LITTLE CREEK CATCHMENT OVERLAND FLOW FLOOD STUDY

VOLUME 1 – FINAL REPORT

PENRITH CITY COUNCIL

JUNE 2017

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Client Penrith City Council	
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LIST OF ABBREVIATIONS

1D	One (1) Dimensional
2D	Two (2) Dimensional
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ALS	Airborne Laser Scanning (same as LiDAR)
ARR	Australian Rainfall and Runoff
BoM	Bureau of Meteorology
DEM	Digital Elevation Model
DRAINS	Hydrologic computer model
EY	Exceedances per Year
GIS	Geographic Information System (a spatial database)
HC Survey	Hydrographic and Cadastral Survey Pty Ltd
IFD	Intensity-Frequency-Duration of Rainfall
LC	Little Creek
LGA	Local Government Area
LiDAR	Light Detection and Ranging (same as ALS)
MIKE-11	1D hydraulic computer model
OEH	NSW Office of Environment and Heritage
PCC	Penrith City Council
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RAFTS	Hydrologic computer model
RMA-2	2D Hydraulic computer model
SWC	Sydney Water Corporation
TIN	Triangular Irregular Network
TUFLOW	1D/2D Hydraulic computer model

FOREWORD

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

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government provides funding for flood studies, floodplain risk management plans and works to alleviate existing problems, to undertake the necessary technical studies to identify and address the problem and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities. The Federal Government may also provide funding in some circumstances.

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

1. Flood Study
Determines the nature and extent of the flood problem

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

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

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

The Little Creek Overland Flow Flood Study constitutes the first stage of the management process for the catchment. This study has been prepared by WMAwater for Penrith City Council and was undertaken to provide the basis for future management of flood liable lands within the study area.

EXECUTIVE SUMMARY

BACKGROUND

The Little Creek catchment is located north of the M4 Motorway and drains to South Creek with the study area covering parts of the suburbs of Oxley Park, Colyton, St Marys and North St Marys. The catchment includes crossings of the Great Western Highway and the Main Western railway line and covers an area of approximately 480 hectares. The study components are to:

- collate available historical flood related data;
- collect detail survey of stormwater infrastructure;
- undertake a community consultation program;
- prepare suitable models for use in a subsequent Floodplain Risk Management Study;
- validate the models against historical events;
- undertake sensitivity of the model results to modelling parameters and assumptions;
- provide design flood levels, depths, velocities, flows and flood extents;
- assess the capacity of the existing drainage network and identify potential upgrades;
- assess the sensitivity to potential climate change effects; and
- assess floodplain planning categories and undertake provisional hazard mapping.

COMMUNITY CONSULTATION

Approximately 3,700 questionnaires were distributed in order to identify flood problem areas and to collate historical flood data. 195 responses were received, 32 had observed an overland flow path near their property, 23 had experienced flooding in their properties with 12 of those properties having experienced inundation above floor level.

MODELLING SUMMARY

The study used hydrologic and hydraulic modelling techniques in order to define flood behaviour in the study area. The modelling programs used in the study are:

- DRAINS Hydrologic model converts rainfall to runoff for input into the TUFLOW model.
- TUFLOW 2D Hydraulic model was established to analyse the flooding behaviour.

MODEL CALIBRATION

In order to provide robust design flood data the models should be calibrated to historical flood data but typically in an urban catchment there is insufficient high quality data available. The March 2014, October 1987 and April 1988 events were chosen for model calibration but the process was limited by the quality and quantity of the available rainfall and flood data.

DESIGN FLOOD MODELLING

Design flood levels in the catchment are a combination of flooding from rainfall over the local catchment, as well as elevated tailwater levels from flooding in South Creek. This study primarily is concerned with the Little Creek flood mechanism but South Creek flood extents should also be considered as part of any floodplain management and flood-related planning activity for the catchment. The study results have been provided to PCC in digital format and mapped in Appendix B as follows:

- Peak flood extents in Figure B1 to Figure B9;
- Peak flood depths in Figure B10 to Figure B18;
- Peak flood levels in Figure B19 to Figure B27;

- Peak flood velocities in Figure B28 to Figure B36;
- Provisional hydraulic hazard in Figure B37 to Figure B45;
- Provisional hydraulic categorisation in Figure B46 to Figure B48.

The design flood results were filtered using the following criteria:

- Depths less than 0.15 m were removed from the result maps;
- Isolated flood patches were removed if they were less than 100 m^2 in area.

OVERVIEW OF FLOOD BEHAVIOUR

The railway embankment of the Western Railway Line forms a major hydraulic feature of the Little Creek catchment. Upstream the natural creek alignment has been replaced by a piped system, and there are several sections where there is no formal overland flow path or easement above the trunk drainage line. There are several locations along this main drainage line where overland flow occurs through private development, when runoff exceeds the capacity of the stormwater network. The pipe capacity assessment indicates that the majority of the stormwater network upstream of the Great Western Highway has less than 50% AEP capacity. Away from the main drainage line, overland flow is generally along the road network. As the catchment is relatively narrow either side of the trunk alignment, there are relatively few major “tributary” overland flow paths in the upper catchment.

Downstream of the railway line, where Little Creek remains primarily an open channel, there is relatively little overbank flooding even in the 1% AEP event but this area is likely to be affected by South Creek flooding for the 5% AEP and larger events on that system.

KEY AREAS OF FLOOD RISK AND PRELIMINARY MITIGATION OPTIONS

A pipe capacity assessment was undertaken and significant portions of the upper catchment drainage network were found to have capacity less than the 50% AEP peak flow.

Based on a "hot spot" analysis, a range of potential flood mitigation options were identified, and are recommended for further investigation (see Section 10.3), these include:

- Increase the stormwater inlet capacity at Canberra/Sydney Street low points
- Increase pipe capacity upstream of Oxley Park detention basins, throughout the Carpenter Street and Bennett Road catchment areas, and particularly from Kent Place to the Great Western Highway.
- Modify the median strip on the Great Western Highway to reduce the obstruction and ponding depth in the roadway, and/or upgrade culverts under road.
- Excavate the reserve Bennett Road to Great Western Highway to provide additional detention storage, and upgrade the inlet to the Great Western Highway culverts.
- Increase pipe capacity from Jacka to Brisbane Street on the western tributary branch.
- Increase the outlet pipe capacity from the Oxley Park basin at Oxley Park Public School and/or modify spillway crest.
- Upgrade the Forrester Road bridge culvert capacity at Little Creek.
- Modify the open channel near Kurrajong Road.
- Increase the railway line cross-drainage capacity at Hobart Street.

1. INTRODUCTION

1.1. Background

The Little Creek catchment area (Figure 1) is within the Penrith City Council (PCC) local government area (LGA) and includes parts of the suburbs of Oxley Park, Colyton, North St Marys and St Marys. The catchment is located north of the M4 Motorway and drains to South Creek. The study area covers an area of approximately 480 hectares.

The area is highly urbanised with a mix of residential, commercial and industrial properties including educational institutions such as Oxley Park Public School and Colyton High School. There are also a number of open spaces including Oxley Park and Colyton runoff detention basins. The western rail line cuts across the catchment in an east to west direction and forms a notable barrier to flow.

The present study was commissioned by PCC with funding and technical assistance from the NSW Office of Environment and Heritage (OEH) to define flood behaviour in the catchment. Flooding problems have been experienced at a number of locations within the catchment during periods of heavy rainfall. The study aims to identify these problem areas so that they can be assessed for possible mitigation options in the future Floodplain Risk Management Study and Plan.

1.2. Objectives

The primary objective of this Flood Study is to define design flood behaviour for a wide range of design flood probabilities and to:

- collect detail survey of stormwater infrastructure;
- undertake a community consultation program;
- prepare suitable hydrologic and hydraulic models of the catchment and floodplain, which are suitable for use in a subsequent Floodplain Risk Management Study;
- validate the models against historical events;
- understand the sensitivity of the model results to modelling parameters and assumptions;
- provide results for flood behaviour in terms of design flood levels, depths, velocities, flows and flood extents within the study area;
- assess capacity of the existing drainage network and identify potential upgrades;
- assess the sensitivity of flood behaviour to potential climate change effects such as increases in rainfall intensities; and
- assess floodplain planning categories (such as flood planning areas, flood control lots, hydraulic categories, and emergency response categorisation), and undertake provisional hazard mapping.

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

1.3. Description of the Catchment and Flood History Overview

The catchment has a history of flooding and there is a need to define the extent of flooding and to determine appropriate development controls and floodplain risk management plans. The catchment experienced severe flooding in August 1986 and October 1987 (Reference 2) with the prolonged rainfall in August 1986 causing significant flooding to residential properties. In October 1987, a short duration intense rainfall (lasting about 90 minutes) occurred, causing damage to residential properties within the catchment. Large deposits of sediments and debris, including tree roots, according to resident accounts caused blocking of the pipe network system at several locations north of the Great Western Highway. Several roads in the catchment were inundated during both events. More recently, a flash flood in March 2014 caused widespread damage including inundation of homes and garages above floor level.

The land use of the Little Creek catchment comprises a mix of residential and commercial developments, including some light industrial, together with areas of open space including the grounds of Oxley Park Public School and Colyton High School, and Ridge Park. There are major detention basins located within Colyton High School, and the Council reserve off Whitcroft Place.

Elevations in the upper part of the catchment reach approximately 60 mAHD along the western catchment ridge (mapping of the topography from LIDAR aerial survey is shown on Figure 2). The overall catchment slope is quite consistent from the upper to lower catchment, with a grade of approximately 0.7% along the main trunk drainage alignment, which is relatively flat. The sides of the catchment valley are generally steeper, with slopes of approximately 2.5%. The catchment runs generally from the south-east to the north-west. The embankments of the Great Western Highway and Main Western Railway Line cross the catchment in an east-west direction, presenting significant obstructions to overland flow.

Drainage elements in the catchment include kerbs and gutters, pits and pipes, and a network of trunk drainage elements including culverts and open channels. These drainage assets are primarily owned by PCC. Extensive survey of the stormwater network and major hydraulic structures (mainly pits, pipes, structures and cross section across creeks) was undertaken (Figure 3) to inform the hydraulic modelling.

1.4. Community Consultation

A newsletter and questionnaire were distributed to residents within the catchment. The newsletter described the role of the Flood Study and requested information on experiences of flooding in the catchment. 195 responses were received from approximately 3,700 distributed questionnaires.

Of those that responded 32 had observed an overland flow path near their property, 23 had experienced flooding in their properties with 12 of those properties having experienced inundation above floor level. There were 18 responses recounting damage to the property. Some peak flood photos from community are shown in Photo 1 (see page 3).

Photo 1: Flood Photographs from March 2014 at Edmondson Avenue, St Marys

Some statistics from the returned questionnaires are shown in Figure 4. The responses identified the following general points:

- Overland flow was frequently observed in some areas, such as Ball Street, Canberra Street, Muscio Street and Great Western Highway.

- Many residents have had their daily routines affected and believe that their safety has been put at risk due to localised stormwater flooding.
- Most of flood damages occurred to garages and some properties (e.g. Carpenter Street) were affected by flooding on an almost annual basis.
- Some affected residents have employed their own flood mitigation measures; including building drains on the side of property to channel the water to the road.

2. AVAILABLE DATA

2.1. Overview

The first stage in the investigation of flooding matters is to establish the nature, size and frequency of the problem. On larger urban river systems such as the Hawkesbury River there are generally stream height and historical records dating back a considerable period, in some cases over one hundred years. However, in smaller urban catchments such as Little Creek there are generally no stream gauges or official historical records available. In some creek systems there are permanent water level gauges or maximum height recorders (post that records a "tide mark") however there is no such data available for the Little Creek catchment.

An understanding of historical flooding must therefore be obtained from an examination of Council records, previous flood assessment reports, rainfall records and local knowledge obtained through community consultation (Section 1.4).

2.2. Data Sources

Data utilised in the study has been sourced from a variety of organisations. Table 1 gives a summary of the type of data sourced, the supplier, and its application for the study.

Table 1: Data Sources

Type of Data	Format Provided (Source)	Application
Ground levels from ALS data (2002)	DEM (PCC)	Hydrologic and hydraulic models
Ground levels from ALS data (2011)	DEM (LPI)	Hydrologic and hydraulic models
Bathymetry of Watercourses	GIS (HC Survey)	Hydraulic model
Indicative Pit/Pipe Layout	GIS (Council)	Preparation of detail survey brief
Pits, Pipes and Hydraulic Structures	GIS (HC Survey)	Hydraulic model
GIS Information (Cadastre)	GIS (PCC)	Hydraulic model
ARR Design Rainfalls	Tabulated (BoM)	Hydrologic model
Rainfall Gauge (Daily)	Spreadsheet (BoM)	Hydrologic model
Pluviometer (Continuous)	Spreadsheet (SWC)	Hydrologic model

2.3. Topographic Data

Airborne Light Detection and Ranging (LiDAR), also known as Airborne Laser Scanning (ALS) survey of the catchment and its immediate surroundings was provided for the study by PCC. There are two sets of LiDAR data collected in 2002 and 2011 respectively. These data typically have accuracy in the order of:

- +/- 0.15m (for 70% of points) in the vertical direction on clear, hard ground; and
- +/- 0.75m in the horizontal direction.

The accuracy of the LiDAR data can be influenced by the presence of open water or vegetation (tree or shrub canopy) at the time of the survey which means in some areas data is missing or the points are of poor quality. The quality of the LiDAR data can also be influenced by the filtering method used by the data provider to identify ground and non-ground points, and to remove extraneous information.

From this data, a Triangular Irregular Network (TIN) was generated. This TIN was sampled to create a Digital Elevation Model (DEM), which formed the basis of the two-dimensional hydraulic modelling for the study (Figure 2).

2.3.1. Comparison of LiDAR Datasets

An analysis and comparison of the available LiDAR datasets was undertaken to determine which would be most suitable for use for the modelling. Typically it is desirable for catchment flood studies to incorporate the most recent available topographic information, as this is most likely to include recent changes to the catchment such as new development or re-development, road works, ground levels, and creek sedimentation or erosion. Additionally, for LiDAR there have been significant improvements in the hardware and software algorithms for obtaining and classifying LIDAR information between 2002 and 2011.

Validation of the available LiDAR datasets was undertaken by comparing levels at the state survey marks and surveyed stormwater inlets to determine whether either of the LiDAR surfaces was significantly more accurate than the other. The comparison was undertaken for approximately 660 stormwater inlet pits from the detail survey obtained for the study, and 20 state survey benchmarks (SSMs).

For the SSM comparison:

- Both the 2002 and 2011 datasets had an average error within 0.02 m.
- The 2011 LiDAR had a slightly smaller “spread” of errors, with a standard deviation of 0.08 m compared to 0.12 m for the 2002 dataset.
- Assuming a normal distribution of the errors, this implies a 95% accuracy of 0.16 m for the 2011 LiDAR, and 0.24 m for the 2002 data.

For the comparison with the detail survey of inlet pits:

- Both the 2002 and 2011 datasets had similar average differences from the detail survey of 0.1 m and 0.13 m respectively. This bias is to be expected due to the nature of the comparison, since the detail survey of the inlet level was collected at the lowest point in the gutter, but the LiDAR is more likely to indicate a higher level, either at the top of kerb or a higher point in the roadway. This bias therefore reflects a typical kerb height of 0.1 m to 0.15 m.
- The 2011 LiDAR had a significantly smaller “spread” of differences, with a standard deviation of 0.11 m compared to 0.21 m for the 2002 LiDAR.
- Assuming a normal distribution of the errors, this implies a 95% accuracy of 0.22 m for the 2011 LiDAR, and 0.42 m for the 2002 data.

A histogram of the mean-corrected differences between the LiDAR and surveyed inlet levels is shown in Diagram 1 and Diagram 2. In both cases, the errors show a similar distribution, although there are more locations showing a significant difference of more than 0.25 m for the 2002 comparison.

Diagram 1: Histogram of differences between 2002 LiDAR levels and surveyed pit inlet levels

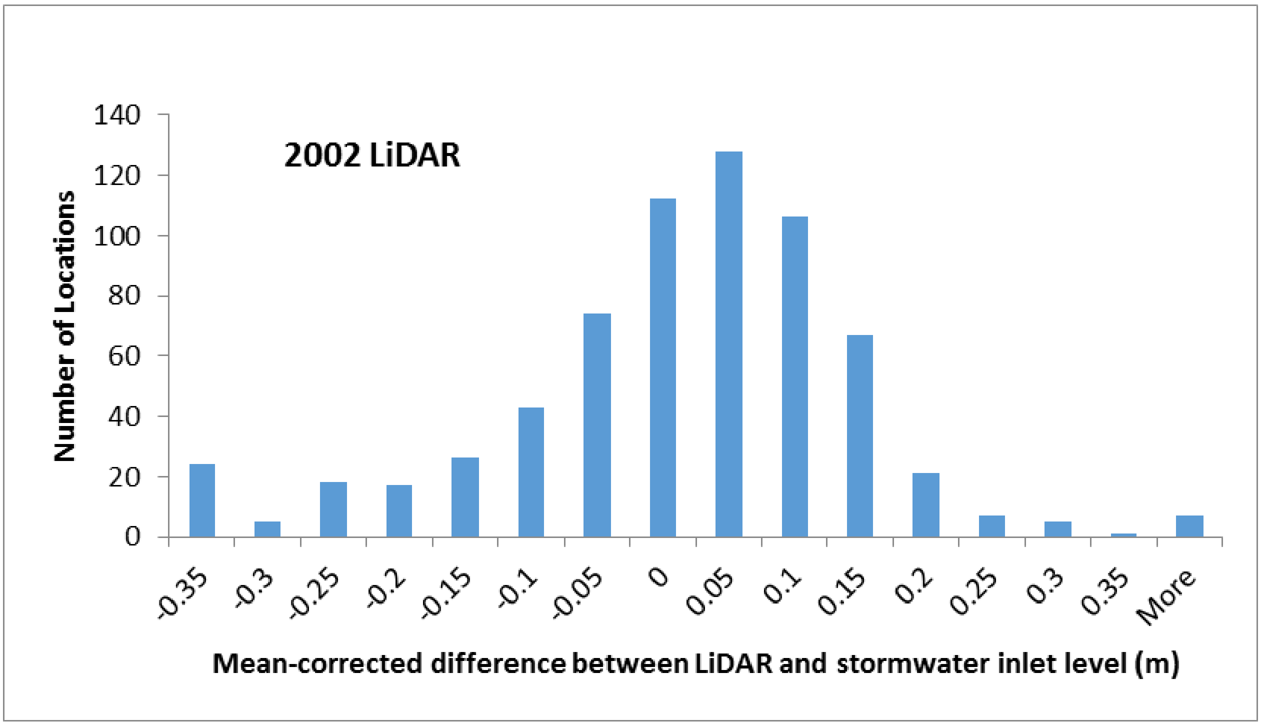
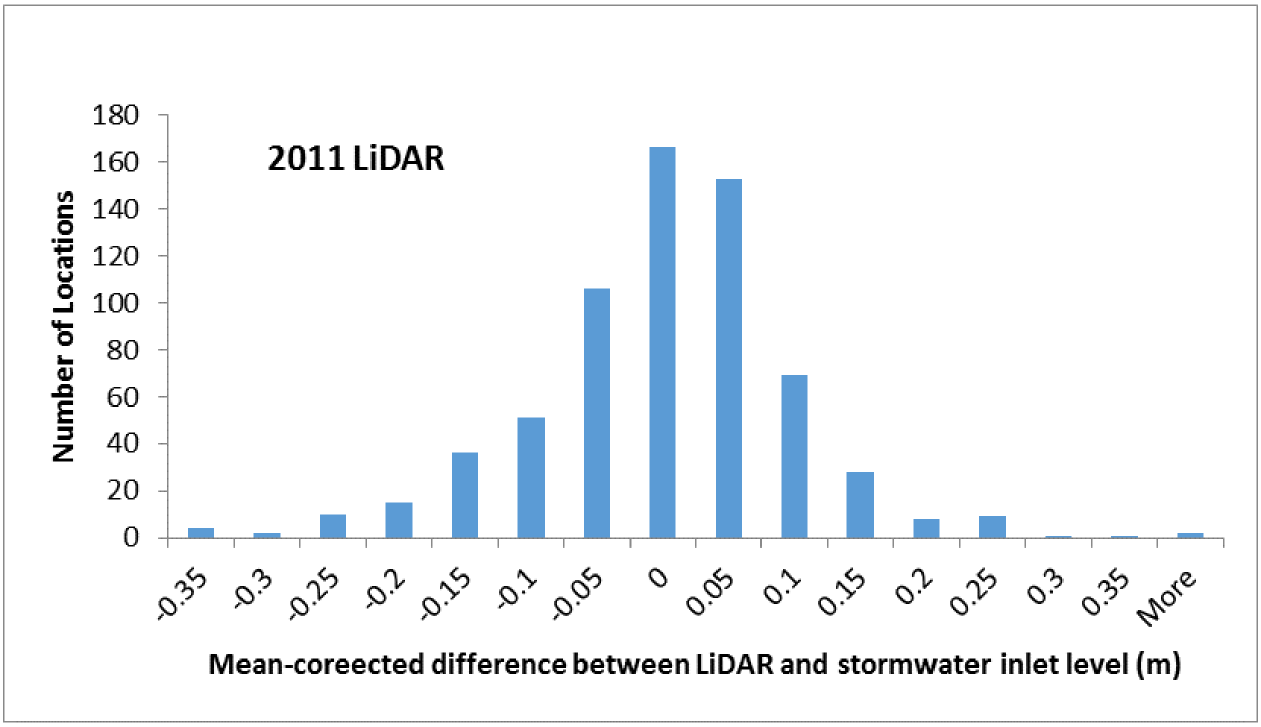


Diagram 2: Histogram of differences between 2011 LiDAR levels and surveyed pit inlet levels



Based on this analysis, the two LiDAR datasets appear to have similar levels of accuracy, with the 2011 LiDAR having slightly fewer locations with significant differences compared with detailed survey in the study area.

A graphic representation of each LiDAR dataset is shown in Diagram 3 (2002) and Diagram 4 (2011). The cooler colours (blue) represent the low-lying areas of the detention basin near Whitcroft Place, with higher elevation represented by the warmer colours.

