Smithfield West Overland Flood Study

Final Report

VOLUME 1: Report and Appendices

Revision 3
March 2016

Catchment Simulation Solutions

Smithfield West Overland Flood Study

Final Report

REVISION / REVIEW HISTORY

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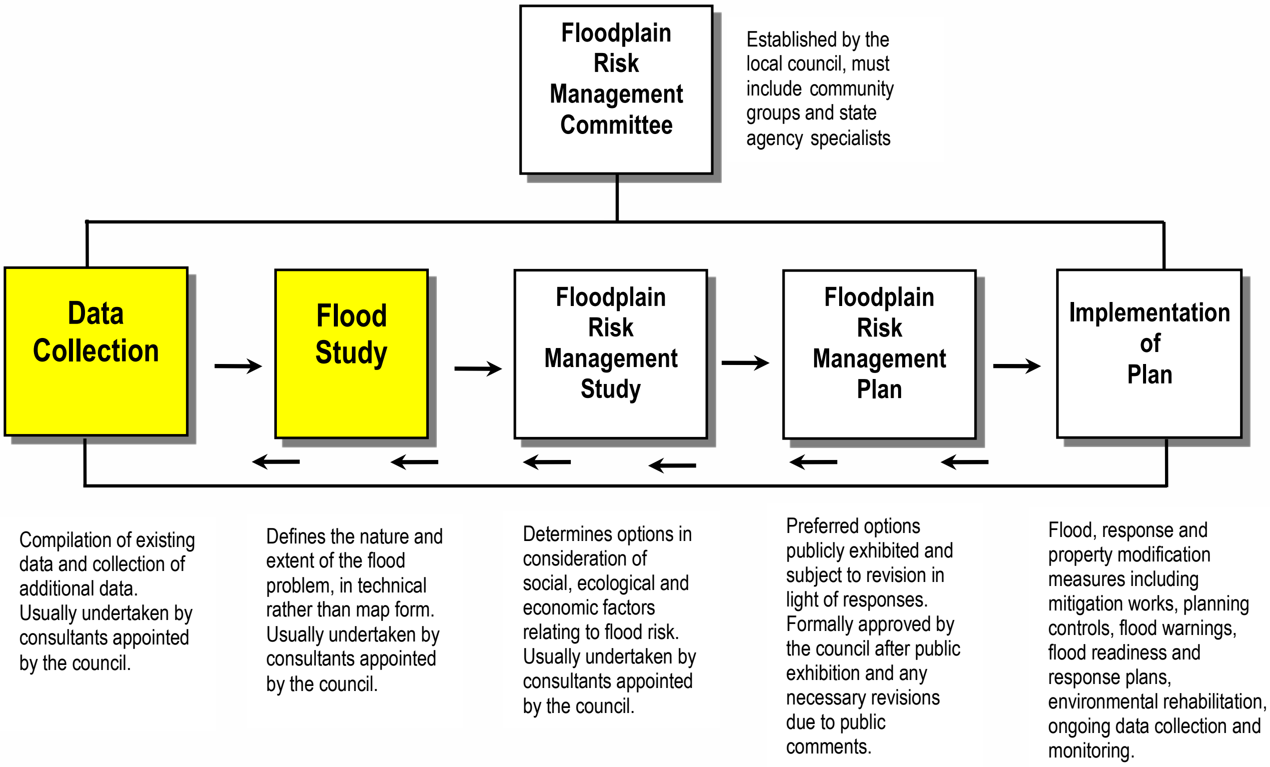
FOREWORD

The NSW State Government's Flood Prone Land Policy is directed towards providing solutions to existing flooding problems in developed areas and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. The Policy is defined in the NSW Government's '*Floodplain Development Manual' (NSW Government, 2005)*.

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

The Policy provides for technical and financial support by the State Government through the following stages:



The Smithfield West Overland Flood Study represents stages one and two of the five stage process outlined above. The aim of the Flood Study is to produce information on flood discharges, levels, depths and velocities, for a range of flood events under existing topographic and development conditions. This information can then be used as a basis for identifying those areas where the greatest flood damage is likely to occur, thereby allowing a targeted assessment of where flood mitigation measures would be best implemented as part of the subsequent Floodplain Risk Management Study and Plan.

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1 INTRODUCTION

1.1 Catchment Description

The Smithfield West catchment is located in the Fairfield City Council Local Government Area and occupies a total area of 152 hectares. The extent of the catchment is shown in **Figure 1**.

The Smithfield West catchment includes the suburb of Smithfield as well as part sections of Prairiewood and Fairfield West. The catchment is highly developed comprising predominately low density residential properties as well as some industrial properties in the vicinity of Market Street. The catchment is drained by a stormwater system that conveys stormwater runoff in a north-easterly direction into Prospect Creek (refer **Figure 1**).

1.2 Purpose of Study

During periods of heavy rainfall across the Smithfield West catchment, there is potential for the capacity of the stormwater system to be exceeded. In such circumstances, the excess water travels overland, potentially leading to inundation of properties. Flooding has been experienced across the Smithfield West catchment on a number of occasions including 1990 as well as more recently in 2012.

A number of flooding investigations have previously been completed in an effort to better understand flood behaviour across the Smithfield West catchment and reduce the impact of flooding on the community. However, these investigations were completed a significant time ago, did not cover the full catchment and did not consider the full range of potential floods. In addition, although each of the previous studies used the best available technology to simulate flood behaviour, there have been significant advances in computer modelling technology since these studies were prepared that provide a more detailed and reliable description of flood behaviour.

In recognition of these limitations and the damage and inconvenience caused by past flooding across the catchment, Fairfield City Council has resolved to prepare a Floodplain Risk Management Plan for the Smithfield West catchment. The first stages in the development of a Floodplain Risk Management Plan involves the compilation of available data and the preparation of a Flood Study. The Flood Study provides a technical assessment of flood behaviour.

This report forms the Overland Flood Study for the Smithfield West catchment. It documents flood behaviour across the catchment for a range of design floods for existing topographic and development conditions. This includes information on flood discharges, levels, depths, flow velocities and flood damage costs for a range of design floods. It also provides estimates of the variation in flood hazard and hydraulic categories across the catchment and provides an assessment of the potential impacts of climate change on existing flood behaviour.

2 METHODOLOGY

2.1 Objectives

Fairfield City Council outlined a range of objectives for the Smithfield West Overland Flood Study. This included:

- to review available flood-related information and historic flood data for the catchment;
- to develop a computer flood model to simulate the transformation of rainfall into runoff and determine how that runoff would be distributed across the catchment;
- to calibrate the computer model to reproduce past floods;
- to use the calibrated computer model to define peak discharges, water levels, depths and velocities for the design 50%, 20%, 10%, 5%, 1% and 0.2% AEP floods, 1 in 10,000-year ARI flood and the Probable Maximum Flood (PMF);
- to verify the design flood results against other studies;
- to produce maps showing floodwater depths and velocities for the 5% and 1% AEP floods as well as the PMF;
- to produce maps showing floodwater levels for the 1% AEP flood; and,
- to produce maps showing flood hazard and hydraulic categories based on definitions provided in the *'Floodplain Development Manual'* (NSW Government, 2005) for the 1% AEP flood and PMF;
- to produce flood risk precinct mapping;
- to quantify the potential impact of climate change on existing design flood behaviour;
- to assess the potential impact of uncertainty on the results produced by the model; and
- to identify properties that are at risk of overland flooding and the associated flood damage that is likely to be incurred.

