtained from the BOM, UNSW, Sydney Water Board, DWR, Penrith City Council and UTS. The report states that a large quantity of such rainfall data was available for both the 1986 and 1988 events with 18 daily read stations and 22 pluviometers. It is noted that several of the pluviometers overflowed during both events;
- Data was obtained for four stream gauges that lie within the catchment (wider South Creek catchment). The four stations are Elizabeth Drive, the Great Western Highway, Richmond Road and also one on Ropes Creek at Debrincat Avenue. A lot of work went into updating the stage/discharge curves for the gauges during the course of the study for all gauges bar Richmond Road which was found to be so affected by backwater as to

be unusable (for deriving discharge hydrographs);

- Flood marks were obtained from flood peak recorders, debris lines and also by interviewing residents. Most of these marks were obtained at road crossings. An attempt was made to get peak levels at other locations, specifically on smaller tributaries, however efforts did not yield any marks. 46 flood marks were found in all;
- The study used the 1986 event for calibration of Ropes Creek whilst the 1988 event was used for calibration of South Creek;
- During calibration to the 1988 event losses were established as being 35 mm (initial). A near zero continuing loss was used and the comment is made that given the clay soils in the area (as well as the antecedent rainfall which was substantial) this was appropriate;
- The calibration of the 1988 event was a good fit and used a Bx value of 1.3. Note calibration work was done in RAFTS. No discussion is made as to how rainfall was distributed throughout the model or how gauges which overflowed may have been adjusted for calibration purposes;
- Design flood modelling was carried out using IFD information from the 1977 version of ARR;
- Suggested losses from ARR 1977 for areas east of the western slopes are an initial loss of 10-35 mm. After considering various factors an initial loss of 10 mm was adopted. The calculated continuing loss is 1 mm/hour;
- Sensitivity runs carried out indicated that the critical durations for the study were 9 and 40 hours;
- As part of the study Mike11 was used to model almost all of South Creek and the lower sections of Badgerys and Kemps Creek. HEC2 was used to model other creeks within the catchment. Mike11 was calibrated, again to the 1986 and 1988 events. RAFTS parameters were not changed and it was found that a good fit could be obtained by simply altering ‘n’;
- The 1990 report found 1% AEP flood levels that were generally in the order of 1 m higher than those predicted by the earlier 1985 study; and
- Predicted 1% AEP flood level upstream and downstream of Bringelly Road are 59.3 mAHD and 58.3 mAHD respectively.

2.1.2. Reference 2 - South Creek Floodplain Risk Management Study, DWR, Volumes 1 and 2, 1991

This study sought to address some of the flooding issues highlighted by both previous flooding events (in 1986 and 1988 significant flood events occurred) and the 1990 study (Reference 1).

Reference 2that led to the construction of a levee at Masterfield Street immediately upstream of Bringelly Road, amongst other flood mitigation works. The study defines flood risk via the categorisation of flood liable land into low and high hazard zones as well as providing hydraulic categories for flood affected land. Overall the study is of little value to the current Study, although it does highlight some areas within the Upper South Creek area which experience problems with flooding and this information will be very useful moving into the Management Study stage of the overall process. The problem areas identified are as follows:

- Masterfield Street;
- Overett Avenue on Kemps Creek; and
- Victor Avenue on Kemps Creek.

As an extension to Reference 1 the study did assess floods other than the 1% AEP, including, the 2%, 5% and the PMF. The 2% AEP event peak flow at Bringelly Road is cited as being 260 m^3/s with the 5% AEP event having a flow of 202 m^3/s. This implies that the 1988 event, with a peak flow (via interpolation) of 210 m^3/s was slightly larger than a 5% AEP event and significantly less than a 2% AEP event. This conclusion is in contrast to the finding from Reference 1 that the 1988 event was approximately a 75Y ARI event.

2.1.3. Reference 3 - Austral Floodplain Risk Management Study and Plan, LCC, 2003

This study focuses on the Austral site which lies downstream of Bringelly Road within Kemps Creek catchment. Given that the current study is focussed on Kemps Creek as far downstream as Bringelly Road there is no overlap between the domains of interest. Nevertheless the study is for an area adjacent to the study area and as such the modelling approach and parameters used are of interest from a regional similarity perspective.

The study notes the following with regards to hydrological/hydraulic parameters:

- Initial losses used varied depending on the frequency of the event. For the 1% event an initial loss of 34 mm was used (mirroring losses adopted in Ref 1). For the 5% event an initial loss of 45 mm was used;
- Continuing loss of 1.0 mm/hour was used for all modelled events;
- A Bx value of 1.3 was used for all RAFTS modelling based on Ref 1;
- An initial loss of 0 and a continuing loss of 1 mm/hour was used in PMF modelling;
- A general observation made regarding flooding is that the in-bank capacity of creeks is ~ 1Y ARI, the flood depth difference between the small and large events is small (1Y ARI to 100Y ARI), in the order of 0.5 m. This flood behaviour results from the fact that the floodplain is extensive relative to upstream catchment size; and
- It is also observed that culverts relieving roads have a capacity in the order of the 2Y ARI flood and that general flow velocities (except immediately downstream of structures) are low, at 1-1.5 m/s.

2.1.4. Reference 4 - South Creek Floodplain Risk Management Study and Plan, LCC, 2004

This study addresses that portion of the South Creek catchment lying within the LCC area and as such is not particularly relevant to the Study in hand. As such no review of this document has been carried out.

2.2. Airborne Laser Scanning Survey

Airborne Laser Scanning (ALS) survey was carried out in the year 2008. Council was able to supply WMAwater with multiple tiles of ground strike points. Each tile has then been surfaced and converted into raster format. The raster tiles were then merged in order to create one metre resolution DEM covering the entire study area. The DEM is shown in Figure.

The ALS derived raster is based on data which has as an accuracy of +/- 0.15 m (for 1^{st} standard deviation i.e. 68% of the data). The density of ground strikes is approximately one per square metre. The ALS data forms the basis of most of the topographic data that will be used in the modelling.

2.3. GIS Layers

Council have supplied various data layers used in model build work and also to provide background in report figures. Specifically the following layers have been received:

- Aerials;
- ALS;
- Land Use data;
- Lot boundaries;
- Easements;
- Road centrelines;
- Cadastre;
- Road names; and
- Water courses.

In particular Council have supplied GIS layers which describe land use to a high degree of detail. A plot showing the data supplied is shown in Figure 4. Layers differentiated are:

- Trees;
- Creeks and dams;
- Golf courses and recreation areas;
- Quarries;
- Residential areas;
- Cultivation areas; and
- Roads.