Diagram 3: 2002 LiDAR data near Whitcroft Place detention basin

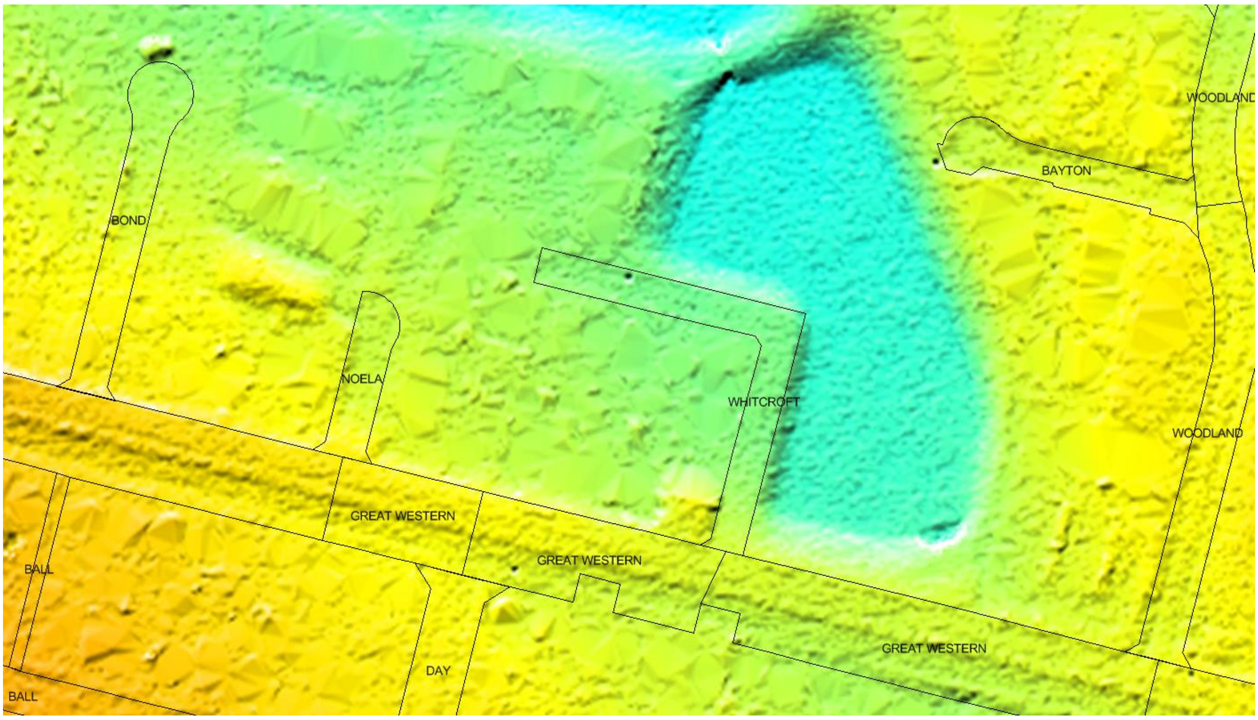
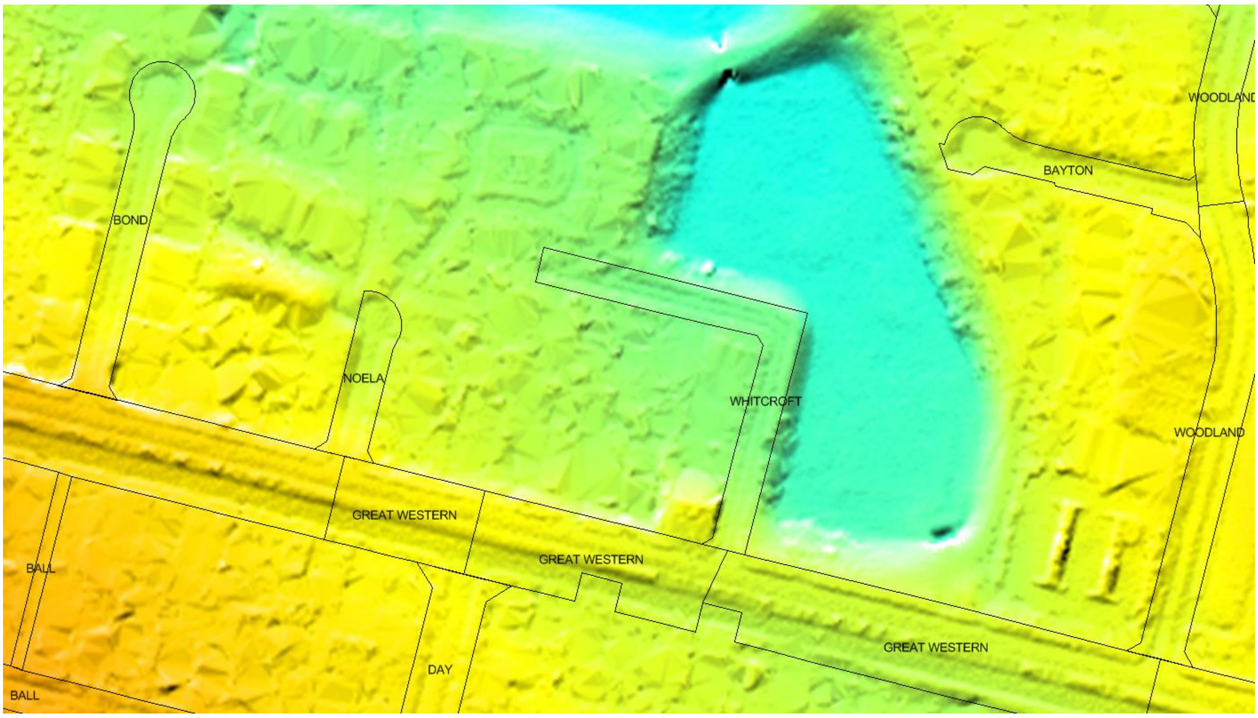


Diagram 4: 2011 LiDAR data near Whitcroft Place detention basin



A qualitative assessment of these images indicates that the 2011 data has less localised scatter, and key hydraulic features relating to overland flow such as roadways, kerb lines, and the median strip on the Great Western Highway are more clearly represented.

Finally, as inspection of the differences between the 2002 and 2011 aerial survey indicated there have been significant changes in the period from 2002, including development of sites, modifications to roadways and the rail embankment, filling and removal of spoil heaps, etc. These changes could not be readily integrated into the 2002 LIDAR due to the scale and nature of the changes.

Based on the above analysis, it was determined that the 2011 dataset was preferable for use as the predominant topographic dataset for input into the 2D hydraulic model for this study area.

2.4. Cross-Section Survey

Within the Little Creek catchment, the topography of the open watercourse areas is not properly captured by the LiDAR data, as most of the watercourses are covered or surrounded by heavy vegetation. LiDAR is less accurate in vegetated areas than for open ground. In the case of open water, the water surface in the watercourse will typically be captured in the LiDAR data, not the bed level.

Supplementary detail survey was therefore obtained to define the bathymetry of key watercourses. Hydrographic and Cadastral Survey Pty Ltd (HC Survey) undertook surveying of these cross-sections (indicative locations shown on Figure 3, based on the database available prior to the survey). Plans of the cross-section survey are provided in Appendix E.

2.5. Pit and Pipe Data

An indicative database of stormwater pits and pipes within the catchment was provided by PCC. Council advised that this database was primarily generated from digitisation of old development plans, and that its expected accuracy was relatively low. The database contained only limited information about the pit of pipe geometry and was therefore of limited use for direct input into the hydraulic TUFLOW model. The database was primarily used to develop a survey brief to obtain more detailed information about the pits and pipes.

The additional detailed survey of drainage pits and pipes was carried out by HC Survey. The survey work also included other hydraulic control structures, such as detention basins and their outlet embankments, culverts, bridges etc. A summary of the detail survey information obtained is presented on Figure 3.

Minor corrections and additions to the detailed pit and pipe survey were required to ensure consistency within the model and to add in detail where field data could not be provided (pit not found or pit lid could not be lifted). These verifications and corrections were mainly undertaken by referring back to Works-As-Executed (WAE) survey plans provided by Council, or other historical plans where available. In some instances, it was necessary to infer the size and invert level of intermediate pipes and junction pits based on the information available upstream and downstream.

2.6. Historical Flood Level Data

Historic flood level data were obtained from the community consultation (Section 1.4). Approximately 20 flood marks of varying quality were reported by local residents. WMAwater followed up with each resident who provided a flood mark, to obtain a detailed description of the location, flood depth, and general flow behaviour in each case. These flood marks were used for hydraulic model validation purposes. The locations of these flood marks are shown on Figure 5. A comparison of calibration results with these flood marks is presented in Section 6.4.

2.7. Historical Rainfall Data

2.7.1. Overview

Rainfall data is recorded either daily (24-hour rainfall totals to 9:00 am) or continuously (pluviometers measuring rainfall in small increments – less than 1 mm). Daily rainfall data has been recorded for over 100 years at many locations within the Sydney basin. However pluviometers have only been installed for widespread use since the 1970s.

Care must be taken when interpreting historical rainfall measurements. Rainfall records may not provide an accurate representation of past flooding due to a combination of factors including local site conditions, human error or limitations inherent to the type of recording instrument used. Examples of limitations that may impact the quality of data used for the present study are highlighted in the following:

- Rainfall gauges frequently fail to accurately record the total amount of rainfall. This can occur for a range of reasons including operator error, instrument failure, overtopping and vandalism. In particular, many gauges fail during periods of heavy rainfall and records of very intense events are often lost or misrepresented.
- Daily read information is usually obtained at 9:00 am in the morning. Thus if a single storm is experienced both before and after 9:00 am, then the rainfall is “split” between two days of record and a large single day total cannot be identified.
- In the past, rainfall over weekends was often erroneously accumulated and recorded as a combined Monday 9:00 am reading.
- The duration of intense rainfall required to produce overland flooding in the study area is typically less than 4 hours (though this rainfall may be contained within a longer period of rainfall). This is termed the “critical storm duration”. For the study area a short intense period of rainfall can produce more severe flooding than sustained rainfall with a higher total depth. If the rain occurs quickly (e.g. a thunder storm), the daily rainfall total may not necessarily reflect the severity of the storm and the subsequent flooding. Alternatively the rainfall may be relatively consistent throughout the day, producing a large total but only minor flooding.
- Rainfall records can frequently have “gaps” ranging from a few days to several weeks or even years.
- Pluviometer (continuous) records provide a much greater insight into the intensity (depth vs. time) of rainfall events. This data has much fewer limitations than daily read data, but

there are far fewer pluviometers available in the vicinity of the catchment.
- Pluviometers have moving parts and automated recording mechanisms, which can fail during intense storm events due to the extreme weather conditions.

Intense rainfall events which cause overland flooding in highly urbanised catchments are usually localised and as such are only accurately represented by a nearby gauge, preferably within the catchment. Gauges sited even only a kilometre away can show very different intensities and total rainfall depths.

2.7.2. Rainfall Stations

Table 2 presents a summary of the official rainfall gauges operated by the BoM located close to or within the catchments (mapped on Figure 6).

Table 2: Daily rainfall stations within 5 kms of the centre of the catchment

Station Number	Station Name	Operating Authority	Distance from centre of the catchment (km)	Elevation (mAHD)	Date Opened	Date Closed	Type
67024	St Marys Bowling Club	BoM	1.0	35	1897	1984	Daily
67003	Colyton (Carpenter St)	BoM	2.0	45	2000	2008	Daily
67083	Mount Druitt Francis Street	BoM	2.3	40	1970	1976	Daily
67025	St Marys	BoM	3.3	24	1947	1973	Daily
67102	St Clair (Juba Close)	BoM	4.7	45	1985	2013	Daily
67116	Willmot (Resolution Ave)	BoM	4.8	30	1995	current	Daily

2.7.3. Analysis of Daily Read Data

An analysis of the records of the daily rainfall stations St Marys Bowling Club (67024) and St Clair (Juba Close) (67102) was undertaken. St Marys Bowling Club and St Clair are located to the west and south of the catchment and are shown on Figure 6. These gauges were chosen for analysis because they had relatively continuous periods of record, which covered the longest combined historical period.

From this data (Table 3) it can be seen that August 1986 was by far the largest event recorded at St Clair. The July 1988, May 1962, July 1904, April 1946 and February 1990 storm events also were significant but of much lesser total rainfall in a single day. Another notable event in the local area not identified in these daily read records (identified by residents), but when flooding is noted to have occurred, was October 1987.

Table 3: Large Daily Rainfalls at St Marys Bowling Club and St Clair

St Marys Bowling Club (67024)		
1897 – Dec 1984		
Rank	**Date**	**Rainfall (mm)**
1	14/05/1962	187.7
2	10/07/1904	166.4
3	16/04/1946	150.9
4	17/02/1932	146.8
5	10/02/1956	146.8
6	25/03/1906	146.6
7	29/04/1963	143.3
8	2/09/1970	139.4
9	13/12/1910	138.4
10	7/08/1967	135.4

St Clair (Juba Close) (67102)		
1985 – July 2013		
Rank	**Date**	**Rainfall (mm)**
1	6/08/1986	262.4
2	3/02/1990	147
3	6/07/1988	140.8
4	7/02/1990	134.6
5	10/06/1991	130.2
6	4/02/1990	116.8
7	10/02/1992	115.8
8	31/01/2001	115
9	9/02/1992	110.4
10	1/05/1988	110

2.7.4. Analysis of Pluviometer Data

Continuous pluviometer records provide a more detailed description of temporal variations in rainfall. The Tenbee and St Marys STP pluviometer stations were analysed. These pluviometer stations are operated by SWC, with St Marys STP having the longest records. The Tenbee pluviometer, about 7 km east to the catchment, closed in October 1988 but the St Marys STP gauge, about 1 km north to the catchment, is still working (see Figure 6 for locations).

The largest storms recorded at these pluviometers are listed in Table 4 but there is very little agreement between them. The 24th October 1987 event produced the highest intensity for three storm bursts at the Tenbee pluviometer (the only pluviometer available with a record of that storm). The major rainfall events tabulated below conform with the dates of observed historical flooding on the catchment.

Table 4: Peak Burst Intensities of Signficant Rainfall Events (mm/h)

	Tenbee (568074)			**St Marys STP (567087)**		
Rainfall Event	30 min	1 hour	2 hour	30 min	1 hour	2 hour
16th January 1986	50	26	13	21	11	11.3
5th August 1986				27	19	17.8
6th August 1986	41	29	23			
24th October 1987	69	47	31			
30th April 1988	47	28	17	30	21	22
6th July 1988	38	23.5	18.5	22	14	11
30th March 2014				49	29	18.8

Table 5: Approximate AEP of Pluviometer Storm Bursts

Rainfall Event	Tenbee (568074)			St Marys STP (567087)		
	30 min	1 hour	2 hour	30 min	1 hour	2 hour
16th January 1986	50% to 20% AEP	> 50% AEP	> 50% AEP	> 50% AEP	> 50% AEP	> 50% AEP
5th August 1986				> 50% AEP	> 50% AEP	> 50% AEP
6th August 1986	50% AEP	50% AEP	50% AEP			
24th October 1987	10% to 5% AEP	10% to 5% AEP	10% to 5% AEP			
30th April 1988	50% AEP	50% AEP	50% AEP	> 50% AEP	> 50% AEP	50% AEP
6th July 1988	> 50% AEP	> 50% AEP	> 50% AEP	> 50% AEP	> 50% AEP	50% AEP
30th March 2014	-	-	-	50% AEP	50% AEP	50% AEP

Rainfall intensities at the gauges were assessed for the 30 minute, 1 hour and 2 hour storm burst durations and compared to frequencies derived from Australian Rainfall and Runoff 1987 (Reference 4) in Table 5. These durations were selected for analysis based upon experience that these types of storm durations would be critical (i.e. produce the highest flood levels) for the size of the Little Creek catchment. It can be seen that for most of the historical floods on record, the rainfalls were only equivalent to approximately the 50% AEP rainfall. The main exception is the October 1987 storm, when rainfalls equivalent to the 10% to 5% AEP were recorded.

This finding should not be interpreted conclusively that these floods were only 50% AEP floods for the catchment. It is likely that the pluviometers missed the most intense local rainfalls, and there were probably parts of the Little Creek catchment which received higher rainfall intensities.

Comparison of significant rainfall events and design rainfall intensities from AR&R 1987 are shown on Figure 7. These charts show a larger range of durations than the summary provided in the tables above.

2.7.5. Radar Rainfall Data

WMAwater also obtained snapshots of the weather radar from the Bureau of Meteorology website, for the March 2014 storm event. The images provide a qualitative understanding of the spatial distribution of rainfall intensity at 6-minute intervals. It is important to note that there are limitations for the use of this information to derive a detailed map of spatial rainfall depths, since as the Bureau of Meteorology identifies:

> *The radar reflectivity is strongly dependent on the diameter of raindrops in the cloud not the amount of rain drops and therefore rainfall rates. Tropical maritime rainfall consists of very many moderate sized raindrops so that the reflectivity is much less than for similar rainfall rates in continental area rain clouds. The latter rain clouds typically consist of very large raindrops but much less in number.*

However the images can provide a broad indication of where the heaviest falls were located, and whether the available rainfall gauges were likely to capture these falls. This information was therefore used to inform sensitivity analysis for calibration of the March 2014 storm (see Section 6.4).

2.8. Design Rainfall Data

New design rainfall depths were released by the BoM in July 2013. Whilst it is expected that the new design rainfall depths will undergo minor revisions as they are independently verified, it is unlikely they will change substantially within the Sydney metropolitan area. The 2013 design rainfall estimates require other information from the revision of ARR including temporal patterns, aerial reduction factors, losses and base flows before they can be used in design flood estimation. Until the completion of the ARR revision project, current advice is that design rainfall intensities and techniques from ARR 1987 should continue to be used (Reference 4).

The design rainfall intensity-frequency-duration (IFD) data were obtained from the BoM online design rainfall tool and provided on Table 6.

Table 6: Rainfall IFD data at the centre of the Little Creek catchment (ARR 1987)

DURATION	Design Rainfall Intensity (mm/hr) From Bureau of Meteorology						Extrapolated using ARR87 methodology	
	50% AEP	20% AEP	10 % AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
5 minutes	97.3	126	144	166	196	219	243	275
6 minutes	91	118	134	156	184	205		
10 minutes	74.4	96.6	110	127	150	167	186	210
20 minutes	54.1	70.1	79.6	91.9	108	121	134	152
30 minutes	43.9	56.8	64.5	74.5	87.7	97.8	109	123
1 hour	29.7	38.5	43.7	50.5	59.4	66.2	74	83
2 hours	19.7	25.4	28.8	33.2	39.1	43.5	48.4	55
3 hours	15.4	19.8	22.4	25.9	30.4	33.8	37.7	42.5
6 hours	10	12.9	14.6	16.8	19.7	21.9	24.5	27.6
12 hours	6.53	8.44	9.55	11	12.9	14.4	15.9	18.0
24 hours	4.17	5.49	6.27	7.29	8.65	9.69	10.9	12.4
48 hours	2.57	3.48	4.04	4.77	5.73	6.49	7.29	8.37
72 hours	1.89	2.6	3.04	3.61	4.37	4.97	5.63	6.50

The Probable Maximum Precipitation (PMP) estimates were derived according to BoM guidelines, namely the Generalised Short Duration Method (Reference 5) and are summarised in Table 7.

Table 7: PMP Design Rainfalls

Duration	Design Rainfall Depth (mm)
15 minutes	162
30 minutes	235
45 minutes	297
1 hour	345
1.5 hours	445
2 hours	520
3 hours	630
6 hours	840

2.9. Previous Studies

2.9.1. City Engineer's Report, Little Creek, Colyton (Reference 1)

In the immediate aftermath of the October 1987 storm, Council undertook an internal investigation into the flood issues observed in the catchment.

The report identified that most development in the catchment was undertaken in the 1950s and 1960s, and that upstream of the Great Western Highway most of the creek system had been "piped" as part of the subdivision works, whereas downstream of the highway an open creek channel mostly remained during that period (1950s and 1960s). By 1987, significant additional sections of the creek had been piped, such as the reach from Oxley Park Public School to Thompson Avenue, and this pipe was designed to have a "one in five year capacity" or 20% AEP. The report identified that after construction of the pipe, significant obstructions to flow such as fences and other structures had been constructed along the overland flow path above the pipe. Negotiations were underway at the time of the report to obtain an easement through the affected properties in this area.

This report estimated the October 1987 storm to be equivalent to a 2% AEP event, however no analysis of rainfall was provided so this estimate cannot be reviewed. Estimates by WMAwater and SKM (Reference 2) indicate it was more likely to be in the order of a 5% AEP event (see following section).

The report identified a number of potential flood mitigation works such as detention basins which were investigated in further detail as part of subsequent flood modelling and design reports by SKM and Council. The study also established preliminary hydrologic modelling in ILSAX which was also further refined as part of the later studies.

In addition to the proposed detention basins, a key finding of the report was that a formal overland flow path would be required through private property for the reach between Oxley Park Public School and Hobart Street. In 2016, an overland flow path exists at the downstream end between Brisbane Street and Hobart Street, but not for the upstream section between the School and Brisbane Street.

2.9.2. Drainage Investigation Little Creek, Colyton (Reference 2)

Sinclair Knight & Partners was commissioned by the PCC to assess the capacity of the existing drainage system in Colyton. The aims of the study were to:

1. Assess the capacity of the existing drainage system;
2. Assess flooding patterns and their causes;
3. Review Council's proposals for flood mitigation;
4. Identify and analyse any additional alternative flood mitigation measures and
5. Evaluate alternative flood mitigation solutions and provide financial analyses.

The hydrologic/hydraulic model established for the study was ILSAX (Reference 4). The 20% AEP, 1% AEP and October 1987 event were assessed and it was determined that the capacity of all pipes was exceeded in the 1% AEP event. The ILSAX model estimated the pipe capacity in Hobart Street (12.4 m^3/s), Kenny Avenue (11.9 m^3/s), Thompson Avenue (15.9 m^3/s), Brisbane Street Park (15.9 m^3/s), Brisbane Street (12.2 m^3/s) and Canberra Street (10.6 m^3/s).

The results suggested that ponding and overland flow would occur in the vicinity of the Canberra Street and Sydney Street intersection, downstream to Thompson Avenue and through the park areas to the railway embankment at Hobart Street.

For the 24th October 1987 event, this study estimated that the maximum 30 minute burst at Erskine Park (Hewitts Gauge) was equivalent to 5% AEP, which is consistent with the estimate by WMAwater based on the Tenbee Gauge (10% to 5% AEP). Unfortunately the Hewitts Gauge data for this event was not able to be obtained for this report[1], but this indicates it showed a similar rainfall intensity to the Tenbee gauge. The rainfall at the Tenbee was close to 5% AEP intensity for very short durations (6 minutes to 12 minutes), but for 30 minutes was exactly halfway between the 10% and 5% AEP design intensities.

The study estimated flows, but hydraulic modelling to determine water levels, depths and velocities across the catchment was not undertaken.

The study recommended construction of mitigation measures referred to as "Alternative 5." This option involved construction of detention basins in the vicinity of the Great Western Highway and in Colyton High School and modification of some pipe sizes to improve the basin performance. The recommended scheme was similar to what was constructed (as of 2016), except it included an additional detention basin immediately upstream of the Great Western Highway, in the reserve area downstream of Bennet Road. This area may warrant further investigation for additional flood mitigation works.

[1] PCC provided a report documenting the data availability from the Hewitts Creek gauge. It was noted that data is only available from June 1991 to November 1998, and there are no reports of flood producing storms in that period. Therefore the raw pluviometer data for this period was not pursued further.

2.9.3. Council Design Summary – Oxley Park Basins (Reference 3)

Council undertook a modelling study in October 1991, primarily to review and refine design of flood mitigation works arising from the recommendations in Reference 2 such as:

- the Colyton High School detention basin;
- the basin in Council land north of the Great Western Highway, near Whitcroft Place; and
- potential pipe upgrades and further basin construction.

The study estimated inflows and outflows to the basins to inform the design, and included significant refinements to the model in the vicinity of the works based on additional survey. The main objective of the report was to document additional work undertaken as part of the basin design, so it did not substantially add to knowledge of the overall catchment flood behaviour further to References 1 and 2.

2.9.4. Penrith Overland Flow Flood "Overview Study" (Reference 6)

Cardno was commissioned by PCC to investigate the overland flow flood behaviour throughout the Penrith LGA for the 5% AEP, 1% AEP and PMF events. The main aim of this study was prioritisation of the sub-catchments for further investigation based on the severity of flood affectation.

The study included major hydraulic structures such as culverts and bridges where available from PCC. The dimensions of some structures were assumed by scaling the photographs where information was not available. It was suggested that the structure data would not be suitable for use in a detailed catchment flood study. No pit or pipe data were included in the study.

The hydraulic model used in this study was SOBEK including both 1D and 2D elements. Design rainfall time-series were applied directly on the model grid as input, which resulted in the generation of overland flow. The design rainfall for the Penrith area were derived from ARR87.

The results showed that Little Creek was ranked within the top 10% of flood affected sub-catchments based on the combined criteria and economic damage.