2.2 Adopted Approach

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

- compilation and review of available flood-related information (Chapter 3);
- the development of an integrated computer based hydrologic/hydraulic model to simulate the transformation of rainfall into runoff and simulate the movement of floodwaters across the Smithfield West catchment (Chapter 4);
- calibration of the computer model to reproduce historic floods (Chapter 5);
- use of the computer models to determine peak discharges, water levels, depths, flow velocities and flood extents for the full range of design events up to and including the PMF for existing topographic and development conditions (Chapter 6);
- use of the computer model results to generate flood hazard, hydraulic category and flood risk precinct mapping (Chapter 7);
- testing the sensitivity of the results generated by the computer model to variations in model input parameters as well as climate change (Chapter 8); and,

- determining the number of properties potentially at risk of inundation and the likely damage that would be incurred during a range of design floods. Also identify flooding "trouble spots" and key infrastructure and roadways that are predicted to be impacted by floodwaters (Chapter 9).

3 Data Collection and Review

3.1 Overview

A range of data were made available to assist with the preparation of the Smithfield West Overland Flood Study. This included previous reports, hydrologic data and GIS data.

A description of each dataset along with a synopsis of its relevance to the flood study is summarised below.

3.2 Catchment History

A range of historic information for the Smithfield West catchment was provided by Fairfield City Council at the outset of the project. This includes records of historic floods as well as mitigation options that have been implemented across the catchment in an effort to reduce flooding problems. A summary of this information is provided below.

3.2.1 Development History

The Smithfield West catchment was originally inhabited by the Cabrogal people for a period of over 30,000 years. European settlement commenced in the early 19th century and the area was subdivided in the 1830's. It was at this time that Smithfield's current street pattern and street names were established.

Smithfield's street pattern and street names began as a venture by John Brenan in the 1840's to establish a settlement around a large meat market, focusing on cattle saleyards. Despite trying to promote interest and hold Fairs, things did not proceed well and the meat market soon closed down. This slowed the progress of Smithfield but the venture gave the district an identity and brought people to the area. Over the coming years the area gradually grew and a church and the first school were built.

The catchment is now highly urbanised, comprising predominately low density residential properties. Much of the current street pattern and street names follow the original survey plan. There is one market garden that remains on Market Street. A significant industrial area that is located on the eastern side of Market Street is also partly contained within the Smithfield West catchment and is home to a range of factories and warehouses. A small commercial strip is also located on Dublin Street, between Brenan and Jane Streets.

As development progressed across the catchment, many of the natural gullies and waterways that historically drained runoff into Prospect Creek were built over and replaced by stormwater pipes. The stormwater pipes were typically designed to "European standards" and do not reflect contemporary design standards. Moreover, the significant increase in impervious surfaces that has occurred since European settlement has resulted in an increase in runoff volume, which further reduces the "design" capacity of the existing stormwater system.

As a result of the limited capacity, the stormwater system can be overwhelmed during significant rainfall events in the catchment. In such instances, the water that cannot be accommodated by the stormwater system must travel overland. This can result in overland flooding.

3.2.2 Flood History

The Smithfield West catchment has experienced overland flooding on a number of occasions. Council records indicate that the most significant contemporary floods occurred in:

- February 1990; and
- April 2012.

The February 1990 event is the most significant flood on record, with six houses being flooded above floor level. This included houses in Moir Street, Hart Street, Victoria Street, Hinkler Street and Chifley Street. Further information on the 1990 flood is documented in the *"Smithfield West Drainage Study"* (refer Section 3.3.1). Photos of the 1990 flood are also provided in Section 3.7.

The April 2012 flood inundated a number of garages, however, no houses were inundated above floor level. Properties around the intersection of Hart and Moir Streets were the most significantly impacted by overland flow. Photos of the 2012 event are included in Section 3.8.3.

3.2.3 Mitigation History

Following the 1990 flood, Council completed an in-house drainage study of the subcatchment area located upstream of The Horsley Drive. 10 houses were identified for house-raising as a result of the study (located adjacent to the main stormwater drainage line on Cartela Crescent, Canara Place, Dublin Street and The Horsley Drive).

Following this study, Council implemented the *"Western Sydney Drainage Initiative - Smithfield West Drain Voluntary House Raising Scheme"*. Since 1993, all ten houses identified by the in-house drainage study have been raised or flood-proofed (refer **Plate 1**).

In addition, where overland flooding is prevalent within the Smithfield West catchment, some open fencing has been installed to reduce the impediment to overland flows. Open fencing has since been installed across a number of properties within the Smithfield West catchment where overland flooding is prevalent (refer **Plate 2**).

In 2009, Council constructed new stormwater pits and pipes along Bourke Street and Moir Street after residents complained of frequent inundation in the area. The pits and pipes were designed to convey flows during floods up to and including the 20% AEP event. However, it should be noted that the stormwater upgrades were located away from the major overland flow paths.

Council has also identified a number of problematic stormwater pits on Brown Street, Lindsay Avenue, Dublin Street (near Neville Street), Cartela Crescent, Canara Place and Hinkler Street on Council's Asset Management System for regular inspection and cleaning. The stormwater pits in Canara Place and Cartela Avenue are listed as "high priority" pits and are checked and cleaned every six months or after heavy storms (whichever occurs first).

Plate 1 26 Cartela Crescent before (top image) and after (bottom image) house raising.

Plate 2 Example of "open" fencing in Cartela Crescent

3.3 Previous Reports

A summary of flood reports that have previously been prepared for the Smithfield West area are provided in the following section. They are listed in chronological order.

3.3.1 Smithfield West Drainage Study: Chifley Street to The Horsley Drive (1996)

The *"Smithfield West Drainage Study: Chifley Street to The Horsley Drive"* report was prepared by Dalland & Lucas for Fairfield City Council. The study was prepared to investigate options for reducing the flooding problem between The Horsley Drive and Chifley Street at Smithfield West.

The report notes that the catchment is heavily urbanised and that the original natural watercourse has been replaced by underground pipes. The report goes on to note that the stormwater pipe system does not have sufficient capacity to convey a 5% AEP flood resulting in significant overland flows during large rainfall events. It also states that urban development has resulted in major obstructions to overland flow paths throughout the catchment.

The report incorporates a significant amount of information on the 10th February 1990 flood, which was estimated to be only slightly less severe than the 1% AEP flood. This includes photographs (refer Section 3.6) as well as flood mark elevations. The location of historic flood marks for the 1990 flood that were extracted from this report are shown in **Figure 2** and the flood mark elevations are summarised in **Appendix A**.

Flood behaviour across the study area was evaluated using an ILSAX model to define hydrology (i.e., rainfall-runoff processes) as well as the capacity of the existing stormwater pipe system and a HEC-RAS hydraulic model to simulate overland flood behaviour. The report notes that the large number of flow obstructions makes the development of a reliable 1-dimensional hydraulic model very difficult.

The computer modelling that was completed as part of the study determined that 18 dwellings would be potentially inundated during the 1% AEP flood with a further 30 predicted to have less than 0.3 metres freeboard.

Five different flood mitigation options were investigated to reduce the flooding problems across the study area. House raising / flood proofing was found to be the most cost effective option but it was still considered to be a relatively expensive option on a cost per property basis relative to other areas within the Fairfield LGA.

3.3.2 Prospect Creek Floodplain Management Plan - Flood Study Review (2006)

The *"Prospect Creek Floodplain Management Plan - Flood Study Review"* was prepared by Bewsher Consulting for Fairfield Council. The report was prepared as part of the *"Prospect Creek Floodplain Management Review 2010"* after a review of previous computer models of Prospect Creek showed some inconsistencies in modelling assumptions relative to other flood studies being completed across the Fairfield City Council LGA at the time.