2.4. Calibration/Validation Data

2.4.1. Calibration/Validation Events Identified

Historical flood events used for calibration will ideally be relatively large. Ideally events would be as large as the flood planning event, as this would provide some confidence in the models definition of flood planning areas and other inputs into the development planning and approval process. In lieu of a 1% AEP event (or larger) other large events are sought. A review of the records has identified three events for use in calibrating and validating the model. These are the:

- April 1988 event which was according to Reference 1 was near a 0.013% AEP event (although Reference 2 analysis would suggest an AEP of ~ 0.033 is more likely);
- 1991 event (around a 10% AEP event for a 6 hour critical duration); and
- 1992 event (around a 10% AEP event for a 6 hour critical duration).

Further important criteria for the inclusion of calibration/validation events is the availability of rainfall data and flood marks or surveyed peak water levels.

2.4.2. Rainfall Data

A variety of sources were used in order to obtain gauged rainfall for the historic events to be modelled. The locations of rainfall stations used are shown in Figure 5. Note that in some cases these stations are no longer in service.

2.4.2.1. April 1988 Event

Raw gauged data for the 1988 event could not be sourced, despite data requests and contact being made with OEH, Sydney Water Corporation, UNSW and the BOM. Gauged data had however been entered into the Reference 1 RAFTS model and so this data was used instead. In order to quantify the design probability of the rainfall it is presented in Figure 6 against IFD data for the locality. As can be seen, for durations ranging from 500 minutes (approximately nine hours) to a one day duration (1440 minutes) the rainfall measured at Narellan (purple) is a good approximation of the 2% AEP event. For longer durations (36 hours and 60 hours) the gauged rainfall at Narellan is clearly touching the 1% AEP line. The critical time of concentration however is within 4-8 hours in all likelihood. The Anderson Equation from Reference 5 calculates 4 hours for the time of concentration. An alternative method which is less empirical, the kinematic approach, typically will give double the Anderson Equation estimate (i.e. 8 hours) given the low slopes in the study area. As such the rainfall plots up as, at maximum, a 2% AEP rainfall. Note however that as per Reference 1 conditions were wet prior to the main burst plotted in Figure 6 and so although a 2% AEP rainfall (or less) the event may have been transformed into a slightly larger (and rarer) event. In the context of assessing the probability of the 1988 event it is worth noting that Reference 1 did indicate that some pluviometers overflowed during the event and so in some cases the actual measured rainfall is likely to be an underestimate.

Rainfall depths from various stations utilised are shown below in Table 2. Note that total rainfall depths indicated occurred over a period of 62 hours.

Table 2: 1988 Rainfall Depths at Various Stations[1]

Station (Average Intensity)	Rainfall depth (mm)
Bangor (4.87mm/h)	301.9
Bangor/ Bringelly (4.68mm/h)	290.1
Bangor (4.35mm/h)	269.6
Bangor (4.55mm/h)	282.0
Bangor (4.58mm/h)	283.9
Bangor (4.74mm/h)	293.8
Bringelly (4.19mm/h)	259.7
Bringelly (4.48mm/h)	277.7
Bringelly (4.66mm/h)	288.8
Narellan/Bangor (4.4mm/h)	275.5
Narellan/Bringelly (4.94mm/h)	306.3
Narellan (4.02mm/h)	249.1
Narellan (4.16mm/h)	257.8
Narellan (4.42mm/h)	273.9
Narellan (4.63mm/h)	286.9

It's noteworthy that the mean rainfall from the above is 280 mm and that the standard deviation is 16 mm, or 6% of the mean rainfall depth. These values indicate a spatially consistent rainfall event which in turn leads to an expectation that available rainfall data should be more than adequate to describe the event for calibration purposes.

2.4.2.2. Validation Events – 1991 and 1992

Historic rainfalls for the 1991 and 1992 events were available for the BOM stations as listed in Table 3 over the page.

Figure 8 shows the 1991 event gauged rainfall for the four stations at which data was obtained versus IFD values. Camden Golf Course shows the largest rainfall and indicates that it is close to a 1% AEP event for the 36 hour duration. For the 4-8 hour range however the event is a 50% - 10% AEP).

Figure 9 shows that for the 1992 event Warragamba and Camden at Brownlow Hill the rainfall is similar to the 1991 event. For longer durations (24 hours) the rainfall reaches the 1% AEP however for the more relevant smaller durations (4 – 8 hour) the event is between a 50% and 10% AEP event.

[1]Note that station names are taken directly from the Reference 1 RAFTS model and indicate different weighted rainfall sets used to apply to different sub-catchments within the RAFTS model.

Table 3: 1991 and 1992 Rainfall Event Depths at Various Stations

Station (Average Intensity)	1991 Rainfall depth (mm)	1992 Rainfall depth (mm)
568038 - Bangor	111.5	63.5
568045 - Warragamba	162.5	271.5
568149 - Camden (Brownlow Hill)	221.5	226.0
568156 - Camden Golf Course	264	197.5

In summary then both the 1991 and 1992 events were, for the Upper South Creek catchment, minor flood events and in the order of 10% AEP events.

2.4.2.3. Design Rainfall Data

Design rainfalls were calculated using the BOM's IFD tool for Bringelly (33.95ºS, 150.75ºE). Corresponding IFD values are presented below.

Table 4: Design Rainfall Depths for various ARI and Durations - Bringelly at Upper South Creek

	1 Year	2 years	5 years	10 years	20 years	50 years	100 years
5Mins	76	98.2	127	145	167	196	219
6Mins	71.1	91.9	119	135	156	184	205
10Mins	58.1	75	97.3	110	127	150	167
20Mins	42.4	54.7	70.9	80.3	92.7	109	121
30Mins	34.4	44.4	57.5	65.2	75.2	88.3	98.4
1Hr	23.2	30	38.8	43.9	50.7	59.6	66.3
2Hrs	15.1	19.5	25.2	28.5	32.9	38.6	43
3Hrs	11.6	15	19.4	21.9	25.3	29.7	33
6Hrs	7.42	9.56	12.3	14	16.1	18.8	21
12Hrs	4.76	6.14	7.93	8.97	10.3	12.1	13.5
24Hrs	3.08	3.97	5.16	5.85	6.76	7.95	8.86
48Hrs	1.95	2.53	3.31	3.77	4.38	5.18	5.78
72Hrs	1.45	1.88	2.47	2.83	3.28	3.89	4.35

Design event rainfall was uniformly distributed across sub-catchments without the application of an Areal Reduction Factor (ARF). An ARF was not used since the intent from Council's perspective is to use the modelling system for the assessment of areas internal to the study area as well at the downstream end. As such use of the total catchment area to derive an ARF would not be appropriate. It is noteworthy however that given the overall lack of sensitivity in the system that the main impact of using an ARF would be on downstream locations (e.g. upstream of Bringelly Road).