2.9.5. Updated South Creek Flood Study (Reference 7)

This study was prepared by Worley Parsons on behalf of PCC, acting in association with Liverpool, Blacktown and Fairfield City Councils. The flood study covers the South Creek catchment extending from Bringelly Road in the south to the Blacktown/Richmond Road Bridge crossing in the north. The total study area is about 240 km^2 and lies within the Hawkesbury, Penrith, Blacktown, Liverpool and Fairfield LGAs.

In this study, RAFTS was adopted as the hydrologic model, MIKE-11 and HEC-2 were adopted for 1D hydraulic modelling and RMA-2 was used for 2D hydraulic modelling. The modelling was calibrated to historical events and was used to simulate the full range of design floods, including the PMF.

A total of 480 cross-sections from a 1990/1991 study covering South Creek and its tributaries were included in the study. Flood marks for the 1986 and 1988 floods were obtained, and considered to be representative of the 1% AEP flood levels in Ropes Creek and South Creek, respectively (see Table 8). These marks have been included as they are of relevance for backwater flooding in Little Creek and for setting tailwater levels for historical flood validation modelling.

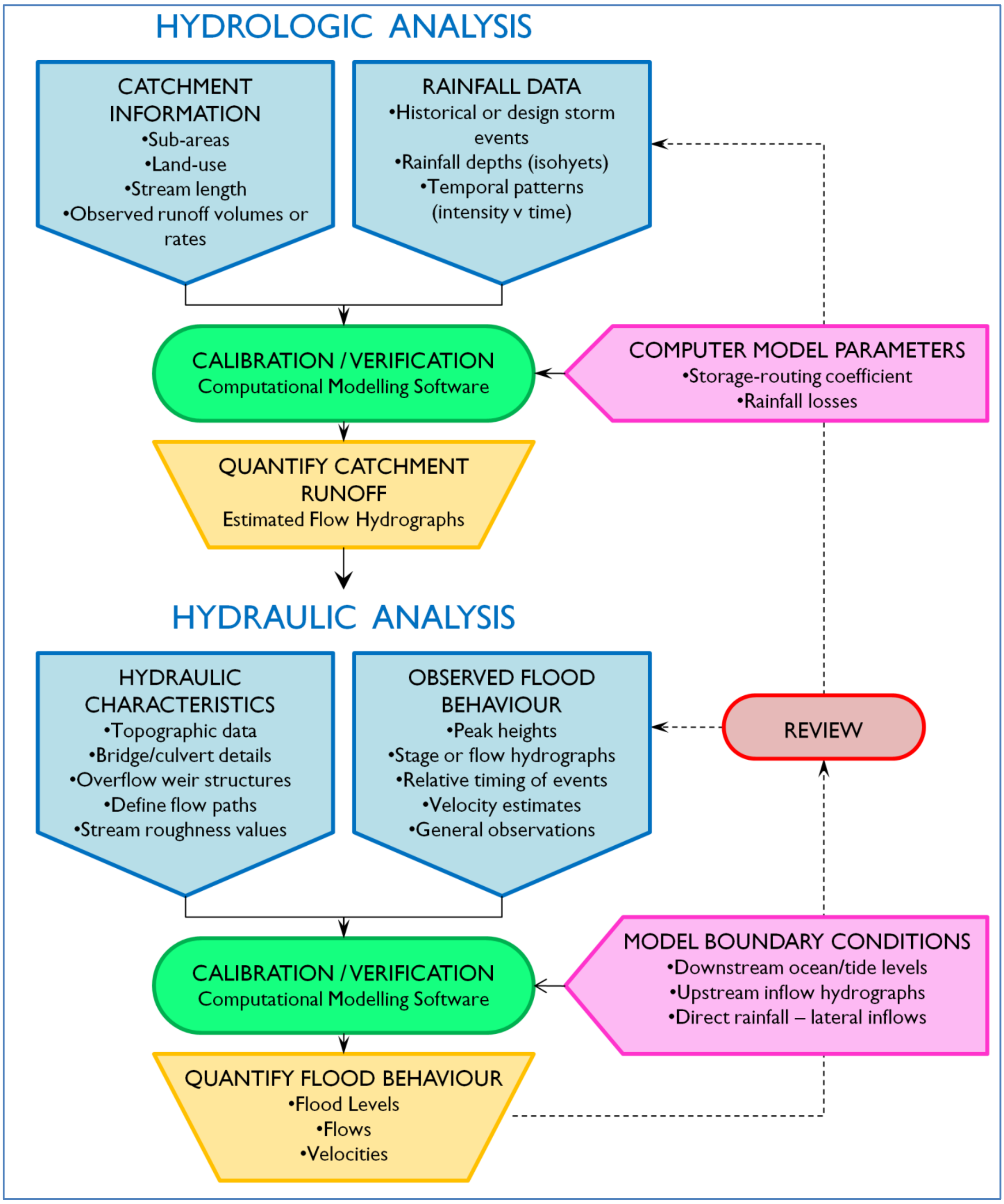
Table 8 1986 and 1988 Historic Flood Marks along South Creek near Little Creek

Location	Recorded 1986 Flood Level (mAHD)	Recorded 1988 Flood Level (mAHD)
F4 Freeway Crossing		26.94
Saddington Street, St Clair	24.36	25.24
Great Western Highway	24.43	24.73
Main Western Railway		22.89
Dunheved Road, Dunheved	21.14	21.25

3. MODELLING OVERVIEW

The urbanised nature of the study area with its mix of pervious and impervious surfaces, and existing piped and overland flow drainage systems, creates a complex hydrologic and hydraulic flow regime. A diagrammatic representation of the Flood Study process to address the issues is shown in Diagram 5.

Diagram 5: Flood Study Process



For this study, the estimation of flood behaviour in a catchment was undertaken as a two-stage process, consisting of:

1. hydrologic modelling to convert rainfall estimates to overland flow runoff; and
2. hydraulic modelling to estimate overland flow distributions, flood levels and velocities.

The broad approach adopted for this study was to use DRAINS, a widely utilised and well-regarded hydrologic model for urban catchments, to conceptually model the rainfall concentration phase (including runoff from roof drainage systems, gutters, etc.). Design rainfall depths and patterns specified in AR&R (Reference 4) were input into the model and the runoff hydrographs were then used in a hydraulic model to estimate flood depths, velocities and hazard in the study area. Hydraulic modelling will be carried out using TUFLOW on a fixed 1.5 m grid.

The sub-catchments in the hydrologic model were kept small (on average approximately 0.8 ha) such that the overland flow behaviour for the study area was generally defined by the hydraulic model. This approach allows the concentration phase of the runoff to be modelled in a conceptual manner, since the scale of these concentration processes is too small to be modelled adequately by the hydraulic model (which has a grid cell size of 1.5 m). The concentration phase refers to runoff from roof/gutter/downpipe systems, intra-lot drainage, and other small scale flow paths in the most upstream parts of the catchment. WMAwater have previously used this method for similar overland flow catchment flood studies, and verified its suitability through comparisons with other commonly used hydrologic approaches.

The DRAINS hydrologic model software (Reference 8) was used to create the flow boundary conditions for input into a 2D unsteady flow (estimates the full storm hydrograph rather than just the peak flow as occurs with a steady state hydraulic model) hydraulic model using the TUFLOW software (Reference 9).

There are no stream-flow records in the catchment, so the use of a flood frequency approach for the estimation of design floods or calibration of the hydrologic model (independently from the hydraulic model) was not possible.

3.1. Hydrologic Model

DRAINS (Reference 8) is a hydrologic/hydraulic model that can simulate the full storm hydrograph and is capable of describing the flow behaviour of a catchment and pipe system for real storm events, as well as statistically based design storms. It is designed for analysing urban or partly urban catchments where artificial drainage elements have been installed.

The DRAINS model is broadly characterised by the following features:

- the hydrological component is based on the theory applied in the ILSAX model which has seen wide usage and acceptance in Australia;
- its application of the hydraulic grade line method for hydraulic analysis throughout the drainage system; and

- the graphical display of network connections and results.

The use of DRAINS within this study was limited to some minor upstream catchment routing and development of hydrological inputs into the TUFLOW hydraulic model. The hydraulic components of DRAINS were not used, such as the routing of flows between sub-catchments ("total" flows), and modelling of the pit/pipe network.

DRAINS generates a full hydrograph of surface flows arriving at each pit. Runoff hydrographs for each sub-catchment area are calculated using the time area method.

3.2. Hydraulic Model

The availability of high quality aerial survey data means that the study area is suitable for two-dimensional (2D) hydraulic modelling. Various 2D software packages are available and the TUFLOW package was adopted for this project.

The TUFLOW (Reference 9) modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water flow equations in two dimensions. The TUFLOW software is produced by BMT WBM and has been widely used for a range of similar projects. The model is capable of dynamically simulating complex overland flow regimes. It is especially applicable to the hydraulic analysis of flooding in urban areas which is typically characterised by short duration events and a combination of supercritical and subcritical flow behaviour, and interactions between overland flow and a sub-surface drainage network.

In addition to 2D modelling of overland flows, TUFLOW can model drainage elements (pipes) as 1D elements as well as modelling creeks or open channels in 1D if required. The 1D and 2D components of the model can be dynamically linked during the simulation.

In TUFLOW the ground topography is represented as a uniformly-spaced grid with a ground elevation and a Manning's "n" roughness value assigned to each grid cell. The grid cell size is determined as a balance between the model result definition required and the computer run time (which is largely determined by the total number of grid cells, and the number of "wet" cells). A cell size of 1.5 m by 1.5 m was found to provide an appropriate balance for this study.

3.3. Calibration to Historical Events

When available, historical flood data can be used to calibrate the models and increases confidence in the estimates. The calibration process involves modifying the initial model parameter values to produce modelled results that concur with observed data. If records are available from multiple storms, validation can be undertaken to ensure that the calibration model parameter values are acceptable in other storm events with no additional alteration of values. Recorded rainfall and stream-flow data are required for calibration of the hydrologic model, while historic records of flood levels, velocities and inundation extents can be used for the calibration of hydraulic model parameters. In the absence of such data, model verification using limited historical data is the only option and a detailed sensitivity analysis of the different model input parameters constitutes current best practice.

For this project, a reasonable dataset of flood levels was available for the March 2014 storm. Limited flood height data is available from community descriptions of events in the late 1980s, but it is not clear in which event precisely these observations occurred. Therefore, the March 2014 storm was used as the primary calibration event, and the October 1987 and April 1988 events were modelled for validation purposes.

3.4. Design Flood Modelling

Design flood modelling was undertaken using the methodology outlined in Australian Rainfall and Runoff (Reference 4):

- design outflows for localised sub-catchments were obtained from the DRAINS hydrologic model, using standard design storms, and applied as inflows to the TUFLOW model;
- the TUFLOW model was used to estimate and map the flood behaviour for a range of flood events;
- sensitivity analysis was undertaken to assess the relative effect of changing various TUFLOW and DRAINS modelling parameters and catchment assumptions.

3.5. Mapping and Interpretation of Results

Results of the design flood modelling are presented in Appendix B, on Figure B1 to Figure B58, and discussed further in Section 7.4.

The SES relies on results from the design flood modelling for their emergency response planning activities. These outputs are provided in Appendix C, Figure C1 to Figure C17.

Part of the design flood modelling methodology is determining which design storm temporal pattern to use. Different catchments have different response times that depend on the area, shape and slope of the catchment, among other things. The critical duration analysis is discussed in Section 7.3, and results are mapping in Appendix D, Figure D1 to Figure D4.

Sensitivity analysis is also an important part of the modelling process, as it can indicate which of the model inputs have the most influence on the results. Sensitivity analysis was used to identify key inputs so that attention could be focussed on those aspects of the model development. Mapping of sensitivity results in provided in Appendix D, Figure D5 to Figure D31, with further discussion provided in Section 8.

Interpretation and discussion of the results for planning and flood risk mitigation purposes is provided in Sections 9 and 10.

4. HYDROLOGIC MODEL SETUP

4.1. Sub-catchment Definition

The total catchment represented by the DRAINS model is 4.6 km^2. This area was represented by a total of 542 sub-catchments giving an average sub-catchment size of approximately 0.8 ha (approximately the size of a football field). This relatively small sub-catchment delineation ensures that where significant overland flow paths exist that these are accounted for and able to be appropriately incorporated into hydraulic routing in the TUFLOW model. The sub-catchment layout is shown in Figure 8.

The method for the PMP (Reference 5) requires that variable rainfall be applied over different parts of the catchment. The ellipses defining the different sub-catchment rainfall depths are shown on Figure D32.

4.2. Impervious Surface Area

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete surfaces occurs significantly faster than from vegetated surfaces. This results in a faster concentration of flow within the downstream area of the catchment, and increased peak flow in some situations. It is therefore necessary to estimate the proportion of the catchment area that is covered by such surfaces.

DRAINS categorises these surface areas as either:

- paved areas (impervious areas directly connected to the drainage system);
- supplementary areas (impervious areas not directly connected to the drainage system, instead connected to the drainage system via the pervious areas); and
- grassed areas (pervious areas).

Within all sub-catchments, a uniform 5% was adopted as a supplementary area across the catchment. The remaining 95% was attributed to impervious (paved) and pervious surface areas, as estimated for each individual sub-catchment. The percentage of pervious surface was estimated by determining the proportion of the sub-catchment area covered by different surface types, and the estimated impervious percentage of each material category as summarised in Table 9. Sensitivity analysis was conducted on these assumptions.

Table 9: Impervious Percentage per Land-use

Material	Impervious Percentage
Low density residential	50%
Thick Vegetation	0
Vegetated Waterways	0
Paved Areas	100%
Railway	100%

The proportion of material within a sub-catchment was determined based upon 2014 aerial photography provided by PCC.

4.3. Sub-catchment Slope

The slope of each sub-catchment was determined using an automated algorithm based on the following characteristics of each area:

- Minimum and maximum elevations based on LiDAR
- The ratio of the catchment area to its perimeter, used to estimate an indicative length

The typical slopes used for each sub-catchment were in the range of 2% to 6%, with an average of 3.5%. The minimum sub-catchment slope was 0.6% and the maximum was almost 14% (for catchments which included sections of railway line cut with very steep embankments).

4.4. Rainfall Losses

Methods for modelling the proportion of rainfall that is "lost" to infiltration are outlined in AR&R (Reference 4). The methods are of varying degrees of complexity, with the more complex options only suitable if sufficient data are available. The method most typically used for design flood estimation is to apply an initial and continuing loss to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

Rainfall losses from a paved or impervious area are considered to consist of only an initial loss (an amount sufficient to wet the pavement and fill minor surface depressions). Losses from grassed areas are comprised of an initial loss and a continuing loss. The continuing loss is calculated from an infiltration equation curve incorporated into the model and is based on the selected representative soil type and antecedent moisture condition.

The adopted loss parameters are summarised in Table 10. These are generally consistent with the parameters adopted flood studies in similar catchments within the Sydney metropolitan area.

Table 10: Adopted rainfall loss parameters

RAINFALL LOSSES	
Paved Area Depression Storage (Initial Loss)	1.0 mm
Grassed Area Depression Storage (Initial Loss)	5.0 mm
SOIL TYPE	3
Slow infiltration rates (may have layers that impede downward movement of water). This parameter, in conjunction with the AMC, determines the continuing loss	
ANTECEDENT MOISTURE CONDITONS (AMC)	3
Description	Rather wet
Total Rainfall in 5 Days Preceding the Storm	12.5 to 25 mm

5. HYDRAULIC MODEL SETUP

5.1. TUFLOW

The TUFLOW model uses a regularly spaced computational grid, with a cell size of 1.5 m by 1.5 m. This resolution was adopted as it produced an appropriate balance between providing sufficient detail for roads and overland flow paths, while still resulting in workable computational run-times. The model grid was established by sampling from a triangulation of filtered ground points from the 2011 LiDAR dataset.

The total area included in the 2D model was 4.6 km^2 and the extents of the TUFLOW model are shown in Figure 9.

5.2. Boundary Locations

5.2.1. Inflows

For local sub-catchments within the TUFLOW model domain, local runoff hydrographs were extracted from the DRAINS model (see Section 4). These were applied to the receiving area of the sub-catchments within the 2D domain of the hydraulic model. These inflow locations (shown on Figure 9) typically correspond with gutter lines and inlet pits on the roadway, or specific drainage reserved. These features have typically been constructed to receive intra-lot drainage and sheet runoff flows in upstream catchment areas.

5.2.2. Downstream Boundary

Several downstream boundary locations were implemented in the model. On the western side of the model, most boundaries were located along the bank of South Creek, and a constant tailwater level was adopted, as shown in Figure 9. This includes the end of the open channel section of Little Creek, which runs from Kurrajong Road to South Creek near the corner of Lee Holm Road and Maxim Place. The tailwater level at this main downstream boundary depends on the water level in South Creek, and thus different tailwater assumptions were adopted for different events.

For the calibration events, the tailwater levels were set as follows:

- October 1987 – 18 mAHD (no significant coincident flooding in South Creek);
- April 1988 – 21.25 mAHD (based on the level at Dunheved Rd from Reference 7); and
- October 2014 – 18.0 mAHD (no significant coincident flooding in South Creek).

The tailwater assumptions for design events are summarised in Table 11 below. These assumptions were specified by Penrith City Council for consistency with other studies in the LGA.

Table 11: Design Event Coincident South Creek Tailwater Assumptions

Little Creek Design Rainfall	South Creek Coincident Tailwater Assumption
50% AEP	Minor coincident flows
20% AEP	Minor coincident flows
10% AEP	Minor coincident flows
5% AEP	5% AEP South Creek
2% AEP	5% AEP South Creek
1% AEP	5% AEP South Creek
0.5% AEP	5% AEP South Creek
0.2% AEP	5% AEP South Creek
PMF	5% AEP South Creek

At the local outflow boundaries to the west of Kalang Avenue and the corner of Forrester Street and Griffiths Street, a "normal flow" boundary was adopted, representing a relatively shallow depth of flow. This assumption reflects that these boundaries are on-grade, and unlikely to be affected by significant tailwater influences.

The downstream invert levels of pipes were adopted as the tailwater level for the pipes flowing out of the 2D model (i.e. assumed inlet control at these pipes), but not flowing directly into South Creek. This assumption reflects that these pipes have a steep enough grade to generate inlet control conditions, and the outlets are unlikely to be affected by significant tailwater influences.

5.3. Roughness Parameter

The hydraulic efficiency of the flow paths within the TUFLOW model is represented in part by the hydraulic roughness or friction factor formulated as Manning's "n" values. This factor describes the net influence of bed roughness and incorporates the effects of vegetation and other features which may affect the hydraulic performance of the particular flow path.

The Manning's "n" values adopted for the study area, including flow paths (overland, pipe and in-channel), are shown in Table 12. These values have been adopted based on site inspection and past experience in similar floodplain environments. The values are consistent with typical values given in Chow, 1959 (Reference 10) and Henderson, 1966 (Reference 11).

The spatial variation in Mannings 'n' is shown in Figure 10. Note that two different values of Mannings 'n' were adopted for different sections of the open channel waterway, based on the observed vegetation density for these sections.

Table 12: Manning's "n" values adopted in TUFLOW

Surface	Manning's "n" Adopted
General residential or light vegetation (e.g. grass)	0.04
Thick Vegetation	0.07
Waterways (Light Vegetation)	0.05
Waterways (Heavy Vegetation)	0.1
Concrete Channel	0.02
Paved Area	0.02
Railway corridor	0.04

5.4. Hydraulic Structures

5.4.1. Buildings

Buildings and other significant features likely to act as flow obstructions were incorporated into the model network based on building footprints, defined using aerial photography. These types of features were modelled as impermeable obstructions to flow. Thus there is no assumed flood storage capacity within the building. Building delineation will be based on aerial photographs, and validated for key overland flow areas by site inspection and use of Google "Streetview" photographs.

Buildings were "blocked out" from the 2D model grid, in line with research undertaken for the AR&R revision (Reference 13). The research project found that "Numerical model trials showed *that on the basis of the available data sets, the best performing method when representing buildings in a numerical model was to either remove the computational points under the building footprint completely from the solution or to increase the elevation of the building footprint to be above the maximum expected flood height.*" The project also found that "*Analysis of flood volumes on the floodplain has shown that in a floodplain with flows passing through the floodplain, achieving peak levels due to peak flow rate rather than peak stored volume, the influence of the flow volume stored inside buildings is not significant to the presented flood levels in the prototype floodplain.*"

5.4.2. Fencing and Obstructions

Smaller localised obstructions, such as fences, can be explicitly represented in TUFLOW in a number of ways including as an impermeable obstruction, a percentage blockage or as an energy loss. These obstructions may also be approximated generally by increasing Manning's roughness for land use areas such as residential, to represent the typical type of fencing used in such areas.

The principles for modelling of fences were as follows:

- The majority of fences in the catchment were not modelled, since they can be difficult to identify and generally will not affect flow behaviour significantly in areas of shallow flow.

- Major flow paths were identified from preliminary design modelling, and fences with the potential to affect flow behaviour were modelled.
- Fences were modelled using 2d_lfcsh layers in TUFLOW, allowing for implementation of blockage and energy losses.
- Sensitivity analysis was conducted to determine whether the assumed losses and blockage factors had a significant influence on flow behaviour. The effect of the fences was generally found to be minimal.

5.4.3. Bridges and Culverts

Detailed schematisation of key hydraulic structures was included in the hydraulic model, at the locations indicated on Figure 9. Culverts and bridges were generally modelled as 1D elements within the 2D domain, based on the scale of the structure and the key flow characteristics in comparison with the 2D cell size of 1.5 m. The decision on whether to model a structure in 1D or 2D was based primarily on the findings of Reference 14.

The modelling parameter values for the culverts and bridges were based on the geometrical properties of the structures, which were obtained from detailed survey (Appendix E), photographs taken during site inspections, and previous experience modelling similar structures.

One of the more complicated of these structures is the large inlet structure at Hobart St, immediately upstream of the railway line (see Appendix E, Structure 03 for survey drawings). Views from within the structure are shown on Photo 2 to Photo 5 below. This structure comprises a large steel inlet grate above a flow mixing chamber. The flow mixing chamber has two compartments, each of which have two 1200 mm diameter pipes flowing into them from upstream. These two compartments flow through two parallel conversion chambers which narrow and increase in height as they approach the brick arch culvert which flows under the railway. The initial chamber dimensions are 2400 mm wide by 1200 mm high (two chambers), and they change shape to adjust the flow area to more closely match the brick arch culvert dimension. This brick arch culvert has dimensions very similar to a 2400 mm diameter pipe.

Photo 2: Brick arch culvert under railway

Photo 3: Hobart St conversion chamber No. 1

Photo 4: Hobart Street mixing chamber and inlet grate

Photo 5: Hobart St conversion chamber No. 2

This complex structure shape is not supported by the standard solution methods available in TUFLOW. Interpretation of the flow conditions was required to determine an appropriate method to schematise the structure in the model. It was determined that under the flood conditions being investigated, when the inlet structure is inundated and subject to pressurised flow, the key hydraulic control from this structure is the inlet capacity of the brick arch culvert which represents the limiting hydraulic conveyance. Therefore, the details of the irregular shaped chambers between the inlet and the brick arch culvert were not explicitly included in the model.

5.4.4. Sub-surface Drainage Network

The stormwater drainage network was modelled in TUFLOW as a 1D network dynamically linked to the 2D overland flow domain. This stormwater network includes conduits such as pipes and box culverts, and stormwater pits, including inlet pits and junction manholes. The schematisation of the stormwater network was undertaken using the detail “pit and pipe” survey collected for the project, as well as information from Council records such as Works-as-Executed plans to validate the information where appropriate. This validation was particularly necessary for some of the larger trunk drainage pipes, which in many instances pass for long distances through private property, and where junction pits are no longer accessible due to development over time.