Smithfield West forms a subcatchment of the larger Prospect Creek catchment. As shown in **Figure 1**, Prospect Creek also forms the downstream boundary of the Smithfield West study area. As a result, flooding along Prospect Creek can result in inundation along the downstream, boundary of the Smithfield West study area. In addition, if flooding along Prospect Creek occurs at the same time as flooding within the Smithfield West subcatchment, it may prevent the local drainage system operating at full efficiency. As a result, the consideration of flooding from Prospect Creek was considered to be an important component of this study.

Hydrology across the Prospect Creek catchment was defined using an XP-RAFTS hydrologic model of the Prospect Creek catchment that was originally developed as part of the *"Review of Prospect Creek Flood Levels"* (Cardno Willing, 2004). However, the original model was updated to accommodate revised areal reduction factors, design rainfall information, rainfall losses and detention basin information. The updated model was verified against a January 2001 flood and was found to provide a reasonable reproduction of the historic peak discharges. The model was subsequently used to simulate a range of design floods and durations. The results produced by the updated XP-RAFTS model are considered to provide the best broad-scale description of contemporary design flow hydrographs across the Prospect Creek catchment. However, the subcatchment delineation that forms the basis of the model is not considered to be detailed enough to reliably define the spatial variation in flows across the relatively small Smithfield West subcatchment.

Flood hydraulics along Prospect Creek were defined using a TUFLOW model that was first developed as part of the *"Review of Prospect Creek Flood Levels"* (Cardno Willing, 2004). However, the model was updated based on the outcomes of a review of the model completed by WBM Pty Ltd as part of the 2006 study. This included updates to culvert loss coefficients, channel cross-sections, 1d-2d connections as well as some topographic updates. The updated model was verified against historic flood mark information for the 2001 flood. The results of the verification showed that the TUFLOW model reproduced historic flood marks to within 0.05 metres (on average) and indicated that the model was providing a reasonable description of flood behaviour along Prospect Creek. Overall, this TUFLOW model is considered to provide the best contemporary description of flood behaviour along Prospect Creek.

The updated XP-RAFTS and TUFLOW models that were developed as part of this previous study were provided by Council for use as part of the current study. It was considered that these models could be incorporated within the current study to define boundary conditions along the downstream extent of the Smithfield West catchment (i.e., along Prospect Creek) and determine the potential impact that coincidental flooding along Prospect Creek may have on flood behaviour across the Smithfield West catchment. Further information on how the XP-RAFTS and TUFLOW models were used to define tailwater elevations along Prospect Creek is provided in Section 5.2.2.

3.3.3 Draft Review of Stormwater Flooding Problems – Moir Street to Victoria Street, Smithfield (2012)

The *"Draft Review of Stormwater Flooding Problems – Moir Street to Victoria Street, Smithfield"* was prepared by FloodMit for Fairfield Council. The report was commissioned to review the February 2012 flood along with past flooding investigations to provide Council with an updated understanding of local flood behaviour / flooding problems between Moir and Victoria Streets at Smithfield West.

The report provides a considerable amount of information on the February 2012 flood, including flood marks that were surveyed by Council following the 2012 flood (refer Section 3.5.2). The report notes that the 2012 event produced peak water levels that were roughly equivalent to the 1990 event in the vicinity of Hart Street, Moir Street and downstream of Victoria Street. However, the 2012 event was considerably (i.e., ~0.4 metres) lower than the 1990 event in the vicinity of Hinkler Street.

An analysis of rainfall records for the 2012 event were completed as part of the study and this determined that the 2012 flood was unlikely to be more severe than a 1 in 40 year ARI (i.e., 2.5% AEP) event. However, it noted that there was considerable variability in rainfall across the Fairfield City Council LGA with rainfall records for other nearby gauges indicating just a 1 in 10 year ARI (10% AEP) event.

The report provided a desktop assessment of the stormwater system and its capacity. This determined that the dual box culvert upstream of The Horsley Drive and the 1350mm diameter pipe downstream of The Horsley Drive would not have sufficient capacity to convey the 20 year ARI (i.e., 5% AEP) event. As a result, overland flow would occur during all events equal to or greater than a 5% AEP event (such as the 1990 and 2012 floods).

The report acknowledges the limitations of the previous 1-dimensional hydraulic computer models of the area and suggests that the development of a fully 2-dimensional computer model should proceed to provide a better representation of design flood behaviour across the catchment. The report also recommends that an "area of significant flow" be delineated which should be kept clear of future development/redevelopment so as not to impede overland flows.

3.3.4 Overland Flood Studies

A range of overland flood studies have been prepared for catchments located across the Fairfield City Council LGA. This includes:

- Canley Corridor Overland Flood Study (SKM, 2009)
- Fairfield CBD Overland Flood Study (SKM, 2010)
- Old Guilford Overland Flood Study (SKM, 2010)
- Smithfield Overland Flood Study (SKM, 2011)

It was considered important to maintain consistency with these previous studies wherever possible. Therefore, the above studies were reviewed and key features that were considered appropriate for application to the current study were identified. This included:

- The TUFLOW software was used to define overland flood behaviour. A 2 metre grid size was adopted to represent the spatial variation in hydraulic characteristics.
- The TUFLOW models were developed to include a representation of the stormwater system as a separate 1-dimensional domain inserted beneath the 2-dimensional domain. This approach allows for the representation of the conveyance of flows by the stormwater system below ground as well as simulation of overland flows in 2D once the capacity of the stormwater system is exceeded.
- For overland catchments draining into a receiving watercourse (e.g., Prospect Creek), it was assumed that floods of equivalent severity were occurring across the local overland catchment and receiving watercourse at the same time during all events up to and including the 1% AEP flood. A 1% AEP flood was retained in the receiving watercourse for all local catchment events greater than the 1% AEP event (e.g., PMF).
- A minimum depth threshold of 0.15 metres has typically been adopted to distinguish between areas of significant and negligible overland flooding.

In general, the overland flood studies used the best available modelling approaches and technology that were available at the time each study was prepared. However, since these overland flood studies were prepared, computer modelling technology has evolved and improved approaches for representing urban overland flooding have been developed. In this regard, the following limitations were identified with the previous studies:

- Buildings were represented in the computer models as completely impervious flow obstructions whereby water is permitted to move around buildings, but not enter them. This approach does not account for the potential storage capacity provided within buildings. This is likely to result in conservative flood level estimates.
- The previous studies acknowledge that fences have the potential to obstruct overland flow. However, they were not explicitly represented in the modelling. The impediment to flow afforded by overland flow obstructions, such as fences, was indirectly represented by increasing the Manning's 'n' roughness value assigned to certain land uses. This approach is considered to provide a reasonable broad-scale description of overland flow behaviour but will likely fail to represent local variations in flood behaviour around specific urban flow obstructions.

- Separate hydrologic models were generally used to define rainfall-runoff processes with flow hydrographs applied to "critical" stormwater pits. This approach may underestimate the capacity of the stormwater system as runoff is not progressively "fed" into upstream stormwater pits and it may fail to represent the path of overland flow travelling to the critical pits. Relatively recent advancements in the TUFLOW software allow application of rainfall directly to the TUFLOW grid avoiding the need for a separate hydrologic model and avoiding some of the limitations associated with application of flows at discreet locations.

It was considered important for the current study to use the best available approaches and technology to represent overland flood behaviour. Further information detailing how the TUFLOW model that was developed for this study overcame the limitations outlined above is provided in Section 4.2.

3.4 Hydrologic Data

3.4.1 Historic Rainfall Data

A number of daily read and continuous (i.e., pluviometer) rainfall gauges are located near the Smithfield West catchment. The location of each gauge is shown in **Figure 3**. Key information for each gauge is summarised in **Table 1**.