2.4.3. Stream Gauge Data

Stream gauge data (level) has been sourced for Station 212320 at Elizabeth Drive (upstream side of road) for each of the three modelled events. This data has been supplied by OEH. The entire period of record was obtained and reviewed in order to identify the 1991 and 1992 events as suitable calibration events. The gauge is used as a source of stage rather than discharge for the following reasons:

- lack of gauge ratings carried out for high flows[2]; and
- the gauge can be backwatered by retardation of flow at Elizabeth Drive (a mechanism likely to be exacerbated by blockage during flood events).

2.4.4. Flood Marks

Seven surveyed flood marks were taken from Reference 1. The flood marks are located at road crossings adjacent to South Creek and upstream of Bringelly Road. The location of flood marks is shown in subsequent figures demonstrating the calibration.

Council were also able to supply eight surveyed flood marks at Masterfield Street for the 1988 event pre the construction of the Masterfield Street levee. It is noteworthy that these points indicate that significant backwatering led to a level pool upstream of Bringelly Road during the 1988 event.

[2]Largest flow for which rating work has been carried out appears to be a flow of ~ 65 m^3/s. The 1988 event resulted in a peak flow of approximately 215 m^3/s.

3. MODELLING

3.1. Introduction

The study's modelling system has been designed to achieve the following objectives:

- Emulate observed flood behaviour;
- Provide a baseline hydraulic modelling system (current catchment conditions);
- Define design flood behaviour under current and proposed catchment conditions; and
- Assess mitigation works under the Floodplain Risk Management Study.

To achieve this, a hydraulic model (1D/2D) has been built. Further details on that build are provided herein.

3.2. Hydrological Modelling

3.2.1. Introduction

Previous versions of the Flood Study have featured a separate hydrological model, XP-RAFTS (RAFTS). This model was built and calibrated for the current study and then utilised in all design runs. Mapping resolution criteria for the Study did, however, necessitate a change in approach.

A Direct Rainfall Method (DRM) was trialled and found to achieve mapping resolution criteria whilst also closely matching previously published results. As such the DRM has been utilised. Section 3.2.3 describes the work carried out to verify the validity of the DRM prior to calibration.

3.2.2. Rainfall on Grid Considerations

DRM is a relatively new approach to hydraulic modelling and as such a discussion of the method and its advantages and disadvantages are presented herein.

For the past few years, many consulting studies carried out for private and government clients, both in Australia and overseas, have been conducted using the DRM. Literature on hydrologic/hydraulic modelling details research outcomes which demonstrate the ability to replicate flood behaviour developed by lumped conceptual hydrological models, and more importantly to match observed data (see Reference 10 and Reference 11).

The main advantages of this approach are:

- Sub-catchments do not require delineation;
- Flows do not need to be artificially applied to certain locations (distributed) as they would need to be given in an approach that utilised separate hydrological/hydraulic models;
- Parameterisation of storage/routing process not required. Routing is based on a relatively high resolution topography and the full St Venant equations;
- No double routing of flows such as in a joint modelling system, and;
- The approach lends itself to the final product which is, of course, mapped flood levels to

inform planning decisions.

Whilst DRM can be used to great advantage it is a relatively new method. As such prior to follow calibration/validation it is best to corroborate the flows derived against more established alternative methods. In complement to such work the methodology utilised to verify the direct rainfall method against the use of a hydrological/hydraulic joint model is described below.

3.2.3. Pre- Calibration Check of Rainfall on Grid Methodology

In order to provide a check on the DRM utilised in the Study, the April 1988 event and the 1% AEP design event peak flood levels and extents were compared to an existing RAFTS model of Upper South Creek.

3.2.3.1. RAFTS Sub-catchment Discretisation

The overall study area of 71 km^2 has been described by 128 separate sub-catchments. Mean sub-catchment size is 0.55 km^2 or 55 ha. Sub-catchments have been discretised with mean catchment size as well as outflow location in mind. For example a sub-catchment will be drawn to terminate upstream of a road as this is likely to be a flow control. A summary of sub-catchment characteristics for each of the 128 sub-catchments is presented in Appendix D.

3.2.3.2. Parameter Values

Values entered into RAFTS include a mixture of sub-catchment descriptive values as well as both sub-catchment specific and global model parameters. Descriptive values include:

- Area
- Fraction impervious/pervious;
- Slope; and
- Roughness (although roughness can also be considered a manipulable parameter within given ranges).

B_x is a global parameter (although can be specified per sub-catchment also) and it relates to catchment storage. Default values of B_x have been derived via empirical study of other catchments in NSW however B_x may also be calibrated. The Reference 1 study found a value of 1.3 for B_x.

Other parameters utilised in the RAFTS modelling include the following:

- Roughness; and
- Losses (various models available but initial/continuing losses are applied herein).

Given that little has changed in the study area since the 1990 study (Reference 1), it was utilised to inform initial (i.e. prior to calibration) parameter settings. Roughness, fraction imperviousness and B_x all came from the Reference 1 RAFTS model. Also the distribution of rainfall station data to specific sub-catchments for the 1988 event came from the Reference 1 RAFTS model.

Other values such as slope and area were estimated using a raster analysis package within a GIS environment. A global roughness value for pervious areas was used, value Manning's 'n' of 0.04.

Hydrographs derived by the hydrological model were applied in the hydraulic model as distributed flow along creeks.

3.2.4. Results Comparing DRM to RAFTS

Table 5: Peak flood levels comparison - 1% AEP design event.

1% AEP Event - Peak Height (mAHD)		
	RAFTS	**DRM**
South Ck - Masterfield St	59.9	60.0
South Ck - Robens Cr	63.8	63.8
South Ck - Catherine Fields Rd	67.3	67.4
Kemps Ck -D/S Bringelly Rd	74.1	74.2
Bonds Ck -D/S Bringelly Rd	73.1	73.1
Bonds Ck -U/S Ingleburn Rd	81.6	81.6

Table 6: Peak flood levels comparison - April 1988 historical event

	Peak Flood Level (mAHD)		
Location	**Observed**	**Apr. 1988 RAFTS**	**Apr. 1988 DRM**
Springfield Rd	77.2	76.9	76.9
Catherine Fields Rd	67.2	67.2	67.2
Barry Ave	66.4	66.1	66.2
Masterfield St	59.4	59.3	59.5
Bringelly Rd - D/S	57.6	58.2	58.2
Robens Cr	63.6	63.7	63.7
Masterfield Lot 1	59.4	59.3	59.5
Masterfield Lot 2	59.4	59.3	59.5
Masterfield Lot 4	59.4	59.3	59.5
Masterfield Lot 5	59.4	59.3	59.5
Masterfield Lot 6	59.4	59.3	59.5
Masterfield Lot 7	59.4	59.3	59.5
Masterfield Lot 17	59.4	59.3	59.5

Results shown on Table 5 and Table 6 demonstrate that resulting peak flood levels utilising the DRM have a reasonable good match against results where hydraulic input was developed in RAFTS. Additionally, peak flood extents were compared between models and results showed a good match. It is noteworthy that some smaller tributaries were mapped as "wet" in the DRM, where no flood water appears in the previously published results as distributed flows were not applied in these smaller tributaries in some cases.