Details of the 1D solution scheme for the pit and pipe network are provided in the TUFLOW user manual (Reference 9). For modelling of inlet pits the “R” pit channel type was utilised, which requires a width and height dimension for the inlet in the vertical plane. The width dimension represents the effective length inlet exposed to the flow, and the vertical dimension reflects the depth of flow where the inlet becomes submerged, and the flow regime transitions from the weir equation to the orifice equation. For lintel inlets, the width was based on the length of the opening. For inlet grates, the width was based on the perimeter of the grate. For combined lintel and grate inlets, the inlet width was the combination of the lintel and grate edge lengths, minus the portion of the grate adjacent to the lintel (to avoid double counting). This method applies to both sag and on-grade pits.

Figure 11A shows the location and dimensions of drainage lines within the study catchment that have been included in the TUFLOW model. Figure 11B shows the number of pipes at locations where there are multiple parallel pipes.

5.4.5. Road Kerbs and Gutters

LIDAR typically does not have sufficient resolution to adequately define the kerb/gutter system within roadways. The density of the aerial survey points is in the order of one per square metre, and the kerb/gutter feature is generally of a smaller scale than this, so the LIDAR does not pick up a continuous line of low points defining the drainage line along the edge of the kerb.

To deal with this issue, Reference 15 provides the following guidance:

> *Stamping a preferred flow path into a model grid/mesh (at the location of the physical kerb/gutter system) may produce more realistic model results, particularly with respect to smaller flood events that are of similar magnitude to the design capacity of the kerb and gutter. Stamping of the kerb/gutter alignment begins by digitising the kerb and gutter interval in a GIS environment. This interval is then used to select the model grid/mesh elements that it overlays in such a way that a connected flow path is selected (i.e. element linkage is orthogonal). These selected elements may then be lowered relative to the remaining grid/mesh.*

The road gutter network plays a key role for overland flow in the Little Creek catchment. Preliminary modelling indicated that a significant portion of the catchment flows were within the roadways, which often traversed perpendicular to the land slope, and the flow depths were in the order of the depth of a typical kerb/gutter system (i.e. 0.1 m to 0.15 m), but using the raw LIDAR data resulted in multiple breakouts of flow over the kerb lines that did not appear to be realistic.

It was determined that in order to resolve these systems effectively, the gutters would be stamped into the mesh using the method described above. The method used was to digitise breaklines along the gutter lines, and reduce the ground levels along those model cells by 0.1 m, creating a continuous flow path in the model.

5.5. Blockage Assumptions

5.5.1. Background

In order to determine design flood behaviour the likelihood and consequences of blockage needs to be considered. Guidance on the application of blockage can be found in AR&R Revision Project 11: Blockage of Hydraulic Structures, 2014 (Reference 12).

Blockage of hydraulic structures can occur with the transportation of a number of materials by flood waters. This includes vegetation, garbage bins, building materials and cars, the latter of which has been seen in the June 2007 Newcastle and August 1998 Wollongong Floods (Photo 6 and Photo 7).

Photo 6: Cars in a culvert inlet – Newcastle (Reference 12)

Photo 7: Urban debris in Wollongong (Reference 12)

The potential quantity and type of debris reaching a structure from a contributing source area depends on several factors. AR&R guidelines suggest adopted design blockage factors are based upon consideration of:

- the availability of debris;
- the ability for it to mobilise, and
- the ability for it to be transported to the structure.

The availability of debris is dependent on factors such as the potential for soil erosion, local geology, the source area, the amount and type of vegetative cover, the degree of urbanisation, land clearing and preceding wind and rainfall. However, the type of materials that can be mobilised can vary greatly between catchments and individual flood events.

Observations of debris conveyed in streams strongly suggest a correlation between event magnitude and debris potential at a site. Rarer events produce deeper and faster floodwater able to transport large quantities and larger sizes of debris, smaller events may not be able to transport larger blockage material at all. Debris potential is adjusted as required for greater or lesser probabilities to establish the *most likely* and *severe* blockage levels for that event.

Table 13: Most Likely Blockage Levels – BDES (Table 6 in Reference 12)

Control Dimension	At-Site Debris Potential		
	High	Medium	Low
$W < L_{10}$	100%	50%	25%
$W \geq L_{10} \leq 3 \times L_{10}$	20%	10%	0%
$W > 3 \times L_{10}$	10%	0%	0%

Notes: W refers to the opening diameter / width
L_{10} refers to the 10% percentile length of debris that could arrive at the site

The likelihood of blockage at a particular structure depends on whether or not debris is able to bridge across the structure inlet or become trapped within the structure. Research into culvert blockage in Wollongong showed a correlation with blockage and opening width. The *most likely* blockage to occur at a structure is determined by considering the potential quantity and type of debris and the structure opening size as in Table 13.

A severe blockage level is proposed where the consequences are very high and Reference 12 suggests a *severe* blockage of twice the *most likely* blockage criteria. At structures where the consequence of blockage is very low, a 0% blockage is suggested.

5.5.2. Blockage for Calibration Events

Photo 8: Outlet of trunk drainage system at Kurrajong Road

It was identified as part of the collection of detail survey that the outlet of the trunk drain at Kurrajong Road was significantly blocked by an accumulation of silt, in the order of 0.3 m to 0.5 m deep. The silt was observed to be relatively cohesive and compacted, with established vegetation, suggesting it has been in a similar condition for some time, and was probably blocked for the March 2014 storm used for model calibration. Photo 8 shows the condition of the outlet in late 2015:

The outlet of a nearby pipe near the corner of Kurrajong Road and Plasser Crescent was also observed to be almost completely blocked (see Photo 9). A level of blockage commensurate with the observed siltation and surveyed geometry of the outlets was therefore assumed for the model calibration against the March 2014 storm.

Photo 9: Outlet of pipe at Kurrajong Road near Plasser Crescent

5.5.3. Adopted Design Blockage

Table 14: Adopted Pit Blockage Factors

Location	Inlet Type	Percentage Blocked
Sag	Side Entry	20%
Sag	Grated	50%
Sag	Combination	Side inlet capacity only, Grate assumed completely blocked
Sag	Letterbox	50%
On Grade	Side Entry	20%
On Grade	Grated	50%
On Grade	Combination	10%

The adopted pit inlet blockage factors were based on Penrith City Council design guidelines for inlet pits, and are summarised in Table 14. Inlet blockages were implemented by reducing the effective inlet width in proportion with the relevant blockage factor. Refer to Section 5.4.4 for details of the pit inlet modelling methodology.

For all bridges and culverts with inlet headwalls (i.e. not pipes which have stormwater pits at the upstream end), a methodology in accordance with the ARR Blockage Guidelines (Reference 12) was incorporated into design event modelling. The Reference 12 methodology considers blockage due to various sources and takes into account the:

- Debris Type and Dimensions - Whether floating, non-floating or urban debris present in the source area and its size;
- Debris Availability – The volume of debris available in the source area;
- Debris Mobility – The ease with which available debris can be moved into the stream;

- Debris Transportability – The ease with which the mobilised debris is transported once it enters the stream; and
- Structure Interaction – The resulting interaction between the transported debris and the bridge or culvert structure.

Debris characteristics were considered to be similar for each of the culverts assessed (i.e. uniform across the catchment), due to both similar catchment characteristics and similar culvert dimensions. A summary of the adopted design blockage levels is provided below in Table 15.

Table 15: Adopted Bridge/Culvert Design Blockage Factors

Event AEP	Selected Design Blockage
AEP > 5% *(frequent)*	25%
5% AEP to 0.5% AEP	50%
AEP < 0.5% *(rare)*	75%

Photo 10: Eastern branch pipe outlet at Kurrajong Road after Council maintenance

During the course of the study, Council undertook maintenance to clear the pipe outlets at Kurrajong Road (see Photo 10).

PCC indicated that for design events, the Kurrajong Road outlets should be modelled as clear for the design flood modelling, based on a commitment by Council to continue maintenance in this area and keep the outlets clear of accumulated silt.

Sensitivity analysis was undertaken on these assumptions, and it was determined that blockage was not a critical issue for design flood levels in the catchment (see Section 8).

5.6. Model Mass Balance Checks

The cumulative mass error from the model is an indication of whether the numerical implementation of the shallow water equations is resulting in artificial creation or destruction of water. A high mass balance error can indicate unreliable modelling results, since the model would not be accurately representing the amount of stormwater runoff in the catchment.

The cumulative error was less than 0.2% for all design events modelled. This is a very low error and is reflective of excellent model "health" and schematisation. This is an advantage of coupled hydrologic/hydraulic modelling approaches over "rainfall on grid" models, which tend to produce mass balance errors from wetting and drying of cells and shallow flow in steep terrain areas.

Table 16: Historic Flood Observations – Depth Estimates

Design Event	Cumulative Mass Error
PMF	-0.12%
0.2% AEP	-0.06%
0.5% AEP	-0.06%
1% AEP	-0.09%
2% AEP	-0.07%
5% AEP	-0.06%
10% AEP	0.00%
20% AEP	-0.01%
50% AEP	-0.16%

6. MODEL CALIBRATION

6.1. Overview

Prior to use for defining design flood behaviour it is important that the performance of the overall modelling system be substantiated. Calibration involves modifying the initial model parameter values to produce modelled results that concur with observed data. Validation is undertaken to ensure that the calibration model parameter values are acceptable in other storm events with no additional alteration of values. Ideally the modelling system should be calibrated and validated to multiple events, but this requires adequate historical flood observations and sufficient pluviometer rainfall data.

Typically in urban areas such information is lacking. Issues which may prevent a thorough calibration of hydrologic and hydraulic models are:

- There is only a limited amount of historical flood information available for the study area. For example, in the Sydney metropolitan area there are only a few water level recorders in urban catchments similar to that of the study area; and
- Rainfall records for past floods are limited and there is a lack of temporal information describing historical rainfall patterns (pluviometers) within the catchment.

In the event that a calibration and validation of the models is not possible or limited in scope, it is best practice to undertake a verification of the models and a detailed sensitivity analysis. This was the approach adopted for this study.

6.2. Summary of Calibration Event Data

The choice of calibration or verification events for flood modelling depends on a combination of the severity of the flood event and the quality of the available data. As is the case with most urban studies there was limited quantitative data available either in the form of flood marks or surveyed flood levels for the study area. There was qualitative information provided by residents through the community consultation process with regard to their properties being flood affected and whether they had been flooded in their yard, garage or above floor level. In some cases this was used to estimate a depth of flooding or an extent of the flow path.

The majority of available flood observations were from the March 2014 storm. March 2014 was a recent event that was identified through the community consultation as having caused significant flooding problems in the study area. Additional storms from October 1987 and April 1988 were also modelled for validation purposes, as there were some anecdotal reports of flooding issues in the late 1980s from long-term residents, and these were known to be relatively intense rainfall events over the catchment. However most residents could not recall which event specifically had caused the flood issues.

A description of each recorded flood level obtained through the community consultation process is given in Table 17 (observations where a depth could be estimated, or there was above floor flooding) and Table 18 (observations indicating a rough extent of the flow path).

Table 17: Historic Flood Observations – Depth Estimates

ID	Resident Description	Event
L032	1980s – 45cm Above ground; 2014 – 10cm above ground	late 1980s and 2014
L080	12.1 cm (5 inches) above ground	2014
L093	20 cm above ground	2014
L106	10 cm at garage	2014
L107	30 cm at front of the house	2014
L108	50 cm at neighbours' garage.	2014
L142	24 cm above ground	2014
L163_1	30 cm at garage	2014
L167	10 cm outside house	2014
L091	Flood in garage	late 1980s
L001	Up to first step on front porch	late 1980s
L053	10 cm on side of building	Unknown
L066	Flood above floor	2015
L101	Flood above floor	Unknown
L115	Flood to garage and shed	Unknown
L121	Flood above floor	late 1980s
L147	Flood above floor	Unknown

Table 18: Historic Flood Observations – Extent Estimates

ID	Resident Description	Event
L087	5 cm near easement	2014
L090	A few centimetres deep	2014
L163_2	lapping at door	2014
L005	2 cm above floor level	Unknown
L047	Flooded down driveway	Unknown
L135	Flood to driveway	2013 and 2015
L149	Flood in yard	Unknown
L151	Food in the park	Unknown
L155	Flood in yard	Unknown
L173	Flood in yard	Unknown
L194	Flood in halfway of driveway	Unknown

The calibration and validation process was limited by incompleteness of the available rainfall data records. The nearest pluviometers were both outside the catchment, and it is likely they do not accurately reflect the rainfall falling within the catchment. Three daily rainfall stations were located closer to or within the catchment, but each of these had relatively short operating periods which did not include the major storms of interest (1986 to 1988, and 2014). Given this level of uncertainty, it was considered inappropriate to deviate significantly from typical modelling parameters used in similar urban catchments from the Sydney metropolitan area.

Comparisons of the rainfall data for the model calibration/validation events with design rainfall intensities from AR&R 1987 (Reference 4) are shown in Figure 7, and summarised in Table 19.

Table 19: Data Available for Calibration Storm Events

Storm Events	Approximate AEP of recorded rainfall	Rainfall (mm)	Duration (h)	Pluviometer Stations
October 1987	10% to 5%	104.5	5	Tenbee (568074)
April 1988	Smaller than 50% AEP	41.5	3	Tenbee (568074)
March 2014	50% to 20%	39	3	St Marys STP (567087)

6.3. Hydrologic Model Validation

A basic hydrologic model validation was undertaken by checking the specific yield for all sub-catchments in comparison with results from similar areas in other studies. The specific yield is calculated by dividing the area in hectare by the maximum flow generated from the sub-catchment. The average specific yield for the 1% AEP event for all sub-catchments was 0.38 ($m^3/s/ha$), which is reasonable compared to results from other urban Sydney catchments, once differences in design rainfalls are taken into account.

6.4. Hydraulic Model Calibration / Validation

6.4.1. March 2014 Calibration

Calibration of the hydraulic model was undertaken by comparing the data (Table 17) collected from the community consultation (Section 1.4) to modelled historical events.

Several sensitivity scenarios were evaluated in the model calibration process, the parameters used in each scenario are shown in Table 20. As a result of this process the soil type, antecedent moisture condition and impervious fraction were modified to understand the sensitivity of the results to reduced initial loss and continuous loss.

Sensitivity to rainfall was also investigated. The St Marys STP pluviometer is located outside the Little Creek catchment to the north. Based on analysis of the radar intensity information for the event (see Section 2.7.5), it seems likely that the average catchment rainfall depth for March

2014 event was higher than recorded at the pluviometer, by about 27% (see Figure 12). Therefore, a scenario was investigated for the calibration where the rainfall was increased by this amount, to determine whether it would produce a closer match to observed flood levels.

Figure 13 shows rainfall hyetographs adopted for the calibration events, and Figure 14 shows the cumulative rainfall depths. Figure 15 shows TUFLOW model results for the March 2014 calibration event

Table 20: Calibration Parameters

Calibration Scenario	Description	Soil Type	Antecedent Moisture Condition	Catchment Average Rainfall Depth (mm)	Sub-catchment average impervious fraction
A	**Base**	3	3	39	30%
B	**Wet and Low Infiltration**	4	4	39	30%
C	**Increase Rainfall by 27 %**	3	3	49.5	30%
D	**Wet and Low Infiltration, Increase Rainfall by 27%**	4	4	49.5	30%
E	**Increased residential area imperviousness (50%)**	3	3	39	60%
F	**Increased residential area imperviousness (50%), and increase in rainfall (27%)**	3	3	49.5	60%

The results shown in Table 21 indicate that model replicates flooding for the historical event in the same locations (Figure 5) that residents have reported flooding in the past.

Table 21: Comparison of Modelled and Observed Peak Flood Depths – March 2014

ID	Recorded Depth (m)	Difference (m) Modelled Depth minus Recorded Depth					
		Calibration Scenario					
		A	B	C	D	E	F
L032	0.10	-0.06	-0.02	-0.02	0.00	-0.03	-0.01
L080	0.13	Not Flooded	-0.01	-0.02	0.04	-0.05	0.01
L093	0.20	-0.09	-0.03	-0.05	0.08	-0.08	0.03
L106	0.10	Not Flooded	-0.04	-0.04	-0.01	-0.05	-0.02
L107	0.30	-0.19	-0.02	-0.02	0.02	-0.04	0.01
L108	0.50	-0.26	-0.08	-0.08	-0.04	-0.10	-0.05
L142	0.24	-0.12	-0.04	-0.04	0.01	-0.06	0.00
L163_1	0.30	-0.15	-0.01	-0.01	0.06	-0.05	0.03
L167	0.10	-0.02	0.05	0.03	0.12	0.00	0.09

It can be observed from Table 21 that the Scenario A typically underestimated the peak flood depths in March 2014 event, while the scenarios B and C with either increased rainfall or reduced infiltration losses resulted in a better match, although slight with a slight bias towards the low side. A combination of both higher rainfall and lower losses (Scenario D) produced flood depths that were typically higher than the observations. Scenario E slightly underestimated the peak depths, while Scenario F with a combination of increased impervious fraction and increased rainfall had the best match. For Scenario F, differences to the food marks were typically less than 0.05 m, with a mix of higher and lower depths compared to observations.

It is not considered appropriate to adopt soil type 4 or an antecedent moisture condition of 4, as this would reflect extreme soil conditions not typically found in the area. Most flood studies in urban areas of Sydney would use a soil type of 2 or 3, and similar intermediate values for antecedent moisture conditions.

The adopted calibration parameters produced a good match with the observed flood behaviour for the historical events analysed. At flood marks where a depth estimate was available, the model typically matched within 0.05 m of the observed flood depth. This is considered to be within the accuracy of the depth estimates, which were provided by community members who observed the flooding. For locations where community members provided a description of the extent of flooding, the modelling typically showed shallow flooding at those locations, with the extent accurate to within one or two grid cells in the horizontal direction.

The final calibration scenario adopted was Scenario F, which combined increased rainfall (based on radar information) and increased impervious surface fraction for residential areas of 50%. This impervious fraction was therefore be adopted for the design event modelling.

6.4.2. October 1987 and April 1988 Validation

While residents provided some descriptions of flooding that occurred in the late 1980s, there was generally little confidence about the year it occurred, or the exact depth. There were several residents who indicated that flooding above floor level occurred in this period, suggesting at least one storm caused relatively severe flooding. As a validation exercise therefore, the October 1987 and April 1988 events were modelled to check that the model indicated some flooding in the observed “hot-spot” areas.

The October 1987 event was chosen because it had some of the most intense observed rainfalls on record near the catchment. The April 1988 event was chosen because it was a large storm that caused widespread flooding regionally, including flooding in South Creek, and local rainfalls were also notable, if only in the order of a 50% AEP event.

Figure 13 shows rainfall hyetographs adopted for the calibration events, and Figure 14 shows the cumulative rainfall depths.

Limited topographical data is available from the period, however it is known that the major detention basins in the upper catchment were not constructed at that time. Historical aerial photographs also provide some indication of the level of development in the catchment. For the purposes of the validation modelling, the detention basins were removed from the DEM by setting ground levels throughout these areas to a similar level as adjacent roadways, and removing any constructed embankments. Mannings 'n' roughness values were also adjusted in some areas based on the differences between the historical aerial photographs and the most recent photographs of the catchment.

Maps of the modelled depths for the two validation events are shown on Figure 16 and Figure 17. As anticipated, the modelled flood depths were greater for the October 1987 event given the more intense local rainfall. The peak depth map for this event shows a good match between areas of significant flooding and those locations where flood issues were reported by the community.

7. DESIGN EVENT MODELLING

7.1. Overview

Design flood levels in the catchment are a combination of flooding from rainfall over the local catchment, as well as elevated tailwater levels from flooding in South Creek. This study primarily is concerned with the Little Creek flood mechanism. South Creek flood extents from Reference 7 should also be considered as part of any floodplain management and flood-related planning activity for the catchment.

7.2. Downstream Boundary Levels – South Creek

In addition to runoff from the catchment, downstream areas can also be influenced by high water levels near South Creek. Consideration must therefore also be given to accounting for the joint probability of coincident flooding from the South Creek

A full joint probability analysis to consider the interaction of these two mechanisms is beyond the scope of the present study. It is accepted practice to estimate design flood levels in these situations using a 'peak envelope' approach that adopts the highest of the predicted levels from the two mechanisms.

Design flood levels for South Creek flooding are provided in Reference 7 and the adopted boundary conditions are summarised in Table 22.

Table 22: Design Flood Levels (Reference 7) in South Creek (downstream)

Design Event	South Creek Peak Level (mAHD)
50% AEP	19.5 to 20.0
20% AEP	19.5 to 20.0
10% AEP	19.5 to 20.0
5% AEP	22.1 to 22.6
2% AEP	22.1 to 22.6
1% AEP	22.1 to 22.6
0.5 % AEP	22.1 to 22.6
PMF	22.1 to 22.6

Sensitivity of the results to lower tailwater levels was tested, and it was found that there was a negligible influence on Little Creek flood levels for South Creek level ranging from 18.0 m to 20.0 m at the confluence.

7.3. Critical Duration - Overland Flooding

To determine the critical storm duration for various parts of the catchment (i.e. produce the highest flood level), modelling of the 1% AEP event was undertaken for a range of design storm durations from 30 minutes to 12 hours, using temporal patterns from AR&R (Reference 4). An envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area (see Figure D1 for 1% AEP results).

It was found that the 2 and 9 hour design storms were critical at different areas of the catchment, but the peak levels produced were very close (typically less than 0.05 m difference). It was determined to use the 2-hour design event as the critical duration for the catchment. Refer to Figure D2 for differences between the peak envelope and the 2-hour storm, for the 1% AEP event.

In the PMF it was found that 2 hour design storms were critical at most areas of the catchment. Figure D3 shows the duration that produced the maximum flood depth at each location for the PMF, and Figure D4 shows the difference between that maximum level and the level using the 2-hour PMF storm.

It was found that longer storm durations produced slightly higher flood levels in the basins between Great Western Highway and Adelaide Street for the 1% AEP and PMF event due to the greater total rainfall depth, but this did not translate into significantly higher peak flood levels elsewhere in the catchment.

The 2 hour duration was therefore adopted for all design flood events.

7.4. Design Flood Results

The results from this study are mapped in Appendix B as follows:

- Peak flood extent in Figure B1 to Figure B9
- Peak flood depths in Figure B10 to Figure B18
- Peak flood levels in Figure B19 to Figure B27
- Peak flood velocities in Figure B28 to Figure B36
- Provisional hydraulic hazard in Figure B37 to Figure B45
- Provisional hydraulic categorisation in Figure B46 to Figure B48

The design flood results were filtered to be consistent with other overland flow studies undertaken in the Penrith LGA, using the following criteria:

- Depths less than 0.15 m were removed from the result maps;
- Where this resulted in patches of isolated flooding, these patches were removed if they were less than 100 m^2 in area.

The results have been provided to PCC in digital format compatible Council's Geographic Information Systems (GIS).

7.4.1. Overview of Flood Behaviour

The railway embankment of the Western Railway Line forms a major hydraulic feature of the Little Creek catchment, and flow behaviour is distinctly different upstream and downstream of the railway corridor.

Upstream of the railway line, there are no reaches of open creek channel. The natural creek alignment has been completely replaced by pipe, and there are several sections where there is no formal overland flow path or easement above the trunk drainage line. This main drainage line runs in a north-westerly direction, originating from the local drainage networks around Kent Place, Bennett Road and Carpenter Street, across the Great Western Highway through two large detention basins at Oxley Park, through Oxley Park Public School, then across Adelaide Street, Sydney Street, Canberra Street, Brisbane Street, Thompson Street, Kenny Avenue, to the railway line at Hobart Street. There are several locations along this main drainage line where overland flow occurs through private development, when runoff exceeds the capacity of the stormwater network. The pipe capacity assessment indicates that the majority of the stormwater network upstream of the Great Western Highway has less than 50% AEP capacity.