The information provided in **Table 1** indicates that daily rainfall records in the vicinity of the study area are available dating back to 1887 (Prospect Reservoir gauge). However, continuous rainfall records are only available from 1984 onwards (Parramatta North - Masons Drive).

3.4.2 Historic Stream Gauge Data

There are no stream gauges located within the Smithfield West catchment. Accordingly, no stream flow information could be found for the study area.

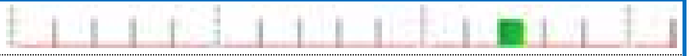
3.5 Topographic and Survey Information

3.5.1 Light Detection and Ranging (LiDAR) Survey

LiDAR data was collected across Sydney in April 2013 by the NSW Government's Land and Property Information department. The LiDAR has a stated absolute horizontal accuracy of better than 0.8 metres and an absolute vertical accuracy of better than 0.3 metres. It is considered that the vertical and horizontal accuracy provided by the LiDAR data is suitable for defining major overland flow paths and is, therefore, suitable for the study.

As the LiDAR was collected relatively recently, it is considered to provide a reliable representation of contemporary topographic conditions across the majority of the catchment. However, LiDAR can provide a less reliable representation of the terrain in areas of high vegetation density. This is associated with the laser ground strikes often being restricted by the vegetation canopy. Errors can also arise if non-ground elevation points (e.g., vegetation canopy) are not correctly removed from the raw LiDAR dataset. Therefore, additional checks were completed across areas of dense vegetation to confirm if the terrain representation was reliable.

Table 1 Available rain gauges in the vicinity of the Smithfield West catchment

Gauge Number	Gauge Name	Gauge Type	Source*	Period of Record		Distance from Catchment	Temporal Availability and Percentage of Annual Record Complete 
				From	To		
67072	Fairfield Heights Post Office	Daily	BOM	Jan 1968	Jan 1975	1.7	
67088	Canley Vale	Daily	BOM	Jan 1886	Dec 1922	3.5	
67019	Prospect Reservoir	Daily	BOM	Jan 1887		3.9	
567083	Prospect Reservoir	Continuous	SW			3.9	N/A
67005	Fairfield Post Office	Daily	BOM	Jan 1930	Dec 1960	4.0	
67017	Greystanes (Bathurst Street)	Daily	BOM	May 2001		4.4	
67070	Merrylands (Welsford Street)	Daily	BOM	Feb 1968	May 2012	5.3	
67008	Guildford	Daily	BOM	Jan 1958	Jan 1977	5.4	
67091	Cabramatta	Daily	BOM	Feb 1945	Jan 1967	5.4	
67006	Fairfield Mwsdb	Daily	BOM	Jan 1961	Oct 1970	5.7	
67119	Horsley Park Equestrian Centre AWS	Daily	BOM	Sep 1997		6.2	
67114	Abbotsbury (Fairfield City Farm)	Daily	BOM	Jan 1999		6.4	
67032	Westmead Austral Avenue	Daily	BOM	Jan 1944	Jan 1992	6.6	
567169	Abbotsbury	Continuous	SW			6.8	N/A
213005	Toongabbie Creek at Briens Rd	Continuous	DNR	April 1979		7.7	
213004	Parramatta Road at Parramatta Hospital	Continuous	DNR	Feb 1979	Jul 1996	8.0	
67035	Liverpool (Whitlam Centre)	Continuous	BOM	Jan 1963	Sep 2001	8.4	
66137	Bankstown Airport AWS	Continuous	BOM	April 1968	June 1992	9.1	
66124	Parramatta North (Masons Drive)	Continuous	BOM	Dec 1984		10.7	

NOTE: * BOM = Bureau of Meteorology, SW = Sydney Water, SCA = Sydney Catchment Authority

Plate 3 provides an example of the LiDAR point density in the vicinity of Market Garden, which includes an open drainage channel with significant vegetation cover. **Plate 3** shows negligible LiDAR ground points in the vicinity of the dense tree/vegetation coverage. Therefore, it appears that non ground points have correctly been removed from the elevation information. However, this also means that the LiDAR does not provide sufficient ground elevation points in the vicinity of dense vegetation to provide a detailed description of the variation in ground surface elevation.

Plate 3 LiDAR data points (yellow crosses) in the vicinity of open channel through Market Garden

Fortunately, the catchment is significantly developed. As a result, the only significant area of dense vegetation occurs across the Market Garden area (near Market and Chifley Streets). Therefore, the LiDAR is considered to provide a suitable description of the variation in ground surface elevations across all areas of the catchment except Market Gardens.

However, it was recognised that the LiDAR data will not pick up the details of the drainage features that are obscured from aerial survey techniques, such as the stormwater system. Therefore, additional survey needed to be collected to ensure the conveyance of the stormwater system could be reliably represented.

3.5.2 2010 Stormwater Survey

Bankstown City Council completed a survey of major (i.e., generally 900mm diameter and above) stormwater pipes across large sections of the Fairfield City Council LGA, including the Smithfield West catchment. The survey provides the alignment and size of major pipes along with the location and details of selected stormwater pits (e.g., pit type, lintel length and pit invert depth). This provided detailed information on 92 stormwater pits and 93 stormwater pipes.

The location of pipes and pits that were surveyed in 2010 by Bankstown City Council is shown in **Figure 2**.

3.5.3 2014 Cross-Section and Stormwater Survey

As discussed, the LiDAR information was not considered to provide a reliable description of the terrain in the vicinity of the open channel draining through Market Gardens. In addition, the 2010 stormwater survey did not cover the full stormwater drainage network. Therefore, it was necessary to collect additional survey information to ensure the conveyance capacity provided by the open channel and stormwater drainage system was fully represented.

Fairfield City Council surveyed critical sections of the stormwater system as well as the open channel draining through Market Gardens in May 2014. The additional data collection included the survey of six cross-sections of the open channel as well as 37 stormwater pits and 45 stormwater pipes. The location of cross-sections and stormwater pits/pipes that were surveyed in May 2014 is shown in **Figure 2**.

3.5.4 Flood Mark Survey

Fairfield City Council also completed a survey of high water marks (i.e., flood marks) across the Smithfield West catchment following the 2012 flood. The flood levels / high water marks were generally reported by residents that were home at the time of the 2012 flood. This ultimately yielded 12 flood marks which are shown on **Figure 2**. The flood mark elevations are also summarised in **Appendix A**.

3.6 GIS Data

A number of Geographic Information System (GIS) layers were also provided by Fairfield City Council to assist with the study. This included:

- <u>Aerial Photography</u> – provides 2014 ortho-rectified aerial imagery at a 0.1 metre pixel size;
- <u>Cadastre</u> – provides property boundary polygons;
- <u>Local Environmental Plan (LEP)</u> – provides zoning / land use information;
- <u>Pipes</u> – provides the alignment and size of stormwater pipes;
- <u>Pits</u> – provides locations of stormwater pits/inlets;

A review of the stormwater pits and pipes layer showed that the GIS information did not always provide a reliable description of the pit locations. For example, **Plate 4** shows the stormwater GIS layers (blue) superimposed on the 2014 aerial imagery. It shows that the stormwater pits are located a significant distance from the “correct” location (up to 30 metres in some instances) and are generally not located along gutters. **Plate 5** also shows that one pit (and the associated connecting pipe) along Victoria Street is not included in the GIS layer. Moreover, the stormwater pit layer provided no information describing the characteristics of each pit (e.g., grate size and lintel size).