The results demonstrate that the DRM produces comparable results to the previous modelling system and as such the method was adopted for the current Flood Study.

3.3. Hydraulics

3.3.1. Modelling Domain

Two models have been built in order to satisfy the Study's requirements. One covers the Kemps and Bonds Creek catchments, whilst the other covers the South and Thompsons Creek catchments. Refer to Figure 11 for model extents and general build details.

3.3.2. Model Grid Size

Grid size used in modelling is 10 m with DEM sampling at 5 m centres (due to TUFLOW's unique centre and cell edge use of DEM heights). This grid size allows for detailed resolution of important hydraulic features such as reservoirs, roads and levees.

3.3.3. River reach and cross sections

Cross-sections from the 1D MIKE11 model (Reference 1) were used to extend the South Creek TUFLOW model from Bringelly Road (i.e. the downstream extent of the hydraulic study area) to Elizabeth Drive so that gauging station 212320 could be used for model calibration. The extension also ensures that there is some distance between the area of interest (upstream of Bringelly Road) and the downstream boundary in the hydraulic model.

3.3.4. Structures

Twenty (20) structures, all at road crossings, were identified during a site visit and the details of these were subsequently obtained by survey. Locations of these structures are shown in Figure 12.

3.3.5. Roughness Values

Roughness values were utilised in hydraulic modelling. Initial values are based on previous experience in similar catchments and are also informed by reference material such as Chow (1959).

Land use mapping was used to create a variable roughness map for application to the hydraulic model. Values used are as shown in Table 7 over the page whilst the spatial distribution of land uses is shown in Figure 4.

Table 7: Roughness values used in modelling for specific land uses

Land use	Manning's "n"
General Floodplain vegetation	0.040
Creeks (in-bank)	0.030
General Residential	0.060
Environmental Management	0.030
Primary production Small Lots	0.050
Creeks (densely vegetated)	0.045
Public Recreation	0.040
Large Lot Residential	0.045
Infrastructure	0.020
Bridges	0.065

3.3.6. Boundary Conditions

The hydraulic model's downstream extent is situated approximately 200 m downstream of Elizabeth Road (see Figure 11). The model boundary is located further downstream of the 1D extension in order to remove boundary effects from results and the following assumptions/settings have been made:

- A slope of 1/1000 is used in establishing the channel that leads to the downstream boundary;
- The channel is 10,000 metres long (simply for the purposes of removing the contrived boundary from areas where actual results were to be taken); and
- A constant water level is used at the boundary, as opposed to a QH boundary. This is done to improve model stability. The water level used is the equivalent to the top of the bank. Given the boundaries location 10,000 metres downstream of Elizabeth Drive the downstream boundary does not impact on model results.

3.3.7. Blockage

Floods in North Wollongong in August 1998 and in Newcastle in June 2007 have emphasized the importance of blockage at hydraulic structures during flood events. Section 3.4.6 of Council's Engineering Specifications establishes the adoption of 50% as blockage criteria for the 1% AEP event. In the current study 50% blockage has been presumed at all structures. The 50% figure is based on standard practice in design flood estimation and is supported by the 50% blockage that was used to achieve suitable results in the 1988 calibration event. Note that both OEH and Council have endorsed the use of 50% blockage in formulating design flood results for the current study.

3.3.8. Design Losses

Design losses utilised in the Study are based on specific research for South Creek presented in Reference 5. As is standard practice pervious and impervious surfaces have been differentiated with respect to losses.

Pervious Surface

- Initial Loss – 15 mm
- Continuing Loss – 1.5 mm/h

Impervious Surface

- Initial Loss – 1 mm
- Continuing Loss – 0 mm/h

3.3.9. Inclusion of Dams

A feature of the landscape within the study area is a number of dams. Sizes of dams vary from the very small rural/residential lot farm dam to the larger dams providing water storage for various enterprises including agricultural, aquaculture and quarry operations. Dams have been incorporated into the modelling work in the following way:

- Dam crests have been digitised as break lines. This means that the highest resolution topographical data (1m ALS) is used to impose the dam crest height into the 2D hydraulic model. As such outflow from dams will be regulated by the accurately defined crest height; and
- Dam storage has been defined using ALS data. The implication of this is that it's appropriately presumed that the dams are not empty at the model start. Due to the nature of ALS data no storage below the waterline present on the date of survey will be incorporated into the hydraulic modelling.

3.3.10. Loss of Catchment Storage Sensitivity Test

The impact of urban development on catchment storage has been modelled for the 5%, 1% AEP and PMF design events. The extent of the development and associated changes in land use were defined using mapping provided by Council. In accordance with land use change, initial and continuing loss values were adjusted to accurately represent change in pervious/impervious areas within the precincts.

4. MODEL CALIBRATION AND VALIDATION

4.1. Introduction

The purpose of model calibration is to provide confidence that:

- The model is able to replicate historic observed flood behaviour; and that
- The model is likely to convert a design rainfall of a specific AEP into a flood event of an equivalent, or near equivalent, AEP.

The typical process in calibration/validation is as follows:

- a rare event for which a relatively large amount of calibration data exists is chosen to be the calibration event. Other preferable characteristics of the calibration event are as follows:
    - Preferably the calibration event will be of a similar magnitude to the flood to be used for design flood planning, i.e. the 1% AEP event;
    - it is an event which has occurred relatively recently and so catchment conditions at the time of the event are relatively similar to those experienced today; and
    - a wealth of rainfall data, at suitably high resolution, is available to accurately describe input to hydrological/hydraulic models for the event.
- During the calibration of the model, hydraulic model parameters may be altered so as to improve the fit of the model results to observed results. Parameters for manipulation include roughness, blockage, energy and rainfall losses;
- Following achievement of a reasonable calibration the modeller will then carry out validation. During the validation process the calibrated model is applied "blind" to one or more observed events (other than the calibration event) and the performance of the model is observed. Note that the key point which distinguishes calibration and validation is that during validation the modeller may not manipulate the model in order to enhance the fit of model results to observed[3].
- Following the validation work it is typical that calibration/validation results will be presented to the Floodplain Management Committee (FMC) for endorsement. If endorsed the next step in the Flood Study process will typically be the carrying out of design runs.