There is a detention basin within the playing fields of Colyton High School. This basin is "off-line" from the main drainage network, which is primarily directed to Carpenter Street and Bennett Road. There is an offtake pipe from Carpenter Street near Dorothy Crescent, which directs some flow into the basin, while the main pipes flow west along Carpenter Street to Bennett Road. There is a large grate at this offtake point (see Photo 11), and during larger storm events flow surcharges out of the grate and along a concrete channel within an overland flow easement at number 77 Carpenter Street.

Photo 11: Overland flow path at 77 Carpenter Street (surcharge grate in foreground)

This concrete channel then enters a covered channel that runs beneath 4A and 3C Sykes Place (see Photo 12) before discharging into the detention basin in the high school. This flow behaviour was confirmed during the site inspection by interviews with witnesses from nearby businesses who have seen the surcharging occur.

Photo 12:Covered open channel inlet under 4A Sykes Street from 77 Carpenter Street

In areas away from the main drainage line, overland flow is generally along the road network. The majority of east-west streets in the upper catchment grade down towards the main drainage line, and both minor and major drainage system flows are conveyed directly to the trunk. As the catchment is relatively narrow either side of the trunk alignment, there are relatively few major “tributary” overland flow paths in the upper catchment.

The most notable exception is a drainage line in the western part of the catchment which originates at a low point in Adams Crescent and runs north across Morris Street, Jacka Street, the Great Western Highway, Cutler Avenue, Edmondson Avenue, Adelaide Street, Canberra Street, and Brisbane Street before joining the main trunk drain via a pipe along the rear of properties on Brisbane and Thompson Streets. At the Great Western Highway, a significant portion of the overland flow along this flow path is likely to be redirected eastwards due to the median strip and cross-fall of the road, sending water towards the main drainage alignment near Whitcroft Place (see Section 10.2.7 for more discussion).

Flow is significantly attenuated by the railway line embankment at the Hobart Street sag point (see Section 10.2.1 for detailed discussion). Downstream of the railway line, where Little Creek remains primarily an open channel, there is relatively little overbank flooding even in the 1% AEP event. The remaining channel through this reach generally has sufficient capacity to convey the flow that discharges through the railway line culverts, as well as local runoff from the lower catchment areas.

The primary area of flood affectation in the lower catchment are parts of the industrial estate along Lee Holm Road and Christie Street. These roads and some of the adjoining sites are relatively low-lying with flat grades. Furthermore, this area is likely to be affected by South Creek flooding for the 5% AEP and larger events on that system.

7.4.2. Summary of Results

Peak flood levels, depths and flows at key locations within the catchment are summarised in below. A tabulated summary of peak flood depth and level results at key locations as shown in Figure 18 are detailed in Table 23, Table 24 and Table 25. The locations coincide with the reporting locations used for the sensitivity analysis discussed in the next section.

A detailed comparison of the results from this study with results from the overview study (Reference 6) was not undertaken. The primary objective of the overview study was "*to establish priorities for detailed overland flow studies,*" and the modelling undertaken omitted details of the catchment drainage such as the pit/pipe stormwater network, road and rail culverts, and detail of urban features such as buildings, road gutters, fences etc. Those features, which have been included in the current study, have a significant influence on flow behaviour in some locations.

For example at Hobart Street, the overview study estimated a peak 5% AEP flood level of 38.6 mAHD, a peak 1% AEP flood level of 39.1 mAHD and a peak PMF level of 40.1 mAHD, compared to 35.8 mAHD, 36.2 mAHD and 40.9 mAHD respectively from the current study. Details of how the rail embankment and cross-drainage culverts were modelled in Reference 6 are not available, however this example illustrates that levels from the current study should be used in preference for design and floodplain management purposes, given the additional detail of the stormwater network, higher 2D model resolution, and other detail included in the modelling. It would not be productive to go into a more detailed comparison of the design flood levels from each study, since estimating accurate design flood levels was not a main objective of the overview study.

Table 23: Peak Flood Levels (mAHD) at Key Locations

ID	Location	AEP								PMF
		50%	20%	10%	5%	2%	1%	0.5%	0.2%	
H001	Patricia_St	48.1	48.3	48.3	48.4	48.4	48.4	48.5	48.5	48.6
H002	Bentley_Rd	49.9	50.1	50.1	50.1	50.1	50.1	50.1	50.1	50.2
H003	Carpenter_St	46.3	46.4	46.5	46.5	46.5	46.6	46.6	46.6	46.8
H004	Colyton_School	45.5	45.8	45.9	46.0	46.2	46.3	46.3	46.3	46.6
H005	Kent_Pl	45.1	45.2	45.3	45.3	45.4	45.4	45.5	45.5	46.0
H006	Shane_St	44.4	44.5	44.6	44.6	44.7	44.7	44.7	44.8	45.3
H007	Bennet_Rd	44.2	44.4	44.5	44.5	44.6	44.6	44.7	44.7	45.2
H008	GreatWestern_Hwy	43.8	43.9	44.0	44.2	44.3	44.4	44.4	44.4	44.8
H009	Ridge_Park_S	41.9	42.0	42.0	42.0	42.1	42.1	42.2	42.3	42.8
H010	Ridge_Park_N	40.6	41.2	41.5	41.8	41.9	41.9	41.9	42.0	42.4
H011	Adelaide_St	39.6	39.8	39.9	40.0	40.1	40.2	40.3	40.4	41.3
H012	Canberra_St	38.9	38.9	39.0	39.0	39.0	39.0	39.1	39.2	40.9
H013	Sydney_St	38.9	38.9	39.0	39.0	39.0	39.0	39.1	39.2	41.0
H014	Brisbane_St	37.7	37.7	37.8	37.8	37.9	37.9	38.0	38.1	40.9
H015	Thompson_Ave	36.4	36.5	36.6	36.6	36.7	36.7	36.8	37.2	40.9
H016	Kenny_Ave	36.1	36.2	36.2	36.2	36.3	36.3	36.7	37.2	40.9
H017	Hobart_St	34.9	35.2	35.5	35.8	36.0	36.2	36.7	37.2	40.9
H018	Plasser_Cres	34.6	34.7	34.8	34.8	34.9	35.0	35.0	35.0	36.5
H019	Kurrajong_Rd	32.4	32.6	32.6	32.7	32.7	32.8	32.8	32.8	33.4
H020	Glossop_St	31.7	31.7	31.7	31.7	31.8	31.8	31.8	31.9	32.4
H021	Forrester_Rd	27.9	28.0	28.0	28.1	28.2	28.4	28.5	28.6	29.5
H022	Maxim_Pl	24.4	24.6	24.6	24.7	24.7	24.8	24.8	24.8	25.1
H023	Structure_8	23.2	23.3	23.4	23.5	23.5	23.5	23.5	23.6	23.8
H024	Structure_9	22.3	22.3	22.3	22.6	22.6	22.6	22.6	22.7	22.8
H025	LeeHolm_Rd	22.7	22.8	22.9	23.0	23.0	23.0	23.0	23.1	23.6
H026	Christie_St	21.3	21.3	21.3	22.1	22.1	22.1	22.1	22.2	22.5
H027	LittleCreek_US	32.0	32.3	32.4	32.5	32.6	32.6	32.7	32.7	33.3
H028	LittleCreek_DS	20.0	20.0	20.0	22.6	22.6	22.6	22.6	22.6	22.6
H029	Camira_St	32.9	32.9	33.0	33.1	33.1	33.2	33.2	33.2	33.3
H030	Morris_St	54.7	54.7	54.8	54.8	54.8	54.8	54.9	54.9	55.0
H031	Jacka_St	53.1	53.1	53.2	53.2	53.3	53.3	53.3	53.3	53.4
H032	Edmondson_Ave	44.8	44.9	44.9	44.9	44.9	44.9	44.9	44.9	45.0
H033	Adelaide_St_W	41.5	41.5	41.5	41.5	41.6	41.6	41.6	41.6	41.7
H034	Carpenter_St_W	47.0	47.1	47.1	47.1	47.2	47.2	47.2	47.2	47.3
H035	Muscio_St	45.7	45.8	45.9	45.9	45.9	45.9	46.0	46.0	46.1
H036	Ball_St	44.2	44.2	44.2	44.2	44.3	44.4	44.4	44.5	44.9
H037	GreatWestern_Hwy_W	52.2	52.3	52.5	52.6	52.6	52.7	52.7	52.7	52.8
H038	Christie_St_N	26.8	26.9	26.9	26.9	26.9	27.0	27.0	27.0	27.3
H039	Glossop_St_W	29.1	29.3	29.3	29.5	29.5	29.6	29.7	29.8	31.0
H040	Forrester_Rd_W	25.8	26.0	26.1	26.2	26.4	26.5	26.6	26.8	28.0

Table 24: Peak Flood Depths (m) at Key Locations

ID	Location	AEP								PMF
		50%	20%	10%	5%	2%	1%	0.5%	0.2%	
H001	Patricia_St	0.3	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.8
H002	Bentley_Rd	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5
H003	Carpenter_St	0.2	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.6
H004	Colyton_School	0.3	0.5	0.6	0.8	1.0	1.0	1.1	1.1	1.4
H005	Kent_Pl	0.2	0.3	0.4	0.4	0.4	0.5	0.5	0.6	1.1
H006	Shane_St	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6	1.1
H007	Bennet_Rd	0.0	0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.9
H008	GreatWestern_Hwy	0.2	0.4	0.5	0.7	0.8	0.8	0.9	0.9	1.2
H009	Ridge_Park_S	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.6	2.2
H010	Ridge_Park_N	0.9	1.5	1.8	2.1	2.1	2.2	2.2	2.3	2.7
H011	Adelaide_St	0.2	0.4	0.5	0.6	0.7	0.8	0.8	1.0	1.9
H012	Canberra_St	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.8	2.5
H013	Sydney_St	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.7	2.5
H014	Brisbane_St	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.8	3.6
H015	Thompson_Ave	0.5	0.6	0.6	0.7	0.7	0.8	0.8	1.3	5.0
H016	Kenny_Ave	0.5	0.6	0.6	0.7	0.7	0.8	1.1	1.6	5.3
H017	Hobart_St	0.5	0.7	1.0	1.3	1.6	1.8	2.3	2.8	6.5
H018	Plasser_Cres	0.1	0.2	0.3	0.3	0.4	0.5	0.5	0.5	2.0
H019	Kurrajong_Rd	0.3	0.5	0.5	0.6	0.6	0.7	0.7	0.7	1.3
H020	Glossop_St	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.7
H021	Forrester_Rd	0.1	0.2	0.3	0.3	0.5	0.6	0.7	0.9	1.8
H022	Maxim_Pl	0.2	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.9
H023	Structure_8	0.0	0.1	0.2	0.3	0.3	0.3	0.4	0.4	0.6
H024	Structure_9	0.0	0.0	0.0	0.4	0.4	0.4	0.4	0.4	0.6
H025	LeeHolm_Rd	0.5	0.6	0.7	0.8	0.8	0.9	0.9	0.9	1.4
H026	Christie_St	0.0	0.0	0.0	0.8	0.8	0.8	0.8	0.9	1.2
H027	LittleCreek_US	0.8	1.1	1.2	1.3	1.4	1.4	1.5	1.5	2.1
H028	LittleCreek_DS	2.3	2.3	2.3	4.9	4.9	4.9	4.9	4.9	4.9
H029	Camira_St	0.2	0.3	0.3	0.4	0.5	0.5	0.5	0.6	0.7
H030	Morris_St	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.4
H031	Jacka_St	0.0	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.4
H032	Edmondson_Ave	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
H033	Adelaide_St_W	0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.4
H034	Carpenter_St_W	0.0	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.3
H035	Muscio_St	0.0	0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.4
H036	Ball_St	0.0	0.0	0.0	0.0	0.1	0.2	0.2	0.3	0.7
H037	GreatWestern_Hwy_W	0.0	0.1	0.3	0.4	0.5	0.5	0.5	0.6	0.7
H038	Christie_St_N	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.4	0.6
H039	Glossop_St_W	1.4	1.6	1.7	1.8	1.9	2.0	2.1	2.2	3.4
H040	Forrester_Rd_W	1.5	1.7	1.9	1.9	2.1	2.2	2.3	2.5	3.7

Table 25: Peak Flows (m^3/s) at Key Locations

ID	Location	Type	AEP								PMF
			50%	20%	10%	5%	2%	1%	0.5%	0.2%	
Q001	Q_Bentley_Rd	Overland	0.0	0.4	0.7	1.1	1.4	1.7	2.0	2.5	5.2
		Pipe	-	-	-	-	-	-	-	-	-
Q002	Q_Carpenter_St	Overland	1.1	1.8	2.2	2.9	3.5	3.9	4.3	5.0	10.2
		Pipe	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.7	0.9
Q003	Q_Kent_Pl	Overland	0.3	0.7	0.9	1.2	1.6	2.0	2.5	3.1	7.9
		Pipe	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2
Q004	Q_Shane_St	Overland	0.3	0.5	0.6	0.7	0.8	0.9	1.0	1.3	12.2
		Pipe	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
Q005	Q_Bennet_Rd	Overland	0.4	1.7	2.9	4.6	6.3	7.8	9.5	11.9	34.8
		Pipe	3.7	3.9	3.9	3.9	3.9	3.9	3.9	3.9	4.0
Q006	Q_GreatWestern_Hwy	Overland	0.1	0.4	0.5	0.8	2.5	5.4	8.5	13.0	69.1
		Pipe	5.2	6.2	6.9	7.6	8.1	8.4	8.7	9.0	9.9
Q007	Q_Ridge_Park_S	Overland	4.4	6.8	8.0	9.4	11.9	14.8	18.1	23.2	89.5
		Pipe	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.5
Q008	Q_Adelaide_St	Overland	0.4	0.8	1.0	2.9	5.5	8.0	11.0	17.4	94.9
		Pipe	3.0	4.0	4.4	5.7	6.1	6.1	6.0	5.4	6.1
Q009	Q_Canberra_St	Overland	1.7	3.6	4.8	6.2	7.4	8.5	11.5	17.9	118.4
		Pipe	2.9	3.9	4.5	6.1	6.5	6.6	6.7	6.3	6.8
Q010	Q_Brisbane_St	Overland	1.6	4.1	5.5	8.0	9.0	10.6	12.3	19.1	109.7
		Pipe	3.6	4.6	5.2	6.7	6.8	6.7	6.7	6.5	6.5
Q011	Q_Thompson_Ave	Overland	2.3	4.3	5.9	7.7	9.7	11.8	13.9	19.5	102.5
		Pipe	4.5	5.6	6.4	7.8	7.5	7.4	7.2	7.1	7.4
Q012	Q_Kenny_Ave	Overland	1.9	3.9	5.7	7.7	10.0	12.4	14.8	17.8	104.7
		Pipe	5.0	6.1	6.7	8.1	7.7	7.3	7.3	7.3	7.5
Q013	Q_Hobart_St	Overland	0.9	3.3	4.5	5.1	5.9	6.6	7.7	8.9	100.4
		Pipe	6.8	7.4	7.8	8.5	8.7	9.0	9.1	9.3	11.5
Q014	Q_Plasser_Cres	Overland	0.1	0.4	0.5	0.6	0.6	0.7	0.9	1.2	56.0
		Pipe	7.6	10.4	11.9	13.5	14.6	15.5	16.7	17.9	24.8
Q015	Q_Kurrajong_Rd	Overland	0.4	0.5	0.6	0.8	1.0	1.3	2.0	2.9	66.6
		Pipe	8.8	12.0	13.5	15.2	16.3	17.1	17.4	18.3	23.8
Q016	Q_LittleCreek_US	Overland	8.8	12.5	14.3	16.0	17.5	18.9	20.2	22.2	82.8
		Pipe	-	-	-	-	-	-	-	-	-
Q017	Q_Glossop_St	Overland	0.0	0.0	0.0	7.6	9.9	11.9	14.0	21.6	95.2
		Pipe	9.3	13.1	14.6	10.9	11.0	11.0	11.0	5.3	5.4
Q018	Q_Forrester_Rd	Overland	3.2	7.6	10.8	16.0	20.1	23.4	26.6	32.6	88.8
		Pipe	10.3	11.5	11.6	9.6	9.6	9.6	9.6	5.2	4.3
Q019	Q_Maxim_Pl	Overland	2.7	9.2	11.8	17.7	21.5	25.1	28.5	36.4	104.0
		Pipe	9.0	9.2	9.3	6.8	6.8	6.8	6.8	3.6	3.6
Q020	Q_Structure_8	Overland	0.0	3.2	7.0	17.0	20.6	23.7	26.5	34.1	84.3

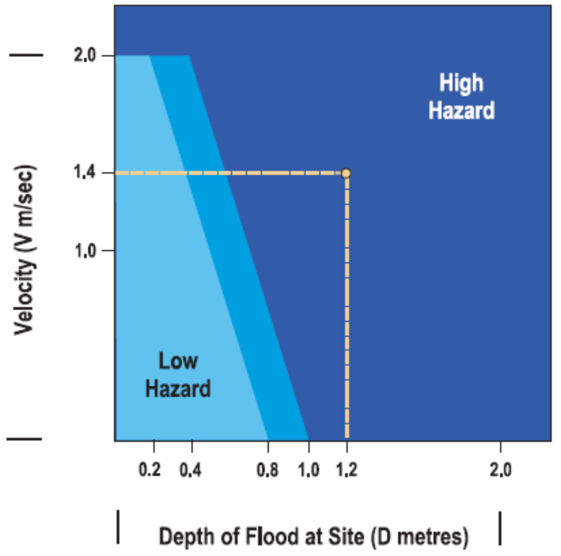
ID	Location	Type	AEP								PMF
			50%	20%	10%	5%	2%	1%	0.5%	0.2%	
		Pipe	10.2	11.9	12.0	6.8	7.0	7.1	7.1	3.5	3.5
Q021	Q_LittleCreek_DS	Overland	10.4	15.7	19.4	24.3	27.9	30.9	33.8	37.9	93.2
		Pipe	-	-	-	-	-	-	-	-	-
Q022	Q_Christie_St	Overland	0.0	0.1	0.2	0.3	0.4	0.5	0.7	1.0	4.6
		Pipe	0.1	0.2	0.2	0.0	0.0	0.0	0.0	0.0	0.0
Q023	Q_LeeHolm_Rd	Overland	1.2	2.3	2.8	3.5	4.0	4.4	4.6	4.9	7.2
		Pipe	0.6	0.5	0.6	0.3	0.3	0.3	0.3	0.3	0.4
Q024	Q_Morris_St	Overland	0.1	0.4	0.6	0.8	1.0	1.2	1.4	1.8	3.6
		Pipe	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.6
Q025	Q_Jacka_St	Overland	0.0	0.1	0.4	0.7	1.0	1.3	1.6	2.0	4.9
		Pipe	0.8	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.3
Q026	Q_Edmondson_Ave	Overland	1.3	2.0	2.2	2.5	2.8	3.0	3.3	3.7	8.6
		Pipe	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Q027	Q_Adelaide_St_W	Overland	1.2	2.2	2.6	3.2	3.8	4.3	4.9	5.6	12.6
		Pipe	1.4	1.6	1.6	1.7	1.7	1.7	1.7	1.7	1.8
Q028	Q_Carpenter_St_W	Overland	1.3	2.7	3.5	4.5	5.4	6.4	7.5	8.9	19.6
		Pipe	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Q029	Q_Muscio_St	Overland	0.9	2.5	3.5	4.6	5.7	6.7	7.8	9.4	20.5
		Pipe	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
Q030	Q_Ball_St	Overland	0.6	1.2	1.5	1.8	2.2	2.5	2.9	3.6	11.3
		Pipe	1.0	1.2	1.4	1.4	1.4	1.5	1.5	1.5	1.6
Q031	Q_Patricia_St	Overland	0.0	0.5	0.9	1.5	2.0	2.4	2.9	3.6	8.9
		Pipe	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
Q032	Q_GreatWestern_Hwy_W	Overland	0.0	0.0	0.0	0.1	0.4	0.7	1.0	1.5	4.3
		Pipe	1.1	1.3	1.3	1.4	1.4	1.4	1.4	1.4	1.4
Q033	Q_Christie_St_N	Overland	0.9	1.2	1.3	1.7	2.0	2.4	2.8	3.4	11.0
		Pipe	0.1	0.2	0.2	0.0	0.0	0.0	0.0	0.0	0.0
Q034	Q_Glossop_St_W	Overland	9.8	13.6	15.2	17.9	20.3	22.2	24.6	28.0	98.6
		Pipe	-	-	-	-	-	-	-	-	-
Q035	Q_Forrester_Rd_W	Overland	13.7	18.0	20.4	22.4	26.0	29.3	32.4	37.6	104.3
		Pipe	-	-	-	-	-	-	-	-	-

Refer to Figure 18 for locations reported in the above tables.

7.4.3. Provisional Flood Hazard Categorisation

Provisional hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual (Reference 18), the relevant section of which is shown in Diagram 6. For the purposes of this report, the transition zone presented in Diagram 6 (L2) was considered to be high hazard.

Diagram 6: Provisional Hydraulic Hazard Categories



Provisional hazard categories for the range of design flood events modelled are displayed on Figure B37 to Figure B45.

7.4.4. Provisional Hydraulic Categorisation

Principles for determining hydraulic categories, namely floodway, flood storage and flood fringe, are described in the Floodplain Development Manual (Reference 18). However, there is no widely accepted technical definition of hydraulic categorisation that would be suitable for all catchments, and different approaches are used by different consultants and authorities, based on the specific features of the study catchment in question.

For this study, hydraulic categories were defined by the following criteria, which correspond in part with the criteria proposed by Howells *et al*, 2003 (Reference 19):

- <u>Floodway</u> is defined as areas where:
  - the peak value of velocity multiplied by depth (V x D) > 0.25 m^2/s **AND** peak velocity > 0.25 m/s, **OR**
  - peak velocity > 1.0 m/s

The remainder of the floodplain is either Flood Storage or Flood Fringe,

- <u>Flood Storage</u> comprises areas outside the floodway where peak depth > 0.2 m; and
- <u>Flood Fringe</u> comprises areas outside the Floodway where peak depth < 0.2 m.

Provisional hydraulic categories for the 5% AEP, 1% AEP and PMF events are displayed on

Figure B46 to Figure B48.

7.4.5. Preliminary Flood Emergency Response Classification

The design flood modelling was assessed in accordance with guidelines for Emergency Response Planning (ERP) outlined in Reference 21. These guidelines are generally more applicable to riverine flooding where significant flood warning time is available and emergency response action can be taken prior to the flood, or where long-term isolation may occur requiring possible resupply or medical evacuation. It is unclear how to apply the classifications in flash flood areas where there is little or no warning, and isolation times will be relatively short.

In urban areas like the Little Creek catchment, flash flooding from local catchment and overland flow will generally occur as a direct response to intense rainfall without significant warning. For most flood affected properties in the catchment, remaining inside the home or building is likely to present less risk to life than attempting to drive or wade through floodwaters, as flow velocities and depths are likely to be greater in the roadway.

The design modelling indicates that in the PMF event some properties will be subject to high hazard flooding with depths greater than 1.0 m covering access routes, prior to potential flooding of buildings. As floor level survey is unavailable for these properties, it is unclear what the depth of above-floor inundation of buildings on these properties may be. If estimated depths were life-threatening, these properties would need to be classified as “Low Flood Island” according to Reference 21, as by the time above-floor inundation occurs the roadways at the property frontages would already be inundated with high hazard flooding.