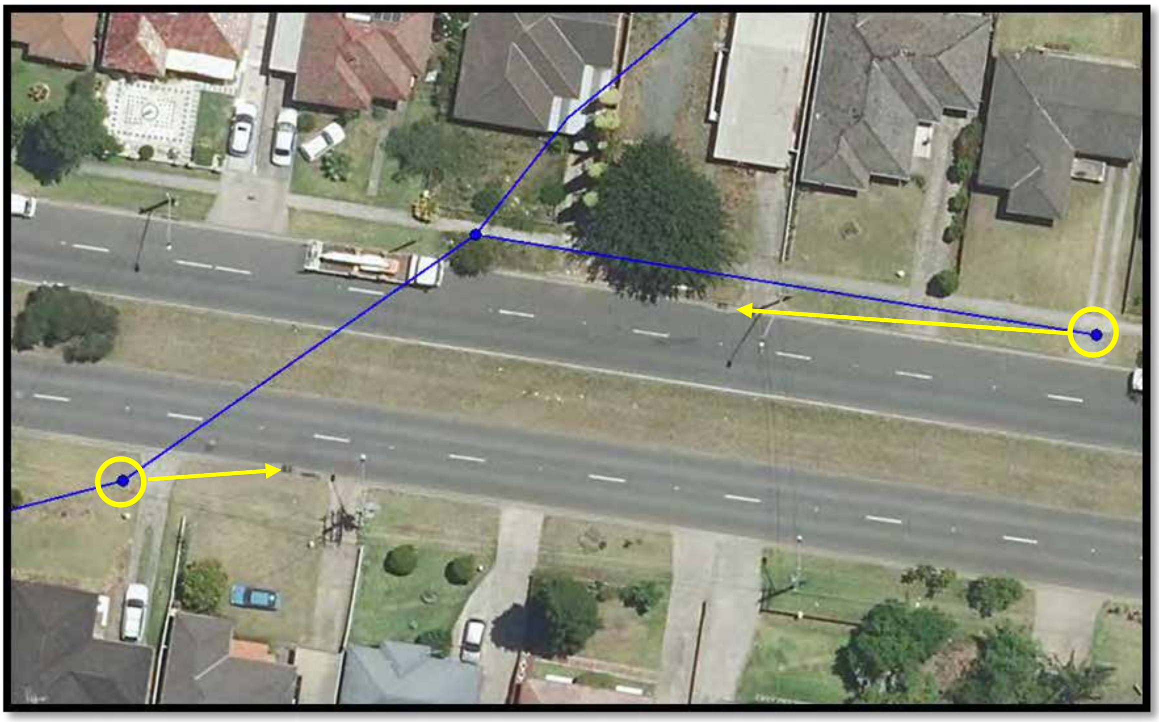


Plate 4 View showing stormwater GIS pits (blue dots) and pipes (blue lines) in vicinity of Victoria Street. They show poor spatial positioning relative to the aerial imagery

Plate 5 Stormwater pit in Victoria Street that is missing from stormwater GIS layer

As discussed above, survey of key stormwater pits and pipes were completed in 2010 and 2014. However, as shown in **Figure 2**, the surveyed stormwater information does not provide a complete description of the stormwater system. Therefore, it was necessary to use the stormwater GIS information to supplement the survey data. However, the spatial accuracy of the GIS information along with grate and lintel size of each pit needed to verified / collected before it could be used as part of the study. The spatial verification was completed based upon the 2014 aerial imagery and lintel size information was estimated using Google Street View.

3.7 Flood Photographs

Fairfield City Council provided a range of photographs of historic floods across the Smithfield West catchment. Most of the photos were taken during the 1990 flood. A selection of these photographs are provided in **Plate 6** to **Plate 11**.

The photos show water extending across roadways and flowing through a number of properties. The photos tend to confirm that some sections of the Smithfield West catchment operate as major overland flow paths during significant rainfall events.

Plate 6 1990 flood looking from 67 Dublin Street

Plate 7 1990 flood looking towards 137C & 139A Victoria Street

Plate 8 1990 flood looking north towards 49 Rhonda Street (near corner of Chifley Street)

Plate 9 1990 flood looking north along Hart Street from Moir Street

Plate 10 1990 flood looking towards 26 and 28 Moir Street

Plate 11 1990 flood looking south towards 26 Moir Street

3.8 Community Consultation

3.8.1 General

A key component of the flood study involved development and calibration of a computer flood model. The computer model is typically calibrated to ensure it is providing a reliable representation of flood behaviour. This is completed by using the model to replicate floods that have occurred in the past (i.e., historic floods).

Although some historic flood information could be sourced from the previous investigations and flood photos, additional information on past flooding was sought from the community to assist with the mode calibration. Therefore, several community consultation devices were developed to inform the community about the study and to obtain information from the community about their past flooding experiences. Further information on each of these consultation devices is provided below.

3.8.2 Flood Study Website

A flood study website was established for the duration of the study. The website address is: http://www.SmithfieldWest.FloodStudy.com.au

The website was developed to provide the community with detailed information about the study and also provide a chance for the community to ask questions and complete an online questionnaire (this online questionnaire was identical to the questionnaire distributed to residents and business owners, as discussed below).

During the course of the study (up to November 2015), the website was visited over 3,000 times.

3.8.3 Community Information Brochure and Questionnaire

A community information brochure and questionnaire was prepared and distributed to potentially flood liable households and businesses within the Smithfield West catchment. The properties that were targeted as part of the mail out were identified by completing a preliminary Probable Maximum Flood (PMF) simulation using the computer flood model (refer Section 4). The brochure and questionnaire were subsequently mailed out to all properties and owners of properties (both residential and business) falling within the preliminary PMF extent. This resulted in 1,712 brochures and questionnaires being distributed. A copy of the brochure and questionnaire is included in **Appendix B**.

The questionnaire sought information from the community regarding whether they had experienced flooding, the nature of flood behaviour, if roads and houses were inundated and whether residents could identify any historic flood marks. A total of 71 questionnaire responses were received. A summary of all questionnaire responses is provided in **Appendix B**. The spatial distribution of questionnaire respondents is shown in **Figure A1**, which is also enclosed in **Appendix B**.

The responses to the questionnaire indicate that:

- The majority of respondents have lived in or around the catchment for about 30 years. Accordingly, most respondents would have been living in the area when the 1990 and 2012 floods occurred.
- 37% of respondents have experienced some form of disruption as a result of flooding in the study area (refer **Plate 12** and **Plate 13**). This includes:
  -> 13 respondents having experienced traffic disruptions; and,
  -> 19 respondents having had their front or back yard inundate.

  The spatial distribution of respondents that have reported past flooding problems is shown in **Figure B1** in **Appendix B** (refer red dots).
- 18 respondents have had their business, house or garage inundated.
- The following streets/areas were identified by several respondents as being particularly susceptible to flooding problems:
  -> Victoria Street/Chifley St (primarily traffic problems)
  -> The Horsley Drive, Moir Street, Hart Street, Market Street, Hinkler Street, Cartela Street, Canara Place (traffic problems, yards flooded and some properties inundated)
- A number of respondents believe flooding is exacerbated by (refer **Plate 14**):
  -> Limited capacity of the exiting stormwater system (18 respondents)
  -> Blockage of the stormwater system (11 respondents)
  -> Overland flow obstructions (e.g., fences, buildings) (8 respondents)