[3] Although changes may be made to applied losses during a validation run and also depending on the circumstances downstream boundary conditions for the hydraulic model may be altered during the validation process.

4.2. Calibration – 1988 event

Calibration has been carried out against the 1988 flood event. The event was selected for calibration because it is a large flood event (approximately 0.013 % AEP), because it is the event for which the most data exists (gauged water level at the Elizabeth Drive gauge as well as 15 peak flood levels), because catchment conditions during the event are reasonably similar to catchment conditions today and because extensive rainfall data existed for representation of the event.

Another event considered for calibration was the 1986 event. The 1986 event like the 1988 event was a large event. Problematically however no flood marks existed for the event and also less rainfall stations were available in 1986 to spatially resolve the rainfall event. Without calibration marks the 1986 event could not be used, particularly given the calibration marks available for the 1988 event.

4.2.1. Calibration Data

4.2.1.1. Flood Marks

A total of fifteen surveyed flood marks are available for the 1988 event. The locations of these are described in Figure 13. All surveyed flood marks are assumed to be peak water levels and have been used in the calibration process.

4.2.1.2. Gauged Data

The 1988 event was gauged at the 212320, the Elizabeth Street gauge[4], peak gauged height was 43.31 mAHD. As noted in Section 2 the gauge is used for stage data only and not discharge. This is appropriate for the following reasons:

- the rating of the gauge is poorly defined for higher levels. That is observations of how discharge varies with stage have not tended to include larger events which is understandable given:
    - the relatively remote location of the gauge;
    - the relatively small amount of warning time between heavy rainfalls and peak flows at the gauge;
    - the difficulty in accessing the site when a large flood event is occurring due to road closures; and
    - the infrequent nature of such large events. For example the last “large” (i.e. greater than 10% AEP) event at the gauge was in 1988, 23 years ago;
- the stage/discharge relationship for the gauge is likely to vary from event to event as blockage conditions at Elizabeth Drive bridge change; and
- stage is always going to be more accurate than discharge at a stream gauge, particularly for large events. Given its reliability use of stage data is preferred where a hydraulic

[4] For the 1988 event there were 2 “Elizabeth Street” gauges. One that was 200 m upstream of Elizabeth Drive (data available from 1987 to present) and another situated 100 m downstream of Elizabeth Drive (gaugings taken from 1987 to 1992 and data not yet digitised). The gauge upstream of Elizabeth Drive has been used herein.

model is applied.

4.2.1.3. Rainfall data

As stated earlier (see Section 2.4.2.1) rainfall data was not able to be sourced for the 1988 event despite extensive efforts. Instead the rainfall used in the 1990 study RAFTS model was used (Reference 1).

4.2.2. Specific Model Setup issues for the Calibration Event

The model was altered so that the Masterfield Street levee, which was constructed following the flood event of 1988, was not included in modelling of the 1988 event.

Additionally in order to make use of the gauged data from the Elizabeth Street gauge it was necessary to extend the model well downstream of the area of interest (bounded to the north by Bringelly Road). This was achieved by using Mike11 cross-sections from the Reference 1 study as discussed previously in Section 4.

4.2.3. The calibration process

The aim of the calibration process is to achieve a match, via manipulation of model parameters, as well as in some cases model build, to observed data points. Throughout the calibration process the following was adjusted in order to optimise fit:

- Roughness;
- Rainfall Losses; and
- Blockage at both Elizabeth Drive and also at Bringelly Road.

Whilst results more than 100 m upstream of Bringelly Road were not impacted by blockage (and so losses and roughness were used in order to match these values) the flood marks at Masterfield Street were optimised by altering blockage at the Bringelly Road structure. Flood level results in the vicinity of Masterfield Street were sensitive to blockage values utilised.

4.2.3.1. Results

Table 8 and Figure 13 through to 15 show the match between the model and the observed flood behaviour/levels. The peak flood level set is well matched by the model (mean absolute error of 0.15 m) and this result indicates that the model is doing a good job of matching observed flood behaviour over a widely spatially distributed set of points.

With regard to the surveyed peak flood levels it is the case that locations have been supplied for the Masterfield Street set but not for the seven points which describe peak flood level at road crossing locations (Reference 1). As such there is some uncertainty with regard to the exact location of these points. All points have been rounded to the nearest decimal place as the nearest 100 mm is indicative of model accuracy. It's noteworthy that the Masterfield peak flood levels are all matched within 0.1 m. The model results in this area were highly sensitive to blockage at the Bringelly Road structure. The match could have been made exact by manipulating blockage, however given that an even blockage of 50% achieved a reasonable match, further manipulation seemed trivial, particularly given that actual blockage during the event was unobserved.

Roughness values used to achieve the result are as shown in Table 7 whilst loss values are initial 35 mm and continuing of 0 mm/h. Zero continuing loss is an extreme value however given the following the use of zero continuing loss in this instance seems reasonable:

- the clay content of soils in the area;
- the large rainfalls associated with the event, particularly the pre-wetting;
- the use of zero continuing loss in Reference 1; and
- the mention again in Reference 1 that some of the pluviometers overflowed and hence presumably underestimated total rainfall.

Table 8: Calibration Results - Peak Water Level at Various Locations - Comparison of Modelled and Observed

Location	Observed Flood Level (mAHD)	Model Flood Level (mAHD)
Springfield Rd.	77.2	76.9
Catherine Fields Rd.	67.2	67.2
Barry Ave.	66.4	66.2
Masterfield St.	59.4	59.5
D/S Bringelly Rd.	57.6	58.2
Robens Cr.	63.6	63.7
Masterfield St. Lot 1	59.4	59.5
Masterfield St. Lot 2	59.4	59.5
Masterfield St. Lot 4	59.4	59.5
Masterfield St. Lot 5	59.4	59.5
Masterfield St. Lot 6	59.4	59.5
Masterfield St. Lot 7	59.4	59.5
Masterfield St. Lot 17	59.4	59.5

Figure 15 shows the match between the modelled and observed stage hydrographs at Elizabeth Drive gauge. As can be seen from the plot the match between the model and the observed hydrograph is reasonable overall and good for peak water level match. The model hydrograph rises late relative to the gauged (9 hours) but then although consistently early, in the first part of the plot (from a stage height of 42 mAHD), tracks the observed data well.

From 9 pm on the 29th the model result begins to track the observed well and generally the shape is from then on a good match. The modelled peak water level occurs at 9 pm on the 30th whilst the observed data set indicates that the peak stage was achieved at 6 pm on the same day. The modelled peak water level is approximately 43.43 mAHD whilst the peak observed height is approximately 43.35 mAHD. The difference is 0.08 m and this constitutes a good match between observed and modelled behaviour.