If a property is unaffected by above floor flooding but nearby streets are flooded, vehicular access from the area may be blocked, causing inconvenience or potentially threatening life if emergency medical care is required during a flood. This issue of flood isolation is less critical for urban flash flooding than for rural flooding as it is unlikely that access will be cut for more than a few hours. For example it is unlikely that provision of food or other supplies to isolated areas will be required in the Little Creek catchment. For this preliminary assessment, some areas have been classified as “Low Flood Island” or “Low Trapped Perimeter Area” where it was assessed that there is a real risk of injury or death if residents become trapped in their homes during a flood. The SES does not provide definitive guidance on flood depth or velocity threshold before a road is “cut,” or on “acceptable” isolation times. For this study, roads have been assessed as potentially cut if the significant majority of the road is flooded by depths greater than 0.3 m.

In light of these considerations, preliminary classification for the majority of the study area catchment is as “Indirectly Affected Area,” or “Not Flood Affected,” with some areas marked as “Low Flood Island/Trapped Perimeter Area,” or as “Overland Escape” or “Overland Refuge” areas. Classification has been undertaken for the PMF, 1% AEP and 5% AEP, and 20% AEP events (Figure C1 to Figure C4).

Properties in the area along the trunk drainage alignment between Oxley Park Public School and Hobart Street should be identified as priorities for any emergency response in this catchment.

7.4.6. Preliminary "True" Flood Hazard Categorisation

The provisional hazards were reviewed in this study to consider other factors such as rate of rise of floodwaters, duration, threat to life, danger and difficulty in evacuating people and possessions and the potential for damage, social disruption and loss of production. These factors and related comments are given in Table 26.

Table 26: Weightings for Assessment of True Hazard

Criteria	Weight [1]	Comment
Rate of Rise of Floodwaters	High	The rate of rise in the creek channels and onset of overland flow along roads would be very rapid, which would not allow time for residents to prepare for the onset of flooding.
Duration of Flooding	Low	The duration for local catchment flooding will generally be less than around 6 hours, resulting in inconvenience to affected residents but not necessarily a significant increase in hazard.
Effective Flood Access	High	Roads within the catchment will generally be inundated prior to property inundation, which may restrict vehicular access during a flood.
Size of the Flood	Moderate	The hazard can change significantly at some locations with the magnitude of the flood, particularly between Canberra Street and Hobart Street. However, these changes in hazard are generally captured by mapping a range of events using the provisional hazard criteria.
Effective Warning and Evacuation Times	High	There is very little, if any, warning time. During the day residents will be aware of the heavy rain but at night (if asleep) residential and non-residential building floors may be inundated with no prior warning.
Additional Concerns such as Bank Erosion, Debris, Wind Wave Action	Low	These issues are a relatively minor consideration in urban environments like the Little Creek catchment.
Evacuation Difficulties	Low	Given the quick response of the catchment pre-flood evacuation is unlikely to occur. There may be significant difficulties evacuating people who become trapped in their houses, but only if the depth is sufficient to present a risk to life. This factor is already captured by the provisional hydraulic hazard classification, and therefore was not given significant weight for assessing true hazard.
Flood Awareness of the Community	Moderate	Urban communities in general have relatively low flood awareness and a short "community memory" for historical flood events.
Depth and Velocity of Floodwaters	High	In areas of overland flow roads are subject to fast flowing water. In the main creek channels velocities and depth would be high. There is always a risk of a car or pedestrian being swept into the open channel while attempting to cross swiftly flowing waters at major creek crossings. However this factor is largely included in the provisional hydraulic hazard calculation metrics.

Note: [1] Relative weighting in assessing the preliminary true hazard.

For the Little Creek catchment, the factors with high weighting in relation to assessment of true hazard are generally related to the lack of flood warning, the dangers of driving on flooded roads, and the potential for flooding of access to residential properties prior to above-floor flooding of buildings occurring. In many cases, it is likely that remaining inside the property will present less risk to life than attempting evacuation via flooded routes, as refuge can generally be taken on furniture etc. There may be some properties where remaining inside would present a high risk to life due to very high flood depths, but these properties will generally already be classified as high hazard using provisional hazard criteria.

When considering the Flood Emergency Response classifications (see Section 7.4.5 above), the higher risk classifications such as “Low Flood Island” overlapped with areas already classified as “high hazard” using the provisional hydraulic hazard criteria.

In general it was found that areas where a high flood hazard would be justified based on consideration of the high-weight criteria in Table 26, the area was already designated high hazard as a result of the depth/velocity criteria used to develop the provisional hazard. Therefore the preliminary “true” hazard categories were assessed to be the same as the provisional hydraulic hazard (see Figure C5 to Figure C7).

7.5. Road Inundation

Flood level hydrographs showing flooding of key road reserves and creek crossings are provided in Appendix C, Figure C9 to Figure C17 (locations shown on Figure C8). These figures are included to provide the SES with an understanding of the period of time that the roads may be subject to hazard and inundation for the design events considered.

8. SENSITIVITY ANALYSIS

8.1. Overview

A number of sensitivity analyses were undertaken for the hydraulic model to establish the variation in design flood levels and flow that may occur if different parameter assumptions were made. These sensitivity scenarios are summarised in Table 27. The 5% AEP and 1% AEP event were analysed.

Table 27: Overview of Sensitivity Analyses

Scenario	Description
Manning's "n"	The hydraulic roughness values were increased and decreased by 20%.
Pipe, Culvert and open Channel Blockage	Sensitivity to blockage of all pipes with inlet headwalls was assessed for 25% and 75% blockage (compared to 50% for design modelling).
Climate Change	Sensitivity to rainfall and runoff estimates were assessed by increasing the rainfall intensities by 10%, 20% and 30% as recommended under the current guidelines. Sensitivity of a rainfall intensity reduction of 20% was also investigated.
Coincident South Creek Flooding Tailwater Level	For the 5% AEP event, the effect of a lower tailwater level was assessed (using the same tailwater as assumed for the 10% AEP and smaller events).
Soil Type	Soil type 2 and 4 were assessed, compared to the design event assumption of type 3.
Fence	The sensitivity of removing fences from the model was assessed.
Pit inlet blockage	The sensitivity of assuming no pit blockage and total pit blockage was assessed.

8.2. Climate Change

8.2.1. Background

Intensive scientific investigation is ongoing to estimate the effects that increasing amounts of greenhouse gases (water vapour, carbon dioxide, methane, nitrous oxide, ozone) are having on the average earth surface temperature. Changes to surface and atmospheric temperatures are likely to affect climate and sea levels. The extent of any permanent climatic or sea level change can only be established with certainty through scientific observations over several decades. Nevertheless, it is prudent to consider the possible range of impacts with regard to flooding and the level of flood protection provided by any mitigation works.

Based on the latest research by the United Nations Intergovernmental Panel on Climate Change, evidence is emerging on the likelihood of climate change and sea level rise as a result of increasing greenhouse gasses. In this regard, the following points can be made:

- greenhouse gas concentrations continue to increase;
- global sea level has risen about 0.1 m to 0.25 m in the past century;

- many uncertainties limit the accuracy to which future climate change and sea level rises can be projected and predicted.

8.2.2. Rainfall Increase

The Bureau of Meteorology has indicated that there is no intention at present to revise design rainfalls to take account of the potential climate change, as the implications of temperature changes on extreme rainfall intensities are presently unclear, and there is no certainty that the changes would in fact increase design rainfalls for major flood producing storms. There is some recent literature by CSIRO that suggests extreme rainfalls may increase by up to 30% in parts of NSW (in other places the projected increases are much less or even decrease); however this information is not of sufficient accuracy for use as yet (Reference 20).

Any increase in design flood rainfall intensities would increase the frequency, depth and extent of inundation across the catchment. It has also been suggested that the cyclone belt may move further southwards. The possible impacts of this on design rainfalls cannot be ascertained at this time as little is known about the mechanisms that determine the movement of cyclones under existing conditions.

Projected increases to evaporation are also an important consideration because increased evaporation would lead to generally dryer catchment conditions, resulting in lower runoff from rainfall. Mean annual rainfall is projected to decrease, which will also result in generally dryer catchment conditions. This is less of a factor in urbanised catchments with a high proportion of paved surfaces and irrigated residential gardens.

The combination of uncertainty about projected changes in rainfall and evaporation makes it extremely difficult to predict with confidence the likely changes to peak flows for large flood events within the study area catchment under warmer climate scenarios.

In light of this uncertainty, the NSW State Government's (Reference 20) advice recommends sensitivity analysis on flood modelling should be undertaken to develop an understanding of the effect of various levels of change in the hydrologic regime on the project at hand. Specifically, it is suggested that increases of 10%, 20% and 30% to rainfall intensity be considered.

8.3. Sensitivity Analysis Results

Maps of the peak flood level sensitivity for each of the sensitivity tests are shown in Appendix D (Figure D5 to Figure D31). Note these maps show the full raw model result extents, including areas with depths below 0.15 m, which were filtered out from the final design flood maps (see Section 7.4).

8.3.1. Blockage Variations

Peak flood level results were found to be not sensitive to the blockage assumption for culverts and bridges with inlet headwalls. The number of blocked pipes is small, and the capacity of these structures during large events like the 5% AEP and 1% AEP is typically small compared with the total flow. The impacts of blockage are therefore relatively small and localised, and blockage is not considered a critical parameter for the design flood modelling.

The sensitivity results for culvert and bridge blockage are mapped on Figure D5 to Figure D8 and tabulated in Table 28.

8.3.2. Removal of Fences

The modelled fences along the main flow paths path were removed for both 1% AEP and 5% AEP event. This was found to have a negligible influence on flood behaviour, with only minor localised variations observed.

The sensitivity results for culvert and bridge blockage are mapped on Figure D9 to Figure D10. The results are not tabulated due to the negligible differences in peak flood level for the majority of the study area.

8.3.3. Roughness Variation

Overall peak flood level results were shown to be relatively insensitive to variations in the roughness parameter. The largest change in flood levels occurs within the Forrester Road and the upstream part of the open channel reach. These roughness variations had an influence within ±0.1 m, and typically less than ±0.01 m. Roughness is therefore not considered to be a significant parameter for the modelling.

The sensitivity results for roughness variation are mapped on Figure D11 to Figure D14, and tabulated in Table 29.

8.3.4. Rainfall Variations

The effect of increasing the design rainfalls by 10%, 20% and 30% was evaluated for the 1% AEP and 5% AEP design events, and also a reduction in rainfall of 20%. Increases in rainfall would significantly increase peak flood levels observed throughout the study area. Generally speaking, each incremental 10% increase in rainfall results in an increase in peak flood levels of between 0.02 m to 0.05 m at most of the locations analysed. Significantly higher increases would occur within Hobart Street, Kenny Avenue and Brisbane Street, as the peak flows would exceed the capacity of the culverts under the railway line, resulting in notable increases in the flood depths upstream. Reductions in rainfall intensity would have similar but opposite effects.

The sensitivity results for rainfall variation are mapped on Figure D15 to Figure D20 and Figure D30 to Figure D31, and tabulated in Table 30.

8.3.5. Soil Type Variations

The adopted soil type for design modelling was type 3. This soil type indicates moderate infiltration rates. Type 2 will have higher infiltration rates and moderately well drained, which will reduce the runoff and reduce the peak flood levels and flows. Type 4 will have high runoff potential, very slow infiltration rates, which will increase runoff and peak flood levels and flows.

Soil type was found to be one of the most critical assumptions for the design flood modelling, with the most influence on peak flood levels throughout the study area. Changing the soil type was found to have an impact of between ±0.05 m and ±0.25m across the study area. This finding was instrumental in the use of soil type as a key calibration parameter. Volume-sensitive locations such as Hobart Street and the detention basins were found to be the most affected by changing soil type. These locations are more sensitive to total runoff volume across the catchment, which is directly influenced by infiltration rates.

The sensitivity results for soil type variation are mapped on Figure D21 to Figure D24, and tabulated in Table 31.

8.3.6. Pit Blockage Variations

Overall peak flood level results were shown to be relatively insensitive to variations in the pit inlet blockage assumption. The largest change in flood levels is at the Hobart Street sag where there is a major inlet structure, accounting for up to half the inflow to the culvert in the 1% AEP event. The blockage assumptions affected peak flood levels by over 1 m at this location for both events tested. Elsewhere, these variations had an influence within ±0.1 m for the 5% AEP event and 0.05 m for the 1% AEP event.

The sensitivity results for pit blockage variation are mapped on Figure D26 to Figure D29, and tabulated in Table 32.

8.3.7. Downstream Tailwater Levels

The following sensitivity scenarios were tested for the downstream tailwater level assumption:

- The South Creek tailwater level for the 5% AEP rainfall event was lowered to be the same as for the 10% AEP event; and
- The South Creek tailwater level for the smaller event was lowered by 2 m (i.e. from between 19.5 mAHD and 20.0 mAHD to between 17.5 mAHD and 18.0 mAHD).

For these scenarios, the lower tailwater level was found to have a negligible impact on Little Creek local catchment flooding. However in larger South Creek events such as the 1% AEP and PMF, this South Creek flood mechanism is likely to present the critical peak flood level for lower parts of the Little Creek catchment, and results from Reference 7 need to be considered as part of Council's development control activities.

The sensitivity results for downstream tailwater variation are shown in Figure D25, but are not tabulated due to the negligible differences in peak flood level for the majority of the study area.

Table 28: Results of Culvert Blockage Sensitivity Analysis

ID	Location	1% AEP Sensitivity Change in Peak Water Level (m)		5% AEP Sensitivity Change in Peak Water Level (m)	
		Blocked Open Structure by 25%	Blocked Open Structure by 75%	Blocked Open Structure by 25%	Blocked Open Structure by 75%
H001	Patricia_St	0.00	0.00	0.00	0.00
H002	Bentley_Rd	0.00	0.00	0.00	0.00
H003	Carpenter_St	0.00	0.00	0.00	0.00
H004	Colyton_School	0.00	0.00	0.00	0.00
H005	Kent_Pl	0.00	0.00	0.00	0.00
H006	Shane_St	0.00	0.00	0.00	0.00
H007	Bennet_Rd	0.00	0.00	0.00	0.00
H008	GreatWestern_Hwy	0.00	0.00	0.00	0.00
H009	Ridge_Park_S	-0.01	0.01	-0.01	0.02
H010	Ridge_Park_N	-0.02	0.02	-0.03	0.03
H011	Adelaide_St	-0.02	0.03	-0.07	0.06
H012	Canberra_St	0.00	0.01	0.00	0.00
H013	Sydney_St	0.00	0.01	0.00	0.00
H014	Brisbane_St	0.00	0.00	-0.01	0.00
H015	Thompson_Ave	0.00	0.00	0.00	0.01
H016	Kenny_Ave	0.00	0.00	0.00	0.01
H017	Hobart_St	0.02	-0.02	0.01	-0.01
H018	Plasser_Cres	0.00	0.00	0.00	0.00
H019	Kurrajong_Rd	0.00	0.00	0.00	0.00
H020	Glossop_St	-0.03	0.05	-0.04	0.04
H021	Forrester_Rd	-0.02	0.03	0.03	0.03
H022	Maxim_Pl	-0.01	0.02	-0.02	0.03
H023	Structure_8	-0.03	0.02	-0.03	0.03
H024	Structure_9	0.00	0.00	0.00	0.00
H025	LeeHolm_Rd	0.00	0.00	0.00	0.00
H026	Christie_St	0.00	0.00	0.00	0.00
H027	LittleCreek_US	-0.01	0.01	-0.02	0.01
H028	LittleCreek_DS	0.00	0.00	0.00	0.00
H029	Camira_St	0.00	0.00	0.00	0.00
H030	Morris_St	0.00	0.00	0.00	0.00
H031	Jacka_St	0.00	0.00	0.00	0.00
H032	Edmondson_Ave	0.00	0.00	0.00	0.00
H033	Adelaide_St_W	0.00	0.00	0.00	0.00
H034	Carpenter_St_W	0.00	0.00	0.00	0.00
H035	Muscio_St	0.00	0.00	0.00	0.00
H036	Ball_St	0.00	0.00	0.00	0.00
H037	GreatWestern_Hwy_W	0.00	0.00	0.00	0.00
H038	Christie_St_N	0.00	0.00	0.00	0.00
H039	Glossop_St_W	-0.02	0.01	-0.01	0.01
H040	Forrester_Rd_W	-0.02	0.04	0.04	0.05

Table 29: Results of Roughness Variation Sensitivity Analysis

ID	Location	1% AEP Sensitivity Change in Peak Water Level (m)		5% AEP Sensitivity Change in Peak Water Level (m)	
		Roughness Decreased 20%	Roughness Increased 20%	Roughness Decreased 20%	Roughness Increased 20%
H001	Patricia_St	-0.01	0.01	0.00	0.00
H002	Bentley_Rd	-0.01	0.01	-0.01	0.01
H003	Carpenter_St	0.00	0.00	0.01	0.00
H004	Colyton_School	0.00	0.00	0.01	-0.01
H005	Kent_Pl	0.00	0.00	0.00	0.00
H006	Shane_St	-0.02	0.02	-0.02	0.01
H007	Bennet_Rd	0.00	0.00	0.00	0.00
H008	GreatWestern_Hwy	0.01	0.00	0.02	-0.02
H009	Ridge_Park_S	0.00	0.00	-0.01	0.01
H010	Ridge_Park_N	-0.01	0.01	-0.01	0.01
H011	Adelaide_St	-0.01	0.01	0.00	0.00
H012	Canberra_St	0.00	0.00	0.00	0.00
H013	Sydney_St	0.00	0.00	0.00	0.00
H014	Brisbane_St	-0.01	0.01	-0.01	0.00
H015	Thompson_Ave	-0.03	0.02	-0.02	0.01
H016	Kenny_Ave	-0.01	0.01	0.00	0.00
H017	Hobart_St	0.04	-0.04	0.05	-0.05
H018	Plasser_Cres	0.00	0.00	0.01	-0.01
H019	Kurrajong_Rd	-0.04	0.03	-0.02	0.03
H020	Glossop_St	-0.01	0.01	0.00	0.00
H021	Forrester_Rd	-0.10	0.10	-0.04	0.10
H022	Maxim_Pl	0.00	0.01	0.00	0.01
H023	Structure_8	-0.01	0.01	-0.01	0.01
H024	Structure_9	-0.01	0.01	-0.01	0.01
H025	LeeHolm_Rd	-0.01	0.01	-0.01	0.01
H026	Christie_St	-0.01	0.01	0.00	0.00
H027	LittleCreek_US	-0.09	0.08	-0.09	0.08
H028	LittleCreek_DS	0.00	0.00	0.00	0.00
H029	Camira_St	0.01	0.01	0.03	0.01
H030	Morris_St	-0.01	0.01	-0.01	0.01
H031	Jacka_St	-0.01	0.01	-0.01	0.01
H032	Edmondson_Ave	-0.01	0.01	-0.01	0.01
H033	Adelaide_St_W	-0.01	0.01	-0.01	0.01
H034	Carpenter_St_W	-0.01	0.01	-0.01	0.01
H035	Muscio_St	-0.02	0.02	-0.02	0.02
H036	Ball_St	0.00	0.00	0.01	0.00
H037	GreatWestern_Hwy_W	0.00	0.00	0.00	0.00
H038	Christie_St_N	-0.03	0.02	-0.02	0.02
H039	Glossop_St_W	-0.14	0.09	-0.09	0.09
H040	Forrester_Rd_W	-0.11	0.10	-0.14	0.11

Table 30: Results of Rainfall Intensity Analysis

ID	Location	1% AEP Sensitivity Change in Peak Water Level (m)				5% AEP Sensitivity Change in Peak Water Level (m)			
		+10%	+20%	+30%	-20%	+10%	+20%	+30%	-20%
H001	Patricia_St	0.02	0.05	0.07	-0.06	0.03	0.05	0.07	-0.07
H002	Bentley_Rd	0.01	0.02	0.02	-0.02	0.01	0.02	0.03	-0.03
H003	Carpenter_St	0.02	0.04	0.05	-0.05	0.03	0.04	0.06	-0.08
H004	Colyton_School	0.03	0.06	0.08	-0.17	0.09	0.17	0.21	-0.23
H005	Kent_Pl	0.04	0.07	0.10	-0.08	0.04	0.07	0.10	-0.07
H006	Shane_St	0.03	0.07	0.10	-0.08	0.04	0.07	0.10	-0.09
H007	Bennet_Rd	0.04	0.07	0.11	-0.08	0.04	0.07	0.10	-0.11
H008	GreatWestern_Hwy	0.04	0.07	0.10	-0.15	0.09	0.14	0.18	-0.25
H009	Ridge_Park_S	0.05	0.10	0.14	-0.10	0.03	0.07	0.11	-0.05
H010	Ridge_Park_N	0.03	0.07	0.10	-0.06	0.03	0.05	0.08	-0.27
H011	Adelaide_St	0.07	0.14	0.20	-0.14	0.07	0.12	0.17	-0.20
H012	Canberra_St	0.06	0.11	0.16	-0.04	0.02	0.03	0.04	-0.04
H013	Sydney_St	0.07	0.12	0.17	-0.04	0.02	0.03	0.04	-0.04
H014	Brisbane_St	0.06	0.13	0.20	-0.08	0.03	0.06	0.09	-0.08
H015	Thompson_Ave	0.04	0.34	0.63	-0.09	0.03	0.07	0.11	-0.08
H016	Kenny_Ave	0.36	0.72	1.02	-0.07	0.03	0.05	0.08	-0.07
H017	Hobart_St	0.43	0.79	1.09	-0.45	0.18	0.35	0.54	-0.51
H018	Plasser_Cres	0.04	0.07	0.09	-0.12	0.06	0.10	0.13	-0.10
H019	Kurrajong_Rd	0.03	0.05	0.08	-0.07	0.03	0.05	0.08	-0.07
H020	Glossop_St	0.01	0.04	0.05	-0.03	0.01	0.03	0.04	-0.04
H021	Forrester_Rd	0.11	0.20	0.30	-0.24	0.09	0.19	0.28	-0.09
H022	Maxim_Pl	0.02	0.04	0.06	-0.05	0.02	0.05	0.07	-0.07
H023	Structure_8	0.02	0.04	0.06	-0.05	0.02	0.04	0.06	-0.05
H024	Structure_9	0.01	0.01	0.03	-0.01	0.00	0.01	0.01	-0.01
H025	LeeHolm_Rd	0.03	0.05	0.07	-0.06	0.03	0.05	0.07	-0.07
H026	Christie_St	0.01	0.01	0.02	-0.01	0.01	0.01	0.02	-0.01
H027	LittleCreek_US	0.04	0.08	0.11	-0.11	0.05	0.09	0.13	-0.16
H028	LittleCreek_DS	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
H029	Camira_St	0.02	0.04	0.05	-0.09	0.05	0.08	0.10	-0.12
H030	Morris_St	0.02	0.04	0.05	-0.04	0.02	0.03	0.05	-0.05
H031	Jacka_St	0.02	0.04	0.05	-0.05	0.02	0.04	0.05	-0.07
H032	Edmondson_Ave	0.01	0.01	0.01	-0.01	0.01	0.01	0.01	-0.01
H033	Adelaide_St_W	0.01	0.02	0.03	-0.03	0.01	0.02	0.03	-0.03
H034	Carpenter_St_W	0.01	0.02	0.03	-0.03	0.01	0.02	0.03	-0.04
H035	Muscio_St	0.02	0.03	0.05	-0.03	0.01	0.02	0.04	-0.05
H036	Ball_St	0.04	0.08	0.12	-0.15	0.08	0.14	0.18	0.00
H037	GreatWestern_Hwy_W	0.03	0.05	0.06	-0.08	0.04	0.07	0.10	-0.22
H038	Christie_St_N	0.03	0.05	0.08	-0.06	0.02	0.05	0.07	-0.05
H039	Glossop_St_W	0.05	0.12	0.19	-0.17	0.06	0.12	0.18	-0.17
H040	Forrester_Rd_W	0.10	0.19	0.28	-0.24	0.10	0.20	0.29	-0.20