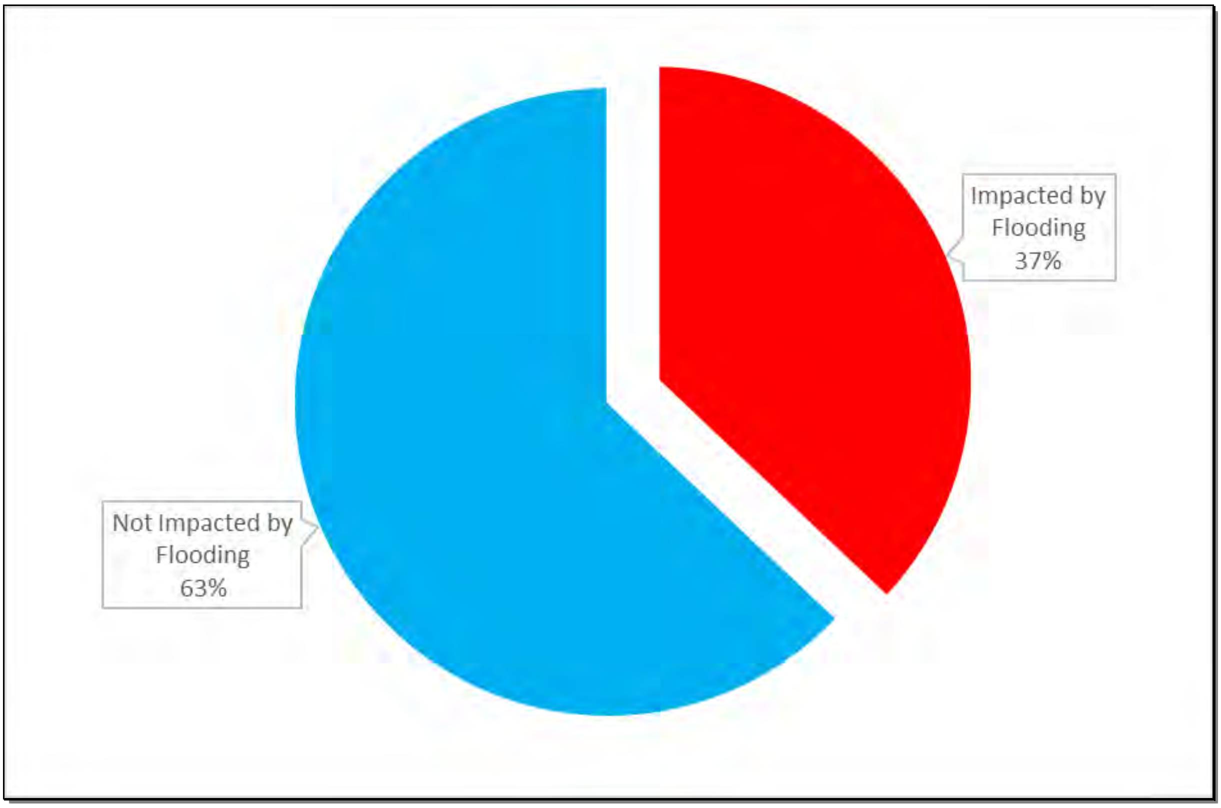


Plate 12 Number of Questionnaire Respondents Impacted by Past Floods

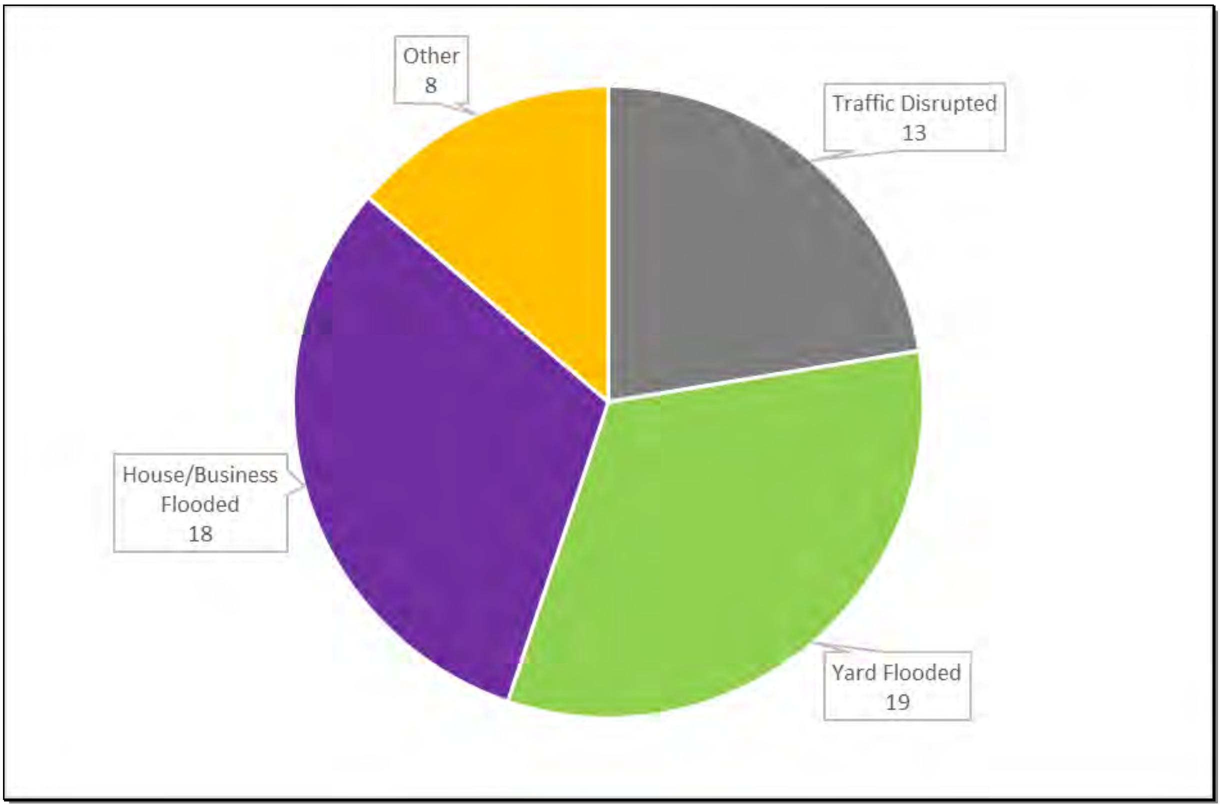


Plate 13 Type of Flood Impact Reported by Questionnaire Respondents

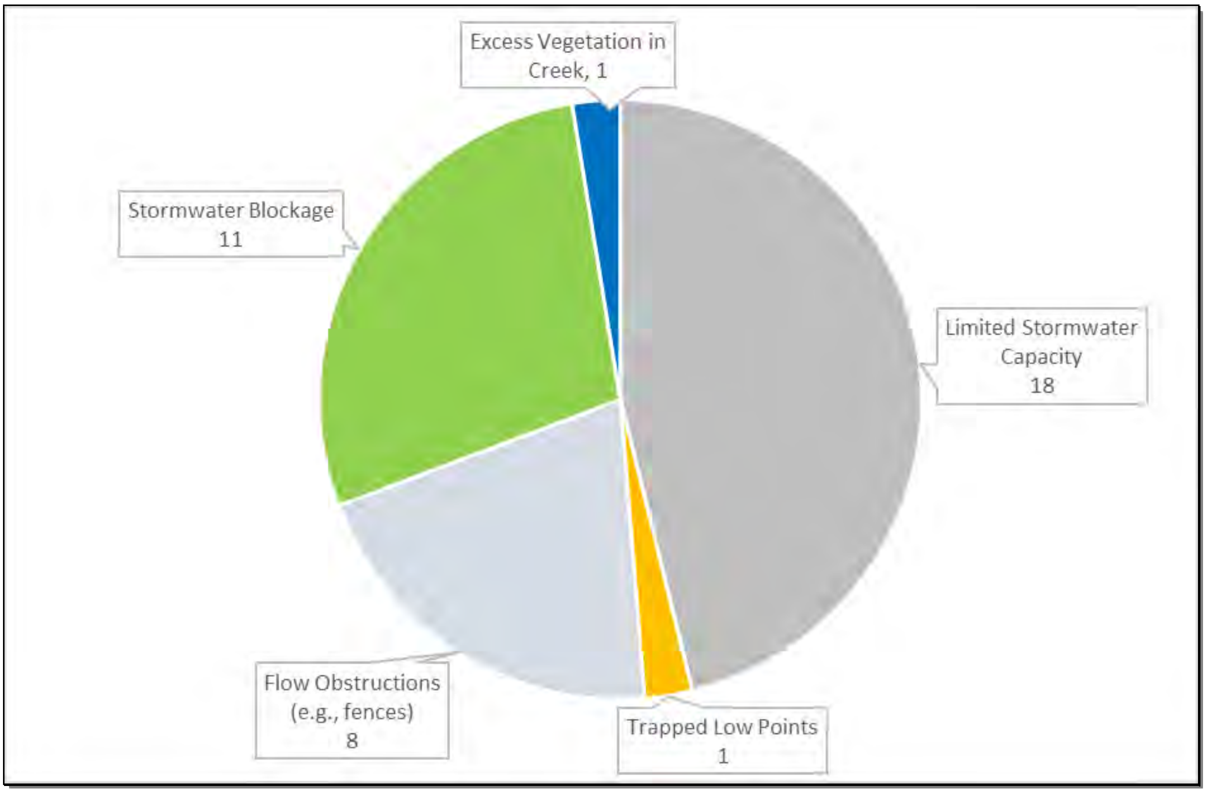


Plate 14 Primary Causes of Flooding Reported by Questionnaire Respondents

A number of respondents provided photos of the 1990 and 2012 floods. A selection of these photographs are provided in **Plate 14** to **Plate 18**. The photos show water extending across a number of front and back yards, around buildings and across roadways. The photos show that the depths of inundation are typically low (i.e., <0.3 metres), although some more significant depths of inundation are evident around Moir Street.

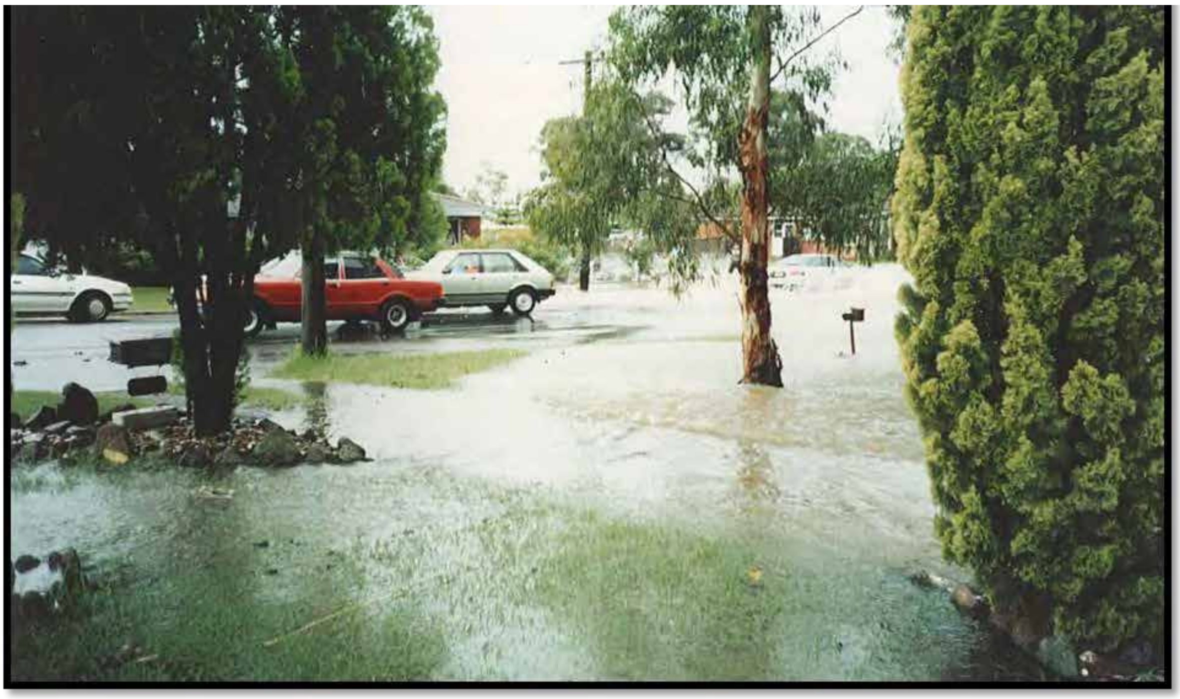


Plate 15 Floodwaters across front yards of 267 and 265 Brenan Street, Smithfield during 1990 flood.

Plate 16 Floodwaters across back yard of 267 Brenan Street, Smithfield during 1990 flood.

Plate 17 Floodwaters looking west along Moir Street from the corner of Moir and Hart Street during the 2012 flood.

Plate 18 Floodwaters looking north toward Moir Street from driveway of 21 Moir Street during the 2012 flood.

The responses to the questionnaires were also analysed to identify historic flood information that could be used to assist in the calibration and verification of the computer model developed for the study. Generally, the flood marks identified from the responses were anecdotal. That is, the flood marks were identified based on the respondent's recollection of how deep floodwaters were during a particular flood. As such, the anecdotal flood marks will generally only provide an approximation of how high a flood reached. There is also a possibility that the observation may not have occurred at the peak of the flood. Accordingly, the reliability of anecdotal flood marks can be questionable.