The volume match between the observed data and the model is good with a discrepancy of 15%. The value of the comparison is undermined slightly by the likely inaccuracy of the measured volume being based on the faulty stage-discharge relationship (with accuracy also compromised by the Bringelly Road control as well as variable blockage).

4.2.4. Validation

4.2.4.1. 1991 Event

Observed data to verify the validation of the model exists at one location only and that is the Elizabeth Drive gauge. Figure 16 demonstrates the fit. The match is reasonable good and achieves a adequate hit respect to the flood peak. The timing of the rising limb is good, with only a small shift forward in time, which may be caused by rainfall data rather than the model. The model indicates that there is a small peak prior to the main peak whilst the observed data does not reflect this. The model makes a good match of peak level and the falling limb of the main peak is well matched by the model. Only the main peak of the event was modelled in order to reduce the model run time duration and as such the second smaller peak was not modelled. The match used losses of 0 mm (initial loss) and 2 mm/h (continuing loss). As with the 1988 event the 1991 event was preceded by a wet period and so it is assumed that the catchment was wet prior to the main burst.

Overall the match between the model and observed data set for the 1991 event is good and firm proof that the model, as calibrated for the 1988 event, is able to replicate observed flood behaviour for the Upper South Creek area.

4.2.4.2. 1992 Event

Figure 17 demonstrates the match between modelled and observed flow stage for the 1992 validation event. As can be seen the match is not ideal and it is particularly evident that there is a large volume discrepancy (note the area under the observed data versus the area under the modelled hydrograph). The model achieves a peak stage of approximately 42.69 mAHD and this is compared to the peak observed height for the event of 42.75 mAHD. Overall the match is not ideal and a review of daily read stations in the area indicates that the pluviometer data used in the modelling is unlikely to be indicative of actual rainfall. This combined with the lack of volume in the model results indicates that input rainfall is not sufficient and hence the volume discrepancy. To optimise the shape and volume march losses of initial 0 mm and continuing 2 mm/h were used.

4.2.5. Discussion

The calibration achieved is good in that peak flood level marks are well matched as is the stage hydrograph. The mean error of 0.15 m is good, particularly relative to the standard freeboard applied to design flood levels for determining building floor levels which is 0.5 m as per Reference 6. The validation work shows that the model is capable of emulating observed behaviour (1991 event) provided that the rainfall data is reasonably representative. Based on the calibration and validation work presented the model is suitable for use in design flood estimation.

5. DESIGN EVENT MODELLING

5.1. Introduction

As per the Brief the 5%, 2%, 1%, 0.5% and 0.2% AEP events have been modelled as has the PMF. All durations from 15 minutes to 72 hour were run for the 1% AEP event and from this work critical durations of two and nine hours were identified.

5.2. Scenarios Modelled

Two scenarios have been modelled and these are “Existing” and “Loss of Catchment Storage Sensitivity Test”.

The “Existing” scenario is the study area as is excluding development work carried out to date on the Oran Park and Turner Road sub-division developments.

The “Loss of Catchment Storage Sensitivity Test” scenario assesses the impact of urban development (without mitigation works) on downstream peak flood levels. It is assumed that if urbanisation of land occurs without compensatory works, such as installation of communal retarding basins, catchment storage decreases (due to increasing imperviousness in the catchment). As mandated by Council, installation of retarding basins to mitigate the adverse effects of urbanisation is mandatory as it aims to compensate for the loss of catchment storage and increase in imperviousness. Nevertheless the run is undertaken without mitigation in order to ascertain how unmitigated urbanisation may impact on flood behaviour in the area.

5.3. Method

5.3.1. Flood Extent Mapping

Design flood extent mapping uses peak flood levels from the two hour event for Kemps and Bonds creeks and the nine hour event for Upper South Creek. Depths less than 150 mm have been removed from the plot as its considered that flood waters less than 150 mm deep should not necessarily be indicative of whether an area is subject to flooding or not. Modelling results are provided where the flowpath's contributing area is larger than 15 ha. Flowpaths with contributing catchment areas smaller than the 15 ha threshold are likely to appear poorly defined (i.e. "puddles" along watercourse alignments may be observed). Flood extents within Turner Road and Oran Park precincts have not been mapped. As development is currently being carried out in those precincts, flood extents and peak do not provide consistency by the tie the hydraulic model was built. Therefore, any result shown within those areas will not be consistent to those with the rest of the catchment.

5.3.2. Provisional Hazard and Hydraulic Categories

Hazard presented is based on calculation of low and high hazard as per Reference 6. In simple terms this means that when depth is greater than one metre or when the velocity/depth product exceeds 1 m^2/s the area is defined as high hazard. The remainder of the flooded extent in the any design event is then classified low hazard.

Hydraulic categorisation is less precise in that although the categories of floodway, flood fringe and flood storage are all qualitatively defined in Reference 6, there is no established method by which this work may be precisely carried out and different approaches are used by different consultants and authorities. Based on work carried out by Howells (Reference 7) and as per previous studies where similar criteria have been used, WMAwater have used an approach whereby the velocity/depth product is used as follows:

Floodway = V*D > 0.25 m^2/s and V > 0.25 m/s or V > 1m/s.

Flood Storage we would define as depth > 1m and NOT in a Floodway

- It is likely that the results provided using the criteria are conservative i.e. in some circumstances the criteria are likely to describe a greater floodway extent than might otherwise be defined. Note that it will be up to the FRMS+P to finalise the map.

5.4. Results

5.4.1. Introduction

Note that the main outputs of the study are digital data layers of inundation extent and provisional hazard and hydraulic categories that will be subsequently used by Council in planning work and DA assessment. Whilst hardcopies of this information are provided herein given the data is presented for an area of approximately 70 km^2 at a high resolution it will be best be viewed digitally or in large format maps. In order to aid comprehension of results in report format profiles and tables are also provided.

5.4.2. General

Design flood results are presented via inundation extent for the 1% AEP event (Figure 18), 1% AEP Provisional Hazard (Figure 19), 1% AEP Provisional Hydraulic Categorisation (Figure 20) and then the 5% AEP and PMF extents (Figure 21 - Figure 22). Also presented in Appendix D are profiles indicating the range of design flood heights for events modelled and how design flood levels interact with road crossings as well as proximate streets. Appendix E contains Provisional Hazard and Hydraulic Categorisation for the 5%, 2%, 0.5% and 0.2% AEP events.

Table 9 presents limited summary results for modelling work.

Table 11 and Table 12 present sensitivity results for the Kemps/Bonds Creek model using the 1% AEP 2 hour event. The results indicate that design flood peak flood levels are insensitive to the variety of settings for which sensitivity was tested. Blockage produced results in which impacts remained less than 0.1 m and hence modelled results remain insensitive to blockage also. The sensitivity due to blockage noted in the calibration section was produced due to the fact that the 1988 event is smaller than a 1% AEP event. Smaller floods have less flow over Bringelly Road and hence a more substantial proportion of flow is dependant on blockage of the structure.