Table 31: Results of Soil Type Variation Sensitivity Analysis

ID	Location	1% AEP Sensitivity Change in Peak Water Level (m)		5% AEP Sensitivity Change in Peak Water Level (m)	
		Soil Type 2	Soil Type 4	Soil Type 2	Soil Type 4
H001	Patricia_St	-0.08	0.02	-0.10	0.03
H002	Bentley_Rd	-0.03	0.00	-0.06	0.01
H003	Carpenter_St	-0.07	0.02	-0.10	0.03
H004	Colyton_School	-0.25	0.03	-0.32	0.12
H005	Kent_Pl	-0.10	0.03	-0.10	0.04
H006	Shane_St	-0.10	0.02	-0.13	0.04
H007	Bennet_Rd	-0.10	0.03	-0.17	0.04
H008	GreatWestern_Hwy	-0.21	0.03	-0.34	0.09
H009	Ridge_Park_S	-0.11	0.04	-0.07	0.03
H010	Ridge_Park_N	-0.08	0.03	-0.49	0.03
H011	Adelaide_St	-0.18	0.06	-0.27	0.07
H012	Canberra_St	-0.04	0.05	-0.06	0.01
H013	Sydney_St	-0.04	0.06	-0.06	0.01
H014	Brisbane_St	-0.09	0.04	-0.11	0.03
H015	Thompson_Ave	-0.10	0.03	-0.11	0.04
H016	Kenny_Ave	-0.07	0.29	-0.09	0.03
H017	Hobart_St	-0.52	0.36	-0.63	0.19
H018	Plasser_Cres	-0.07	0.02	-0.09	0.04
H019	Kurrajong_Rd	-0.06	0.02	-0.07	0.03
H020	Glossop_St	-0.03	0.01	-0.05	0.01
H021	Forrester_Rd	-0.21	0.08	-0.07	0.08
H022	Maxim_Pl	-0.05	0.02	-0.07	0.02
H023	Structure_8	-0.04	0.01	-0.06	0.02
H024	Structure_9	-0.01	0.00	-0.01	0.00
H025	LeeHolm_Rd	-0.04	0.01	-0.04	0.02
H026	Christie_St	-0.01	0.00	-0.01	0.00
H027	LittleCreek_US	-0.11	0.03	-0.19	0.05
H028	LittleCreek_DS	0.00	0.00	0.00	0.00
H029	Camira_St	-0.04	0.01	-0.08	0.04
H030	Morris_St	-0.05	0.01	-0.08	0.01
H031	Jacka_St	-0.06	0.01	-0.14	0.01
H032	Edmondson_Ave	-0.01	0.00	-0.02	0.00
H033	Adelaide_St_W	-0.03	0.00	-0.04	0.01
H034	Carpenter_St_W	-0.03	0.01	-0.06	0.01
H035	Muscio_St	-0.04	0.01	-0.09	0.01
H036	Ball_St	-0.17	0.03	-0.01	0.09
H037	GreatWestern_Hwy_W	-0.11	0.01	-0.35	0.03
H038	Christie_St_N	-0.03	0.01	-0.03	0.01
H039	Glossop_St_W	-0.18	0.04	-0.20	0.06
H040	Forrester_Rd_W	-0.21	0.07	-0.14	0.09

Table 32: Results of Pit Inlet Blockage Sensitivity Analysis

ID	Location	1% AEP Sensitivity Change in Peak Water Level (m)		5% AEP Sensitivity Change in Peak Water Level (m)	
		No Blockage	Full Blockage	No Blockage	Full Blockage
H001	Patricia_St	-0.01	0.07	-0.01	0.09
H002	Bentley_Rd	0.00	0.02	0.00	0.03
H003	Carpenter_St	0.00	0.02	0.00	0.03
H004	Colyton_School	0.01	0.02	0.01	0.12
H005	Kent_Pl	0.01	0.06	0.01	0.07
H006	Shane_St	0.00	0.06	0.00	0.08
H007	Bennet_Rd	0.00	0.08	-0.01	0.10
H008	GreatWestern_Hwy	-0.06	0.14	-0.10	0.25
H009	Ridge_Park_S	-0.02	0.04	0.01	0.03
H010	Ridge_Park_N	0.00	0.00	0.01	-0.03
H011	Adelaide_St	-0.02	0.08	-0.08	0.05
H012	Canberra_St	-0.01	0.08	-0.01	0.06
H013	Sydney_St	-0.01	0.08	-0.01	0.06
H014	Brisbane_St	0.00	0.19	-0.02	0.10
H015	Thompson_Ave	0.01	1.40	-0.01	0.77
H016	Kenny_Ave	0.01	1.80	-0.01	1.14
H017	Hobart_St	-0.13	1.87	-0.14	1.62
H018	Plasser_Cres	-0.02	0.15	-0.06	0.23
H019	Kurrajong_Rd	0.01	0.02	0.01	0.06
H020	Glossop_St	0.01	-0.10	0.01	-0.07
H021	Forrester_Rd	0.04	-0.25	0.03	-0.09
H022	Maxim_Pl	0.01	-0.07	0.01	-0.12
H023	Structure_8	0.01	-0.08	0.01	-0.11
H024	Structure_9	0.00	-0.02	0.00	-0.01
H025	LeeHolm_Rd	0.00	0.04	0.00	0.05
H026	Christie_St	0.00	0.00	0.00	0.00
H027	LittleCreek_US	0.04	-0.78	0.04	-0.87
H028	LittleCreek_DS	0.00	0.00	0.00	0.01
H029	Camira_St	0.00	-0.14	0.00	-0.09
H030	Morris_St	0.00	0.05	0.00	0.05
H031	Jacka_St	-0.01	0.07	-0.02	0.08
H032	Edmondson_Ave	0.00	-0.01	0.00	-0.02
H033	Adelaide_St_W	0.00	0.01	-0.01	0.01
H034	Carpenter_St_W	0.00	0.02	0.00	0.02
H035	Muscio_St	0.00	0.02	0.00	0.02
H036	Ball_St	-0.06	0.15	0.00	0.25
H037	GreatWestern_Hwy_W	0.00	0.07	0.00	0.13
H038	Christie_St_N	0.00	0.00	0.00	0.00
H039	Glossop_St_W	0.03	-0.48	0.04	-0.48
H040	Forrester_Rd_W	0.04	-0.27	0.04	-0.27

9. PRELIMINARY FLOOD PLANNING AREA

9.1. Background

Land use planning is one of the most effective means of minimising flood risk and damages from flooding. The Flood Planning Area (FPA) identifies land that is subject to flood related development controls and the Flood Planning Level (FPL) is the minimum floor level applied to development proposals within the FPA.

The process of defining FPAs and FPLs is somewhat complicated by the variability of flow conditions between mainstream and local overland flow, particularly in urban areas. Traditional approaches that were developed for riverine environments and "mainstream" flow areas often cannot be applied in steeper urban overland flow areas.

Defining the area of flood affectation due to overland flow (which by its nature includes shallow flow) often involves determining at which point it becomes significant enough to classify as "flooding" rather than just drainage of local runoff. The difference in peak flood level between events of varying magnitude may be minor in areas of overland flow, such that applying the typical freeboard can result in a FPL much greater than the Probable Maximum Flood (PMF) level.

The FPA should include properties where future development would result in impacts on flood behaviour in the surrounding area and areas of high hazard that pose a risk to safety or life. Further to this, the FPL is determined with the purpose to decrease the likelihood of over-floor flooding of buildings and the associated damages.

The Floodplain Development Manual suggests that the FPL generally be based on the 1% AEP event plus an appropriate freeboard. The typical freeboard cited in the manual is that of 0.5 m; however it also recognises that different freeboards may be deemed more appropriate due to local conditions. In these circumstances, some justification is called for where a lower value is adopted.

Further consideration of flood planning areas and levels are typically undertaken as part of the Floodplain Management Study where council decides which approach to adopt for inclusion in their Floodplain Management Plan.

9.2. Methodology

The methodology used in this report is consistent with that adopted in a number of similar studies throughout the Sydney metropolitan area. It divides the flood area between "mainstream" flooding and "overland" flooding areas using the following criteria:

- Mainstream flooding: Areas along the main creek or trunk drainage alignment, where flow is sufficiently deep and there is sufficient relief that 0.5m freeboard can be added to the flood surface and the extent can be "stretched" to include adjacent land. The mainstream part of the study area was defined as the Little Creek trunk drainage alignment extending from Kent Place Colyton at the upstream extent, down through the open channel section to South Creek at the downstream extent. The FPA along this reach was defined as the peak flood level plus a 0.5 m freeboard, with the level extended perpendicular to the flow direction either side of the flow path.

- Overland flooding: Other lots in the study area were classified as “flood control lots” and therefore within the FPA, if they were affected by the modelled 1% AEP flood extent (after applying filtering as described in Section 7.4). Therefore this classification only includes areas affected by flood depths greater than 0.15 m in the 1% AEP event.

Figure B49 identifies the extent of the preliminary FPA developed using the methodology above.

9.3. Identification of Preliminary Flood Control Lots

Flood Tagging is the process where lots are identified as flood liable. The “tagged” lots will be subject to Section 149(2) notification (under NSW Local Government Act) indicating that their properties are subject to flood related development controls. This simply means that should development of the lots occur, flooding will need to be considered and Council’s LEP, DCP and any other relevant flood related policies will apply.

Flood tagging is undertaken using a three step process, shown below:

Step 1. GIS Analysis	Step 2. Desktop Analysis	Step 3. Ground Truthing
• Lots affected according to the "mainstream" or "overland flow" criteria identified from modelling results	• Initial tagged lots identified in Stage 1 are mapped with 1% flood depths, building footprint information and the DEM (ground elevation data) • Manual visual review undertaken to identify lots which may have been omitted or may have unessecarily been identified as tagged	• The lots ideintified for further review in Step 2 are assessed as part of a site visit • WMAwater engineers visit each lot, assess local ground conditions and determine their validity for tagging • A final tagging status is assigned, based on a range of assessment criteria

Preliminary tagging was undertaken for the Little Creek study area, based on hydraulic model results, using the mainstream and overland criteria described above.

A verification process was then undertaken to refine the delineation of flood control lots. This process is referred to as “ground truthing.” Some potentially flooded lots are omitted from tagging during the initial assessment, due to the approximations required to construct the computational model of the catchment, and due to the sensitivities of GIS processing. Furthermore, some lots may be initially identified as flood control lots, which in reality are unlikely to be subject to significant flooding. Ground truthing was undertaken first through desktop analysis, and then a site visit for properties requiring detailed investigation. The results of this process were provided in GIS format to Council.

10. DESIGN FLOOD BEHAVIOUR ANALYSIS

10.1. Pipe Capacity Assessment

The design flood modelling was analysed to determine how frequently the stormwater pipe system capacity is likely to be exceeded throughout the catchment. Defining the maximum capacity of a pipe is not straightforward, as it depends on multiple factors including the shape of the pipe, the flow regime (e.g. upstream or downstream controlled), the inlet and outlet connections, the pipe grade, and other factors. For example, the nominal flow capacity of a pipe may increase with significant head to drive flow at the upstream end, but this "maximum" flow may be only slightly larger than when the soffit of the pipe is first exceeded, and the upstream afflux is an undesirable outcome in terms of reducing surface flooding.

TUFLOW provides output indicating the proportion of the cross-section area of the pipe that has flow in it. For the purposes of the pipe capacity assessment, pipes were assumed to be "full" when the flow area equalled or exceeded 85% of the pipe cross-sectional area. This is the point at which circular pipes tend to be close to their most efficient, since at 100% of cross-sectional area the additional friction from the top of the pipe reduces the pipe conveyance more than the slight increase in flow area. Similarly, box culverts designed for a supercritical flow regime will typically be designed for free surface flow approximately 80% of the depth of the culvert, as when flow touches the pipe soffit it will typically "trip" the flow regime to become sub-critical, resulting in lower capacity, depending on the pipe grade. Furthermore, due to energy losses associated with adjoining pits, inlets, culvert bends etc., some culverts may never become "100% full," although they may be 90% full for a range of design flood events (e.g. from the 5% AEP through to the PMF). In such circumstances, it is informative to know the design storm for which the pipe is almost at its maximum capacity.

Figure 19 shows the outcomes of the pipe capacity assessment. In the upper catchment area, the majority of pipes have less than 50% AEP capacity (that is, they are effectively "full" in the 50% AEP event). The main trunk system varies along its length with capacity summarised as follows:

- Less than 50% AEP capacity upstream of Bennett Road
- Between 20% AEP and 10% AEP capacity from Bennett Road to Great Western Highway
- Between 5% AEP and 2% AEP capacity from the Oxley Park basins to Brisbane Street
- Between 20% AEP and 10% AEP capacity from Brisbane Street to Kenny Avenue
- Less than 50% AEP capacity from Kenny Avenue to Hobart Street;
- 5% AEP capacity in the old brick arch culvert under the railway line immediately downstream of Hobart St;
- 2% AEP to 1% AEP capacity from the Railway Line to Plasser Crescent;
- Less than 50% AEP capacity from Plasser Crescent to the Kurrajong Road outlet

These results are fairly typical for urban trunk drainage networks, although there appear to be "choke points" at Hobart Street and at the Kurrajong Road outlet. The low capacity upstream of Hobart Street is probably more related to the overland flow obstruction presented by the railway embankment, rather than purely related to the pipe size at this location. This is because the deep ponding at the Hobart Street low-point submerges the pipe junction and pressurises the stormwater network at this location, causing the pipes to be “full” even in relatively small events.

At the Kurrajong Road outlet, the relatively shallow grade of the open channel to Glossop Street may cause a backwater effect, submerging the pipe outlet and causing the pipe to be “full” even in relatively small events.

Therefore, upgrading the pipe sizes at these locations would not necessarily improve the capacity at these choke points. More discussion of these issues is provided in the hot-spot analysis and discussion of potential mitigation options below.

10.2. “Hot-Spot” Analysis

Some of the key flood-prone areas within the catchment, termed “hotspots,” are discussed in more detail in this section of the report. Figure 20 provides an overview of the locations discussed.

10.2.1. HS1 – Hobart Street Low Point

The Hobart Street low point is located to the southern side of the railway line (Location HS1, Figure 20). Photo 13 and Photo 14 show the inlet pits on each site of Hobart Street. The contributing upstream catchment is more than half of the total Little Creek catchment area. The railway embankment acts as a significant obstruction to overland flow at this location.

Photo 13: Trunk drain inlet on Hobart Street

Photo 14: Drainage pit along Hobart Street

The railway embankment crest is 40.6 mAHD at the Hobart Street low point, approximately 6.1 m above the road level. There are four 1.2 m pipes draining the catchment towards this location, and the outlet from the area is a single arch culvert under the railway line with dimensions of approximately 2.4 m by 2.4 m. There is a large junction pit between Hobart St and the railway line, which has a large overland flow inlet grate above (see Photo 13). Excess overland flow exceeding the pipe capacity ponds in the low point along Hobart Street.

Design flood levels within the low point, flows within the pipes underneath Hobart Street and the overland flow over Hobart Street are summarised in Table 33. The locations of these results, as well as localised depth mapping and hydrographs for the 1% AEP event are shown on

Figure B50.

Table 33: Design flow behaviour near the Hobart Street sag point

Event	Peak Flood Level (m AHD)	Peak Flood Depth (m)	Peak Inflow (m^3/s)		Peak Outflow (m^3/s)	
			Pipe	Overland	Pipe	Overland
50% AEP	34.9	0.5	5.5	1.9	7.6	0.0
20% AEP	35.2	0.7	6.6	3.9	10.4	0.0
10% AEP	35.5	1.0	7.1	5.7	11.8	0.0
5% AEP	35.8	1.3	8.4	7.7	13.5	0.0
2% AEP	36.0	1.6	8.0	10.0	14.6	0.0
1% AEP	36.2	1.8	7.5	12.4	15.6	0.0
0.5% AEP	36.7	2.3	7.6	14.8	16.6	0.0
0.2% AEP	37.2	2.8	7.6	17.8	17.8	0.0
PMF	40.9	6.5	7.9	104.7	24.8	79.9

The low point has a peak flood depth of 0.5 m in the 50% AEP event, increasing significantly to 1.8 m in the 1% AEP event, and to 6.5 m in the PMF event. This is due to the height of the railway embankment. For larger flood events when the capacity of the culvert under the railway line is exceeded, the embankment acts like a dam wall, retaining a large volume of water to the south.

Modelling indicates that the embankment would only be overtopped in extreme events such as the PMF event. In the 1% AEP event approximately 10 existing residential buildings are affected by flooding. However in the PMF event this increases to over 200, with the backwater effect of the railway embankment stretching back to Oxley Park Public School on Adelaide Street, 5 to 6 blocks to the south. Floor level survey would be needed to confirm how many properties would be affected by over-floor inundation for each event.

Note that there is no overland flow out of the area (across the railway embankment) for events up to the 0.2% AEP event. In the PMF, flow will overtop the embankment, and a significant portion of flow will be diverted eastwards along the railway corridor, out the study area catchment and towards Ropes Creek.

It can be seen from the 1% AEP hydrographs on Figure B50 that the railway outflow pipe capacity is significantly higher than the inflow pipe capacity to Hobart Street, however there is also a significant amount of overland flow that arrives at this location. When the additional overland flow arriving at the area exceeds the outflow capacity there is a sharp rise in the flood level, rising by approximately 1 m in a period of 20 to 30 minutes for the design 1% AP event.

The older brick arch culvert under the railway line, which is at the exit to the inlet structure in Photo 13, appears to have slightly less capacity than the trunk drain further downstream of the railway line. This suggests that design flood depths in the sag point could be reduced by increasing the cross-drainage capacity at the railway line, although this would involve considerable expense (see Section 10.3 below for more discussion).

10.2.2. HS2 – Plasser Crescent to Kurrajong Road

Between the railway line and Kurrajong Road there is an industrial subdivision, which was constructed over the creek line in the 1980s. The primary trunk drainage line under this subdivision is a single 2.7 m by 2.1 m box culvert, which splits into two culverts with dimensions of 2.4 m by 1.2 m immediately before the outlet at Kurrajong Road (see Photo 15 and Photo 16). Downstream of Kurrajong Road, the creek becomes open channel to South Creek, with occasional road crossings (notably Glossop Street and Forrester Road). There is an additional outlet at Kurrajong Road further east which drains local flows from the industrial subdivision area (see Photo 16). Prior to the development occurring, this was the primary creek crossing, and several of the existing culvert barrels at this location were blocked off. The new outlet is further east, and the two swales (from the old and new outlets) join slightly further downstream from Kurrajong Road (see Photo 17).

Photo 15: Primary trunk drain outlet at Kurrajong Rd showing stagnant water

Photo 16: Secondary Kurrajong Rd outlet

Photo 17: Open channel confluence

The invert of the culvert outlet on the main drainage branch is 30.8 mAHD, and there is a concrete apron constructed at a similar level. However it can be seen in Photo 15 that accumulated silt and reeds at this location result in standing water at the downstream end of the culvert.

Design flood levels within the Kurrajong Road low point, flows within the main trunk drain, and the overland flow at Plasser Crescent and Kurrajong Road are summarised in Table 34. The locations of these results, as well as localised depth mapping and hydrographs for the 1% AEP event, are shown on Figure B51.

Table 34: Design flow behaviour at the Kurrajong Road sag point

Event	Peak Flood Level (m AHD)	Peak Flood Depth (m)	Plasser Crescent flow (m^3/s)		Kurrajong Road flow (m^3/s)	
			Pipe	Overland	Pipe	Overland
50% AEP	32.4	0.3	8.0	0.1	8.8	0.4
20% AEP	32.6	0.5	11.1	0.4	12.0	0.5
10% AEP	32.6	0.5	12.6	0.5	13.5	0.6
5% AEP	32.7	0.6	14.4	0.6	15.2	0.8
2% AEP	32.7	0.6	15.8	0.6	16.3	1.0
1% AEP	32.8	0.7	16.3	0.7	17.1	1.3
0.5% AEP	32.8	0.7	16.6	0.9	17.4	2.0
0.2% AEP	32.8	0.7	17.5	1.2	18.3	2.9
PMF	33.4	1.3	23.0	56.0	23.8	66.6

There is relatively little overland flow through this area from upstream due to the presence of the railway embankment, which blocks overland flow in all events modelled except for the PMF. Ponding within Kurrajong Road and Plasser Crescent is therefore primarily a result of local catchment runoff. The local sag point in Kurrajong Road has a peak flood depth of 0.3 m in the 50% AEP event, increasing moderately to 0.7 m in the 1% AEP event, and to 1.3 m in the PMF event.

The pipe capacity assessment indicates that the outlet pipes from the trunk drainage network become full in a 50% AEP event, well below Council's 5% AEP objective for such systems. This may be a reflection of a backwater effect from the open channel and the influence of the Gloosop Street bridge, rather than the culverts themselves being too small. It is recommended that works to lower the tailwater in the open channel section be investigated to determine whether the flood issues further upstream in Hobart Street would be alleviated. Such works might involve regrading or altering the channel construction, or increasing capacity of the cross-drainage at Glossop Street. These potential flood mitigation works are discussed further in Section 10.3.

10.2.3. HS3 – Canberra Street Low Point

The Canberra Street low point is located slightly west of the intersection between Canberra Street and Sydney Street (Location HS3, Figure 20 and Photo 18). There is also a sag point within Sydney Street slightly south of the intersection (Photo 19).

Photo 18: Canberra St sag point

Photo 19: Sydney St sag near Canberra St

The roads and land rise upwards away from this sag point in all directions. This location is therefore slightly unusual compared to other roads that cross the main drainage line, since there is higher land on the downstream side of the sag point. Water must therefore pond to a greater depth in the road before overland flow can exit through properties to the north-west. The situation is exacerbated since there is no formal overland flow path or easement through properties to the north-west, unlike at Brisbane Street and Thompson Avenue.

Furthermore, there are relatively few inlets or local drainage connections into the main trunk stormwater pipes. Council records indicate the trunk drain consists of four 1.2 m diameter pipes running to the north-west at this location. This could not be fully confirmed during the detail survey process, since the local inlet pits in Canberra Street did not directly connect to the larger trunk drainage pipes. This lack of local drainage connections possibly exacerbates the ponding issues at this location.

Design flood levels within the Canberra Road low point, flows within the main trunk drain, and the overland flow upstream and downstream are summarised in Table 35. The locations of these results, as well as localised depth mapping and hydrographs for the 1% AEP event, are shown on Figure B52.

Table 35: Design flow behaviour near the Canberra Street low point

Event	Peak Flood Level (m AHD)	Peak Flood Depth (m)	Peak Inflow (m^3/s)		Peak Outflow (m^3/s)	
			Pipe	Overland	Pipe	Overland
50% AEP	38.9	0.5	3.4	0.2	3.6	1.0
20% AEP	38.9	0.5	4.4	0.5	4.6	3.4
10% AEP	39.0	0.6	4.9	0.6	5.2	4.6
5% AEP	39.0	0.6	6.6	1.1	6.6	6.2
2% AEP	39.0	0.6	7.3	3.7	6.8	7.5
1% AEP	39.0	0.6	7.4	6.2	6.7	8.8
0.5% AEP	39.1	0.7	7.5	9.2	6.7	11.6
0.2% AEP	39.2	0.8	7.1	15.2	6.5	18.0
PMF	40.9	2.5	7.6	100.2	6.5	115.3

The community consultation feedback identified that there were significant issues with flooding of residential property in the vicinity of this location. The modelling results confirm that inundation of properties above floor level is possible even in relatively frequent events such as the 50% AEP or 20% AEP events, since some properties have relatively little elevation above surrounding ground levels.