Nevertheless, the questionnaire responses were analysed and the anecdotal descriptions of historic flood behaviour were converted to historic flood mark elevations through interrogation of the LiDAR information.

A summary of the historic flood marks that were extracted from the questionnaire responses are summarised in **Table 2**. As shown in **Table 2**, two flood marks were extracted from the community responses for the 2012 flood and one flood marks was extracted for the 1990 flood.

Table 2 Historic Flood Marks extracted from Questionnaire Responses

Event	**Location**	**Description**	**Flood Level** *(mAHD)*
1990	76 Dublin Street	1 ft high in backyard	33.00
2012	10 Canara Place	Over 1 metre flowing from Cartela Road to 10 Canara Place	31.15
2012	85 Market Street	20cm – stock loss	22.53

4 COMPUTER FLOOD MODEL

4.1 General

Computer models are the most common method of simulating flood behaviour through a particular area of interest. They can be used to predict flood characteristics such as peak flood level and flow velocity and the results of the modelling can also be used to define the variation in flood hazard.

The TUFLOW software was used to develop a computer model of the Smithfield West Catchment. TUFLOW is a fully dynamic, 1D/2D finite difference model developed by BMT WBM (2012). It is used extensively across Australia to assist in defining flood behaviour.

The following sections describe the model development process, as well as the outcomes of the model calibration.

4.2 Model Development

4.2.1 2D Model Extent and Grid Size

A 2-dimensional computer model of the Smithfield West Catchment was developed using the TUFLOW software (version 2013-12-AD). The extent of the model area is shown in **Figure 4**. As discussed, Prospect Creek forms the downstream boundary of the Smithfield West catchment. However, as shown in **Figure 4**, the TUFLOW model was extended to also incorporate Prospect Creek. This was done to ensure the interaction between local catchment flows, flows along Prospect Creek and flows from the Holroyd City Council LGA were represented.