Table 9: Selected results from design modelling

	South Creek	Kemps Creek	Bonds Creek
Design events critical duration	9h	2h	2h
PMF critical duration	4h	1h	1h
Catchment area (km^2)	61	5.5	4.5
Inundated extent in 1% AEP (km^2)	9.5 (16%)	0.6 (11%)	0.4 (9%)
Inundated extent in PMF (km^2)	16.6 (27%)	1.3 (24%)	1.0 (22%)
Mean flood depth 1% AEP (m)	0.61	0.39	0.46
Mean flood depth PMF (m)	1.10	0.78	0.81
1% AEP Extent High Hazard / Low Hazard	3.58 km^2 (37%) / 5.92 km^2 (62%)	0.09 km^2 (15%) / 0.52 km^2 (85%)	0.14 km^2 (29%)/ 0.34 km^2 (71%)

Figure 23 shows the 1% AEP velocity distribution for a cross-section extracted from the model approximately 500 m upstream of Bringelly Road on South Creek. The figure is presented as it was considered that it would be of benefit to a discussion herein regarding the provisional hydraulic categorisation.

A key hydraulic categorisation is the floodway extent as it is this which will likely define the first cut on which areas can/cannot be utilised for residential development. No precise quantifiable definition for floodway exists however Reference 6 describes it as the extent, within the larger flood extent, responsible for conveying the majority of flow and notes that floodways will often be aligned with obvious natural channels.

Figure 20 shows that near the entire flood extent has provisionally been defined “floodway”. As described above floodway has been defined on the basis of quantitative criteria (see Section 5.3.2 for details). Whilst at first assessment it may seem that the criteria used to define floodway are overly conservative Figure 23 does provide some insight into the result presented in Figure 20. As can be seen the 1% AEP depth throughout the sampled cross-section exceeds 1 m in all areas. That is, the floodplain being modelled is relatively flat and well defined and this explains why inundation extents for events of differing rarity do not vary significantly. The floodplain shape also justifies a floodway extent that includes the majority of the 1% AEP flood extent. Notwithstanding the provisional hydraulic categorisation presented herein, it will be up to the subsequent Floodplain Risk Management Study and Plan to update the hydraulic categorisation from provisional to final.

Table 10: Summary of structure and road crossing results

Culvert	U/S or D/S controlled						Flow at Full capacity						Flood Level Above Road Level					
	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Barry Ave (Rileys Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Anthony Rd (Rileys Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Camden Valley Way (Rileys Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Camden Valley Way (South Ck)	D/S	D/S	D/S	D/S	D/S	D/S	No	No	Yes	Yes	Yes	Yes	No	No	No	No	No	Yes
Northern Rd (approx 1 Km south of Carrington Rd - trib to South Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Masterfield St (South Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	No	No	Yes	Yes	Yes	Yes
Bringelly Rd (Thompsons Ck)	U/S	U/S	U/S	U/S	U/S	U/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Bringelly Rd (Kemps Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Eastwood Rd (Kemps Ck)	D/S	D/S	D/S	D/S	D/S	D/S	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Heath Rd (Kemps Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Park Rd (Bonds Ck)	D/S	D/S	D/S	D/S	D/S	D/S	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Heath Rd (Bonds Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Rickard Rd (Bonds Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Ingleburn Rd (Bonds Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Bringelly Rd (Bonds Ck)	D/S	D/S	D/S	D/S	D/S	D/S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Table 11: Sensitivity Results - Roughness, Blockage and Downstream Boundary

Location	Base case: 100y ARI-2h	25% Decrease in Manning's "n"	25% Increase in Manning's "n"	50% decrease in blockage	50% increase in blockage	Downstream boundary decreased by 0.5m	Downstream boundary increased by 0.5m
	Peak heights (mAHD)	Difference (m)	Difference (m)	Difference (m)	Difference (m)	Difference (m)	Difference (m)
Bonds Creek - DS Bringelly Rd.	72.98	-0.12	0.03	0.04	-0.03	0.00	0.00
Bonds Creek - US Bringelly Rd.	73.87	-0.20	-0.01	-0.09	0.05	0.00	0.00
Kemps Creek - DS Bringelly Rd.	74.13	-0.03	0.01	0.01	-0.02	0.00	0.00
Kemps Creek - US Bringelly Rd.	74.31	0.03	0.00	-0.02	0.03	0.00	0.00
US McCann Rd	79.88	0.00	0.02	0.00	0.00	0.00	0.00
US Eastwood Rd	79.59	0.03	0.02	-0.02	0.02	0.00	0.00
US Heath Rd	84.81	0.01	0.03	0.00	0.01	0.00	0.00
Back of 146B Dickson Rd	76.53	-0.26	0.01	-0.02	0.02	0.00	0.00
US Ingleburn rd	81.70	-0.12	0.00	-0.05	0.05	0.00	0.00
US Rickard rd	85.13	-0.22	0.02	0.00	0.00	0.00	0.00

Table 12: Sensitivity Results – Energy Loss, Infiltration, Climate Change

Location	Base case: 100y ARI-2h	Energy loss at structures decreased by 50%	Energy loss at structures increased by 50%	Hydrological losses: Initial loss 15 mm - Continuing loss 3 mm/h	Hydrological losses: Initial loss 1.4 mm - Continuing loss 1 mm/h	Increase in imperviousness by 5%	Climate change: Rainfall intensity increased by 10%	Climate change: Rainfall intensity increased by 20%	Climate change: Rainfall intensity increased by 30%
	Peak heights (mAHD)	**Difference (m)**	**Difference (m)**	**Difference (m)**	**Difference (m)**	**Difference (m)**	**Difference (m)**	**Difference (m)**	**Difference (m)**
Bonds Creek - DS Bringelly Rd.	72.98	0.01	0.01	0.00	0.03	0.00	0.02	0.04	0.07
Bonds Creek - US Bringelly Rd.	73.87	-0.03	-0.01	0.00	0.07	0.00	0.05	0.10	0.15
Kemps Creek - DS Bringelly Rd.	74.13	0.01	0.00	0.00	0.05	0.00	0.03	0.06	0.09
Kemps Creek - US Bringelly Rd.	74.31	-0.02	0.00	0.00	0.05	0.00	0.03	0.06	0.10
US McCann Rd	79.88	0.00	0.00	0.00	0.05	0.00	0.03	0.06	0.09
US Eastwood Rd	79.59	-0.01	0.00	0.00	0.09	0.01	0.07	0.12	0.17
US Heath Rd	84.81	0.00	0.00	0.00	0.05	0.00	0.04	0.07	0.10
Back of 146B Dickson Rd	76.53	0.00	0.00	0.00	0.07	0.00	0.05	0.10	0.15
US Ingleburn rd	81.70	0.00	0.01	0.00	0.06	0.00	0.04	0.08	0.12
US Rickard rd	85.13	0.00	0.00	0.00	0.09	0.01	0.06	0.12	0.16

5.4.3. Loss of Catchment Storage Sensitivity Test

As per previous details the sensitivity of design peak flood levels has been assessed against loss of catchment storage, such as would occur in the case of limited urban development (unaccompanied by mitigation works). To facilitate the work the proposed extents of the Turner Road and Oran Park developments have been utilised.