The results in Table 35 indicate that overland flows out of the low-point significantly exceed the upstream overland inflows for the range of design events modelled. This suggests that the majority of ponding is caused by runoff and overland flows from the immediate local catchments, rather than along the main drainage line. Furthermore, the pipe capacity analysis (Figure 19) suggests the pipe has approximately 5% AEP capacity, but significant ponding was modelled to occur even in the 0.5% AEP event, when the pipes were discharging only about 50% of full capacity.

These results strongly suggest that additional inlet capacity at the low point could significantly increase the amount of flow discharging via the trunk drain, and reduce the depth of ponding and peak overland flow through properties in the vicinity of the low point. See Section 10.3 below for further discussion.

10.2.4. HS4 – Oxley Park Detention Basins

The Oxley Park detention basins are located on Council land to the north of Great Western Highway, along the main Little Creek trunk drainage alignment (Location HS4, Figure 20). There are two adjoining basins separated by an internal control embankment, with a single 1.2 m diameter linking pipe (Photo 20). Inflows to the basin occur primarily from three 1.5 m diameter pipes crossing the Great Western Highway, as well as overland flow across the Great Western Highway in the 5% AEP and larger events. There is overland flow upstream from Bennett Road in smaller events, but it is blocked from reaching the basin by the Great Western Highway. The outlet from the downstream basin is a single 1.2 m diameter pipe (Photo 21), which is covered by a large debris screen, and discharges into three 1.2 m pipes under Oxley Park Public School.

Photo 20: Control structure in Oxley Park basins

Photo 21: Oxley Park basin outlet and spillway

Design flood levels within the basins, and flows into and out of the basins (pipe and overland), are summarised in Table 36. The locations of these results, as well as localised depth mapping and hydrographs for the 1% AEP event, are shown on Figure B53.

Table 36: Design flow behaviour at the Oxley Park detention basins

Event	Peak Flood Level (m AHD)	Peak Flood Depth (m)	Peak Inflow (m^3/s)		Peak Outflow (m^3/s)	
			Pipe	Overland	Pipe	Overland
50% AEP	40.6	0.9	5.2	0.0	2.7	0.3
20% AEP	41.2	1.5	6.2	0.0	3.7	0.5
10% AEP	41.5	1.8	6.9	0.0	4.2	0.6
5% AEP	41.8	2.1	7.6	0.4	3.7	3.8
2% AEP	41.9	2.1	8.1	2.0	4.1	6.5
1% AEP	41.9	2.2	8.4	4.8	4.2	8.9
0.5% AEP	41.9	2.2	8.7	7.9	4.3	12.0
0.2% AEP	42.0	2.3	9.0	12.5	3.4	18.6
PMF	42.4	2.7	9.9	67.8	4.1	89.6

These results indicate that the basins significantly attenuate peak flows, reducing flood risk for downstream areas, particularly for more frequent flood events. However, in the 1% AEP event, although total peak flows to downstream areas are reduced, the peak overland flow through the Oxley Park Public School is slightly increased compared to the overland flow into the basin. This result occurs because there is a relatively flat spillway from the basin, such that when the basin fills and overtops the spillway there is a relatively fast increase in outflow, and the outlet pipe has significantly lower capacity than the inlet pipes to the system. This means that under some circumstances, there can be a higher proportion of overland flow compared to pipe flow downstream of the basins compared to upstream of the basins.

In light of these results, it may be appropriate to increase the size of the basin outlet to reduce the proportion of overland flow exiting the basin across the spillway in larger floods, with the aim of reducing risk to those present in the school downstream. The current outlet pipe (1.2 m diameter) is significantly lower capacity than the three 1.2 m diameter pipes immediately downstream. See Section 10.3 for more discussion of potential flood mitigation options.

Flood issues upstream of the basins at the Great Western Highway are discussed further below (Hot spot 6, Section 10.2.6).

10.2.5. HS5 – Shane Street Low Point

There is a low point in the Shane Street cul-de-sac immediately adjacent to the intersection with Bennett Road. There is also a sag point in Kent Place slightly upstream to the south-east. Both of these sag points were identified as part of the community consultation process, with properties being flooded above floor level in the March 2014 storm, which was probably in the order of a 10% AEP to 5% AEP storm. The sag points are located in the upstream part of the Little Creek catchment (Location HS5, Figure 20). Photos of the Shane Street / Bennett Road intersection are shown in Photo 22 and Photo 23.

Photo 22: Shane Street and Bennett Street intersection

Photo 23: Shane Street near Bennett Street

Design flood levels within the Shane Street low point, flows within the pipes, and the overland flow immediately upstream and downstream of the sag point are summarised in Table 37. The pipe inflows are the sum of the flows in the pipes down Bennett Road, as well as those through private property from Kent Place. The upstream overland flow path occurs through private property from Kent Place towards Bennett Road, as well as along Bennett Road itself. The downstream flows are those through the reserve / easement between Bennett Road and the Great Western Highway. The locations of these results, as well as localised depth mapping and hydrographs for the 1% AEP event, are shown on Figure B54.

Table 37: Design flow behaviour near the Shane Street low point

Event	Peak Flood Level (m AHD)	Peak Flood Depth (m)	Peak Inflow (m^3/s)		Peak Outflow (m^3/s)	
			Pipe	Overland	Pipe	Overland
50% AEP	44.4	0.2	3.1	1.0	4.0	0.2
20% AEP	44.5	0.3	3.2	2.3	4.4	1.1
10% AEP	44.6	0.3	3.2	3.5	4.5	2.4
5% AEP	44.6	0.4	3.2	4.9	4.6	4.2
2% AEP	44.7	0.4	3.2	6.2	4.6	6.0
1% AEP	44.7	0.5	3.2	7.6	4.6	7.9
0.5% AEP	44.7	0.5	3.3	9.3	4.6	9.9
0.2% AEP	44.8	0.6	3.2	11.7	4.6	12.9
PMF	45.3	1.1	3.2	46.1	4.6	52.1

There is a 1.05 m pipe draining the upstream catchment along Bennett Road (which splits into two 0.9 m diameter pipes), as well as a single 0.6 m diameter pipe from Kent Place, which expands to a 0.9 m diameter pipe at Shane Street. Downstream of the Shane Street and Bennett Road intersection, the trunk drain consists of three 1.2 m diameter pipes.

It can be seen from the results in Table 37 that none of the pipes in this location, neither upstream or downstream of Shane Street, have sufficient capacity to meet Council's 5% AEP design flow objective. The pipes convey less than half the total peak flow in the 5% AEP event through this area. Although ponding in the road reserve is relatively shallow, there are several low-lying properties in this area that are vulnerable to flooding above floor level even in relatively frequent storm events.

In light of the above, it is likely that additional pipe capacity in this area, particularly along the branch from Kent Place to Bennett Road, and from Bennett Road through to the Oxley Park detention basins, would significantly reduce flood damages to property and nuisance flooding of roads through this area. See Section 10.3 for more discussion of potential flood mitigation options.

10.2.6. HS6 – Great Western Highway Low Point (East)

There is a sag point at the Great Western Highway (GWH) low point, along the main drainage line, immediately upstream of the Oxley Park detention basins (Location HS6, Figure 20). Photo 24 shows views upstream and downstream from the GWH at this sag point.

This hot-spot location is immediately in between hot-spots HS4 and HS5 discussed above. It is analysed separately here to focus specifically on the effect of the Great Western Highway on flow behaviour.

Photo 24: Great Western Highway at HS6, looking downstream (left) and upstream (right)

Photo 25: Outlet from GWH cross-drainage pipes

Photo 26: Inlet to GWH cross-drainage pipes

Design flood levels upstream and downstream of the road, flows within the pipes, and the overland flow are summarised in Table 38. The locations of these results, as well as localised depth mapping and hydrographs for the 1% AEP event, are shown on Figure B55.

Table 38: Design flow behaviour at the Great Western Highway low point (east)

Event	Peak Flood Level Upstream (m AHD)	Peak Flood Level Downstream (m AHD)	Peak Inflow (m^3/s)		Peak Outflow (m^3/s)	
			Pipe	Overland	Pipe	Overland
50% AEP	43.8	41.9	5.2	0.2	5.2	0.0
20% AEP	43.9	42.0	5.7	1.1	6.2	0.0
10% AEP	44.0	42.0	5.9	2.4	6.9	0.0
5% AEP	44.2	42.0	6.1	4.2	7.6	0.4
2% AEP	44.3	42.1	6.1	6.0	8.1	2.0
1% AEP	44.4	42.1	6.2	7.9	8.4	4.8
0.5% AEP	44.4	42.2	6.2	9.9	8.7	7.9
0.2% AEP	44.4	42.3	6.3	12.9	9.0	12.5
PMF	44.8	42.8	6.4	52.1	9.9	67.8

There are three 1.5 m diameter pipes crossing the Great Western Highway at this location (outlet shown in Photo 25, inlets shown in Photo 26). The model results indicate that these pipes generally have some spare capacity compared to those immediately upstream, and flow through these pipes could potentially be improved by upgrading the inlet structure in the reserve (Photo 26).

It can be seen from the results in Table 38 and Figure B55 that the Great Western Highway acts as a significant obstruction to overland flow at this location, particularly from the elevated median strip. Overland flow arriving at the southern side of the road is detained and ponds to a much greater depth than water on the northern side of the road. The elevation of the water is significantly higher than the water level in the basins downstream (i.e. the water is "perched").

This suggests that if the obstruction to overland flow could be reduced, or the pipe capacity under the road could be increased, the water would drain more readily into the detention basins and the flood risk within and upstream of the road could be significantly reduced for a range of flood events. See Section 10.3 for more discussion of potential flood mitigation options at this location.

10.2.7. HS7 – Great Western Highway Low Point (West)

There is another low point further west on the Great Western Highway, between Jacka Street and Cutler Avenue (Location HS7, Figure 20). This sag point has a smaller upstream catchment area than HS6, and therefore lesser quantities of overland and pipe flow arrive at this location. Photo 27 and Photo 28 shows photographs at this sag point looking upstream and downstream.

Design flood levels upstream and downstream of the road, flows within the pipes, and the overland flow are summarised in Table 39. The locations of these results, as well as localised depth mapping and hydrographs for the 1% AEP event, are shown on Figure B56.

Photo 27: GWH low point (upstream side)

Photo 28: GWH low point (downstream side)

Table 39: Design flow behaviour near the Great Western Highway low point (west)

Event	Peak Flood Level (m AHD)	Peak Flood Depth (m)	Peak Inflow (m^3/s)		Peak Outflow (m^3/s)	
			Pipe	Overland	Pipe	Overland
50% AEP	52.2	0.0	0.8	0.0	1.1	0.0
20% AEP	52.3	0.1	1.1	0.1	1.3	0.0
10% AEP	52.5	0.3	1.2	0.4	1.3	0.0
5% AEP	52.6	0.4	1.2	0.7	1.4	0.1
2% AEP	52.6	0.5	1.2	1.0	1.4	0.4
1% AEP	52.7	0.5	1.2	1.3	1.4	0.7
0.5% AEP	52.7	0.5	1.2	1.6	1.4	1.0
0.2% AEP	52.7	0.6	1.2	2.0	1.4	1.5
PMF	52.8	0.7	1.3	4.9	1.4	4.3

There is a 0.9m pipe draining the catchment towards this location, and the outlet from the area is two 0.75 pipes. Excess overland flow exceeding the pipe capacity will pond in the low point along Great Western Highway, on the upstream (southern side). The median strip is relatively high, and the sag point is relatively shallow. As a result, a significant portion of the overland flow that arrives at the sag point from Jacka Street is diverted eastwards along the Great Western Highway (towards HS6), rather than exiting the sag point along the drainage line to the north (towards Cutler Avenue).

It can be seen from Photo 28 that the houses on the downstream side of the sag point are lower than the road level, and therefore vulnerable to flooding from overland flow exiting the sag point. In this regard, the diversionary effect of the median strip is actually beneficial in that it reduces the risk of flooding for these properties, and other properties along the tributary flow path to the north. The flow is instead diverted towards the main trunk drainage line and the Oxley Park basins.

This benefit comes at the expense of increased flood depths along the main road itself, as well as exacerbating the flood issues at location HS6 discussed above. However, the depths along the GWH are relatively shallow, and if flooding at HS6 is mitigated it may be preferable to retain the existing flow behaviour at HS7, rather than increasing flows towards Cutler Avenue, where the drainage system does not have sufficient capacity to meet Council's 5% AEP design standard (see pipe capacity analysis on Figure 19).

On balance therefore, it appears that the existing flow situation at HS7 is likely to produce preferable flow behaviour compared to alternatives of increasing pipe capacity under the road, or removing the obstructing effect of the median strip. However, further investigation of such options may still be warranted at the floodplain risk management stage.

10.3. Discussion of Preliminary Flood Mitigation Options

Based on the hot-spot analysis presented above, and further inspection of the flood study model results throughout the study area, WMAwater developed a list of flood mitigation options that could potentially reduce flood risk in the catchment. These options would require further modelling assessment before implementation, for example as part of a subsequent floodplain risk management study and plan for the catchment.

The potential options are discussed below roughly in order of perceived feasibility, from most feasible to least feasible, based on a preliminary assessment of likely benefits, cost, barriers to implementation and community acceptance.

The location of the potential options is illustrated on Figure 21. The options are designated "FM" for "Flood Mitigation."

10.3.1. Increase Canberra/Sydney Street Inlet Capacity (FM1)

The analysis for hot spot HS3 (Section 10.2.3) identified that generally there is additional pipe capacity in trunk drain for events up to the 5% AEP magnitude. However significant ponding is anticipated to occur in the sag point in smaller events, as there is potentially insufficient inlet capacity to drain local runoff into the trunk system.

There are relatively few kerb inlet pits and local pipe connections to the trunk system at this point, and there are several low-set houses at risk of inundation above floor level in relatively frequent events. Flooding of properties in this area was reported in in the March 2014 storm.

It is recommended that additional inlet pits and local drainage connections at this location be investigated further.

10.3.2. Increase Pipe Network Capacity in Upstream Catchment (FM2)

The pipe capacity assessment and hot spot analysis (HS5 and HS6) identified that the stormwater network from Kent Place to the Great Western Highway, and in the Bennett Street and Carpenter Road catchments, does not have sufficient capacity to convey the 50% AEP flow. There are several low-set houses at risk of inundation above floor level in relatively frequent events, and some properties reported inundation occurring on an annual basis in this area (see Section 1.4). Flooding of properties in this area was reported in in the March 2014 storm.

It is recommended that upgrades of the pipe network along this reach be investigated, with a view to meeting Council's drainage design objective of conveying the 5% AEP flow. Upgrading the system through to the Great Western Highway would result in a greater proportion of flow reaching the Oxley Park Detention Basin as pipe flow rather than overland flow, reducing flood risk to property and people in this reach.

Such an upgrade would involve works on pipes through private land, so engagement with the land owners would be required. However, those properties which would require works to upgrade the pipe network would be the same properties to directly benefit from the works. Given the recent experiences of flooding in the area, this may present an opportunity to obtain community approval for such works.

10.3.3. Increase Pipe and Overland Flow Capacity Across Great Western Highway into the Oxley Park Detention Basins (FM3)

These two options are discussed together, because they would probably require joint investigation to determine a feasible and cost effective solution. Both options are related to the observation from the hot-spot analysis that peak flood levels upstream of the Great Western Highway are significantly higher than those within the Oxley Park detention basins, and therefore if the flow conveyance could be increased across the road, the upstream flood levels could be reduced and more water detained in the basins. This could be an increase in pipe capacity, removal of obstructions to overland flow, or a combination of both measures. Optimisation of the outlet structure may also be required to ensure there were not adverse impacts downstream (see FM6 below).

The hotspot analysis (HS6, Section 10.2.6) identified that the median strip and road cross fall at the Great Western Highway sag point presents a significant obstruction to overland flow (see Photo 24). At the eastern sag point on the main trunk drainage line, modelling indicates significant flooding of properties upstream of the road, and that the water level is significantly higher than the level within the detention basins downstream of the road.

This indicates that if the median strip could be modified to reduce the flow obstruction (for instance by introducing small gaps at regular intervals along the strip), flood risk upstream of the road could be significantly reduced.

Similarly, if the pipe capacity under the Great Western Highway could be increased, along with the inlet capacity within the public reserve just upstream, this would result in a greater proportion of flow reaching the Oxley Park Detention Basin as pipe flow rather than overland flow. This may have a similar effect of lowering flood levels for properties upstream of the Great Western Highway.

This option is also related to option FM4 below.

10.3.4. Detention Storage from Bennett Rd to Great Western Hwy (FM4)

This option would also potentially mitigate flooding at the same location as the median strip modification. There is an overland flow easement/reserve between Bennett Road and the Great Western Highway where significant flood depths occur. This area could potentially be excavated to increase temporary flood storage and locally reduce flood levels, both for properties adjacent to the reserve as well as for traffic on the Great Western Highway.

This option could potentially be combined with an upgrade to the trunk drain inlet structure immediately upstream of the Great Western Highway (see Photo 26, Section 10.2.6). A larger inlet could potentially increase capture of overland flow in the reserve through the pipes under the Great Western Highway, reducing the flow depth across the road as well as for neighbouring properties.

These works have the advantage that they could potentially significantly improve flood risk at this location without requiring works within the Great Western highway road reserve itself, and the traffic disruption and costs that would be associated with such works.

10.3.5. Increase Pipe Capacity from Jacka St to Brisbane St (FM5)

The pipe capacity assessment indicated that for the western stormwater branch from Jacka Street to Brisbane Street, the existing pipes have less than 50% AEP capacity. There are several low-set houses at risk of inundation above floor level in relatively frequent events. Flooding of properties in this area was reported in in the March 2014 storm.

It is recommended that upgrades of the pipe network along this reach be investigated, with a view to meeting Council's drainage design objective of conveying the 5% AEP flow.

Such an upgrade would involve works on pipes through private land, so engagement with the land owners would be required. However, those properties which would require works to upgrade the pipe network would be the same properties to directly benefit from the works. Given the recent experiences of flooding in the area, this may present an opportunity to obtain community approval for such works.

This option would involve upgrades to a significant length of pipe, making it relatively expensive and also difficult to coordinate with land owners. Therefore this option is likely to be less feasible than other pipe upgrades in the catchment discussed above.

10.3.6. Alter Outlet Capacity and Spillway Crest from Oxley Park Basins (FM6)

The hotspot analysis (HS4, Section 10.2.4) identified that although the basins are effective at attenuating total peak flows for a range of flood events, in larger flood events it may result in an increase in peak overland flow through the Public School immediately downstream. The outlet pipe from the basins (a single 1.2 m diameter pipe) acts as a restriction to flow to detain water in the basin. However this pipe flows into three 1.2 m diameter pipes immediately downstream of the basin.

It may therefore be appropriate to increase the outlet pipe capacity from the basin slightly. This would be expected to reduce the flow attenuation for more frequent smaller events, and instead make increased use of the basin storage for larger storms such as the 1% AEP event. This could potentially reduce the overland flow hazard and flood risk for downstream areas significantly.

A related option would be to investigate changes to the outlet spillway crest height and/or profile, in order to potentially increase the detention storage volume and further mitigate flood risk downstream.

It is therefore recommended to investigate increasing the outlet pipe size from the Oxley Park basin. It is also recommended that a dam break analysis be undertaken to understand the level of risk to people and property from possible failure of the outlet spillway, and whether further basin modifications are warranted to mitigate that risk.

It is important that such analysis should consider multiple storm durations, not just the catchment critical storm duration investigated as part of this study, to avoid over-optimising the basin based on design storm characteristics, while potentially resulting in sub-optimal performance for real storms with slightly different characteristics.

10.3.7. Upgrade Forrester Rd Bridge Culvert Capacity (FM7)

Design flood modelling results indicate that the Forrester Road bridge would be overtopped by flow in Little Creek in the 50% AEP event (see Figure C10). Upgrades to the bridge should be investigated to determine whether it would be cost effective to meet Council’s 5% AEP design objective, and whether the risk of blockage of the structure could be reduced (see Section 5.5).

10.3.8. Modify Open Channel near Kurrajong Road (FM8)

Works-as-executed survey from the industrial estate at Plasser Crescent / Kurrajong Road indicates that the outlet to the open channel section of Little Creek has an invert of 30.8 mAHD. However, LIDAR and detail survey indicates that in the open channel immediately downstream, the creek invert levels are between 31.0 mAHD and 32.0 mAHD. The upstream invert of the Glossop Street culverts, approximately 250 m downstream, are 29.7 mAHD, 1.1 m below the outlet at Kurrajong Road (a grade of 0.4%).

It should be investigated whether construction of a small low-flow concrete dish drain along the base of the creek channel would reduce the propensity for sediment to accumulate at the culvert outlets from Kurrajong Road. This could reduce maintenance requirements for the outlets and possibly increase the flow capacity for the trunk drain, mitigating the flooding issues upstream of the railway line at Hobart Street.

10.3.9. Increase Railway Line Cross-Drainage Capacity (FM9)

As discussed in the hot spot analysis (HS1, Section 10.2.1), the railway embankment obstructs overland flow from the Hobart Street sag point, resulting in significant ponding depths if flow arriving at the sag point exceeds the cross-drainage capacity under the railway line.

It should be investigated whether increases to the pipe capacity, or construction of a new high-flow relief structure through the railway embankment, could mitigate flood risk for people and property in Hobart Street. The main risk at this location is the significant risk to life associated with the PMF event when inundation in Hobart Street would be over 6 m deep and submerge hundreds of properties. In the 1% AEP event, although the peak depth in Hobart Street is estimated to be in the order of 2 m deep, there are relatively few properties at risk. Works in the railway corridor are likely to be extremely expensive and these costs may prove prohibitive for the feasibility of this option.

10.4. Peak Height Profiles

The peak flood level profile along the main trunk drainage line is provided on Figure B57, split into sections upstream and downstream of the railway line. At most locations, the range from the 50% AEP flood level to the 0.2% AEP flood level is less than 1 m, with the PMF being significantly higher. The most significant range in flood levels occurs at the Hobart Street trapped low point, and within the Oxley Park detention basins.

10.5. Cross Catchment Flows to Ropes Creek

In the PMF event, when flows reach sufficient depth to overtop the railway embankment at Hobart Street, modelling indicates that a significant portion of the flow will exit the catchment towards Ropes Creek to the east, rather than returning to Little Creek.

Figure B58 shows a flow hydrograph of the flow which exits the catchment, along with water level hydrographs at Hobart Street and within the railway corridor. The peak flow exiting the catchment is just under 20 m^3/s, compared to a total PMF peak overland flow of 110 m^3/s arriving at Hobart Street from upstream.

Further investigation may be required with regards to Ropes Creek flood risk, to determine whether these additional inflows would significantly change the PMF flood risk for that catchment.

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Appendix A

APPENDIX A: GLOSSARY

FLOOD PROBABILITY TERMINOLOGY

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

Annual Exceedance Probability (AEP) is expressed using percentage probability. It expresses the probability that an event of a certain size or larger will occur in any one year, thus a 1% AEP event has a 1% chance of being equalled or exceeded in any one year.

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

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

Glossary - from the Floodplain Development Manual (April 2005 edition)

Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, Government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power. **redevelopment:** refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the

	flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the "flood liable land" concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL's are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the "standard flood event" in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.

flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: • the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or • water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property

	damage to both premises and vehicles; and/or • major overland flow paths through developed areas outside of defined drainage reserves; and/or • the potential to affect a number of buildings along the major flow path.
mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to “water level”. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.