Results on the impact of the developments are provided below. Note that upper locations referred to are shown in Figure F1 of Appendix F whilst other locations can also be identified from Figure 13.

Table 13: Future Development Results - Peak Water Level at Various Locations

	5% AEP		1% AEP		PMF	
Location	**Existing**	**Future**	**Existing**	**Future**	**Existing**	**Future**
Turner Road U/S Camden Valley Way	92.1	92.1	92.7	92.7	96.7	96.7
Oran Park 1	83.3	83.3	83.4	83.4	84.1	84.1
Oran Park 2	82.5	82.6	82.6	82.7	83.2	83.2
Oran Park 3	82.8	82.8	82.9	82.9	83.6	83.7
Catherine Fields Rd.	67.3	67.3	67.4	67.4	68.6	68.6
Barry Ave.	66.2	66.2	66.4	66.4	68.0	68.0
Robens Cr.	63.7	63.7	63.8	63.8	64.8	64.8
Masterfield St.	59.7	59.7	60.0	60.0	61.7	61.7
D/S Bringelly Rd	58.1	58.1	58.3	58.3	60.3	60.3

Table 14: Future Development Results - Peak Flow Rates at Various Locations

	5% AEP		1% AEP		PMF	
Location	**Existing**	**Future**	**Existing**	**Future**	**Existing**	**Future**
Turner Road U/S Camden Valley Way	31.7	30.4	43.4	42.5	146.2	141.8
Oran Park 1	18.7	18.2	30.4	28.6	130.7	128.4
Oran Park 2	10.3	10.6	15.4	15.2	75.8	71.5
Oran Park 3	26.6	26.1	39.9	38.1	267.6	259.6
U/S Bringelly Rd	237.4	235.6	360.6	356.7	1773.6	1754.7

5.5. Discussion

A key finding from the results is the lack of flood height sensitivity for events of markedly different probability of occurrence. For example flood levels for the 5% AEP event are less than 0.5 m lower than the 1% AEP. Comparison of the flood extent for the 1% and 5% AEP events suggests that whilst depths have changed slightly the flood extents for the two events are near identical.

A further finding of note is that design flood levels are highly insensitive to the variety of parameter settings that have been made in the modelling work. This is a facet of the modelled system. Its properties that make it insensitive are as follows:

- A wide floodplain which means that changes in discharge translate into minimal changes in flood level;

- Roads and other downstream hydraulic controls tend to create backwater areas and are often overtopped. This negates the importance of structure blockage and losses as well as contributing to the following point; which is that
- Low flow velocities owing to low slopes and downstream road controls mean that roughness has very little impact on model results.

A positive implication of the sensitivity modelling is that design flood levels should be utilised with a great deal of confidence in their accuracy that is, insensitive model results mean that the model design estimates are robust. Presuming a freeboard of 0.5 m is used (as per Reference 6) it is certainly the case that residential/business building floor levels can be set with a great deal of confidence as regards their future safeguarding against flooding.

The insensitivity of the flood level results for most locations, particularly relevant to freeboard of 0.5 m, is also shown in the modelling of future development impact. The impact of loss of catchment storage is largest at downstream volume sensitive locations (such as upstream of hydraulic controls like Bringelly Road). The impact in the 1% AEP event is however limited to 0.3 m. Note also that whilst discharges are more impacted by the development, changes to flows are also negligible.

6. CONCLUSIONS

The Study has defined design flood behaviour for the Upper South Creek area. The model used to define design flood liability is calibrated and validated and is shown to be relatively insensitive to model parameters. As such it is expected that design flood estimates are accurate, hence the model can be used with confidence in regard to safe guarding future development against risk of inundation.

The range of flood heights is relatively compressed in that a relatively small difference exists between peak flood levels for design events of markedly different probability. This is a facet of the local topography i.e. relatively wide and well defined floodplain. Mapping of provisional hydraulic categories has been provided and this work identified that, based on specific criteria, the floodway area approximately corresponds to the inundation extent. This unusual result is again related to the specific topography of the study area, i.e. a well defined and high flow/storage capacity overbank area.

Modelling carried out in order to assess the impact of urbanisation within the currently on-going sub-division developments of Oran Park and Turner Road has identified that urbanisation produces small impacts on the volume and peak flow of flood runoff. For downstream volume sensitive locations this can lead to an exacerbation of peak flood levels. In the 5% AEP event for example the two developments (without mitigation elements implemented in modelling) result in an increase in flood heights at Bringelly Road of 0.1 m.

7. ACKNOWLEDGEMENTS

WMAwater wishes to acknowledge the assistance of Camden City Council staff in carrying out this work as well as the Office of Environment and Heritage for providing funding assistance towards the Study under the NSW Floodplain Management Program.

8. REFERENCES

1 South Creek Flood Study, DWR, 1990

2 South Creek Floodplain Management Study, DWR, Volumes 1 and 2, 1991

3 Austral Floodplain Risk Management Study and Plan, LCC, 2003

4. South Creek Floodplain Risk Management Study and Plan, LCC, 2004

5. Pilgrim H (Editor in Chief), Australian Rainfall and Runoff – A Guide to Flood Estimation, Institution of Engineers, Australia, 1987.

6. NSW State Government, Floodplain Development Manual, April 2005.

7. Howells, L., McLuckie, D., Collings, G. & Lawson, N. (2003). *Defining the Floodway – Can One Size Fit All?,* Proceedings of 43rd Floodplain Management Authority of New South Wales Annual Conference, Forbes, 25-28 February 2003.

8. Brown Consulting, March 2007 Oran Park Precinct Masterplan - Stormwater Quantity Management & Flooding

9. GHD, March 2007 (2nd Draft) Turner Road Precinct Planning - Water Sensitive Urban Design Strategy

10. Gray, S. And Ball, J.E. 32nd Hydrology and Water Resources Symposium, Newcastle 2009

11. Clark *et al*. 32nd Hydrology and Water Resources Symposium, Newcastle 2009