The TUFLOW software uses a grid to define the spatial variation in topography and hydrologic/hydraulic properties (e.g., Manning's 'n' roughness, rainfall losses) across the study area. Accordingly, the choice of grid size can have a significant impact on the performance of the model. In general, a smaller grid size will provide a more detailed and reliable representation of flood behaviour relative to a larger grid size. However, a smaller grid size will take longer to perform all of the necessary hydraulic calculations. Therefore, it is typically necessary to select a grid size that makes an appropriate compromise between the level of detail provided by the model and the associated computational time required. A grid size of 2 metres was ultimately adopted and was considered to provide a reasonable compromise between reliability and simulation time.

Elevations were assigned to grid cells within the TUFLOW model based on the Digital Elevation Model derived from LiDAR data and ground survey for areas not accurately defined by LIDAR. As the LiDAR data was collected in 2013, it was considered to provide a reliable representation of contemporary topographic conditions across the study area.

4.2.2 1D Domain

A dynamically linked 1-dimensional (1D) network was embedded within the 2D domain to represent the drainage channel within Market Gardens. The culvert located within Market Gardens was also represented as part of the 1D network. The flow carrying capacity of the drainage channel was defined using the surveyed cross-sections gathered by Council (refer Section 3.5.4).

As Prospect Creek forms the downstream boundary of the Smithfield West catchment, it was considered important to include a representation of Prospect Creek in the TUFLOW model. Therefore, Prospect Creek was included as an additional 1D domain embedded within the 2D domain. The creek conveyance characteristics were defined using cross-section information extracted from the TUFLOW model prepared for the *"Prospect Creek Floodplain Management Plan - Flood Study Review"* (Bewsher Consulting, 2006).

The extent of the 1D domains is shown in **Figure 4**.

4.2.3 Material Types

The TUFLOW software employs material polygons to define the variation in hydrologic (i.e., rainfall losses) and hydraulic (i.e., Manning's 'n') properties across the study area. The material polygons for this study were developed using an automated remote sensing approach that takes advantage of the full range of information collected by LiDAR, particularly multiple returns, LiDAR intensity as well as aerial imagery (Ryan, 2013).

The automated approach provides a detailed spatial description (i.e., 1m grid size) of the variation in materials/land use across the catchment. However, there were several misclassifications that were identified. These are primarily associated with shadowing effects and occasional misclassification of buildings. Therefore, some manual updates to the remote sensing outputs were completed to ensure a reliable description of material types was provided across the study area.

The spatial distribution of the different material types is shown in **Figure 4**. As shown in **Figure 4**, the study area was subdivided into eight different material types:

- Buildings;
- Roads;
- Trees;
- Water;
- Grass;
- Shrubs / Long Grass;
- Concrete;
- Crops (i.e., Market Gardens area);

4.2.4 Rainfall Losses

During a typical rainfall event, not all of the rain falling on a catchment is converted to runoff. Some of the rainfall may be intercepted and stored by vegetation, some may be stored in small depression areas and some may infiltrate into the underlying soils.

To account for rainfall "losses" of this nature, the TUFLOW model incorporates a rainfall loss model. For this study, the "Initial-Continuing" loss model was adopted, which is recommended in 'Australian Rainfall and Runoff – A Guide to Flood Estimation' (Engineers Australia, 1987) for eastern NSW.

This loss model assumes that a specified amount of rainfall is lost during the initial saturation/wetting of the catchment (referred to as the "Initial Loss"). Further losses are applied at a constant rate to simulate infiltration/interception once the catchment is saturated (referred to as the "Continuing Loss Rate"). The initial and continuing losses are effectively deducted from the total rainfall over the catchment, leaving the residual rainfall to be distributed across the catchment as runoff.

The catchment includes extensive urban areas that are relatively impervious. Urbanisation effectively separates the catchment into two hydrologic systems, i.e:

- rapid rainfall response and low rainfall losses for impervious areas; and,
- slower rainfall response and high rainfall losses for pervious areas.

In recognition of the differing characteristics of the two hydrologic systems, the rainfall losses were varied spatially based on the different material types / land uses across the model area. Initial and continuing losses were applied to each material type based on design values documented in 'Australian Rainfall and Runoff – A Guide to Flood Estimation' (Engineers Australia, 1987) and are summarised in **Table 3**. The initial losses applied to pervious surfaces was adjusted as part of the TUFLOW model calibration to reflect antecedent conditions (based on consideration of rainfall in the days leading to the main event).

Table 3 Rainfall Loss Values

Material Description	**Rainfall Losses**	
	Initial Loss *(mm)#*	**Continuing Loss Rate** *(mm/hr)*
Grass	10-20	2.5
Trees	10-20	2.5
Shrubs	10-20	2.5
Roads	1	0.0
Concrete	1	0.0
Water	0	0.0
Buildings	1	0.0
Crops	10-20	2.5

NOTE: #: Initial rainfall losses were varied for each calibration event based on antecedent conditions. Further information on the specific rainfall losses that were adopted for each calibration event is provided in Section 5.2.4 and 5.3.4. An initial loss of 10mm was adopted for all design flood simulations.

4.2.5 Manning's 'n' Roughness Coefficients

Manning's 'n' is an empirically derived coefficient that is used to define the resistance to flow (i.e., roughness) afforded by different material types / land uses. It is one of the key input parameters used in the development of the TUFLOW model.

Manning's 'n' values are dependent on a number of factors including vegetation type/density, topographic irregularities and flow obstructions. All of these factors are typically aggregated into a single Manning's 'n' value for each material type, representative Manning's 'n' values can be obtained from literature (e.g., Chow, 1959). However, the Manning's 'n' values found in literature are only valid when the flow depth is large relative to the material/vegetation height and the material is rigid.

When using a "direct rainfall" computer model, the depth of flow across much of the study area will be shallow (often referred to as "sheet" flow). In such instances, the depth of flow can be equal to or less than the height of the vegetation and the vegetation is not necessarily rigid (e.g., grass can bend under the force of flowing water). Accordingly, Manning's 'n' values obtained from literature are generally no longer valid for shallow flow depths.

Research completed by McCarten (2011) and others (Engineers Australia, 2012) indicates that Manning's 'n' values will not be "static" and will vary with flow regime/depth. Specifically, the research indicates that Manning's' 'n' values will typically decrease with increasing flow depths. This is associated with the resistance to flow at higher depths being driven by bed resistance only, while at shallow depths, the resistance is driven by vegetation/stem drag as well as bed resistance (i.e., the "effective" roughness is higher at shallow depths).

In an effort to represent the depth dependence of Manning's 'n' values in the TUFLOW model, flow depth versus Manning's 'n' relationships were developed for each material type. The relationships were developed using the modified Cowan method, which is documented in the USGS water supply paper 2339 titled 'Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains' (Arcement & Schneider). The modified Cowan method was selected as it allows the Manning's 'n' values to be calculated based on the depth of the flow relative to the vegetation/obstruction height. The Manning's 'n' calculations are included in **Appendix C** and the final Manning's 'n' values for each material type at each depth are summarised in **Table 4**.

Table 4 Manning's 'n' Roughness Values

Material Description	Depth Varying Manning's 'n' Values							
	$Depth_1$ *(metres)*	n_1	$Depth_2$ *(metres)*	n_2	$Depth_3$ *(metres)*	n_3	$Depth_4$ *(metres)*	n_4
Grass	<0.03	0.110	0.05	0.075	0.07	0.055	>0.10	0.030
Trees	<0.30	0.160	1.50	0.110	>2.00	0.080		
Shrubs	<0.30	0.137	1.00	0.077	>1.50	0.047		
Roads	<0.04	0.017	0.10	0.021	>0.15	0.020		
Concrete	<0.005	0.034	>0.005	0.015				
Crops	<0.10	0.133	0.50	0.093	>0.50	0.059		
Buildings	<0.03	0.030	>1.0	1.000				
Water	0.013 for all depths							

4.2.6 Building Representation

The Smithfield West catchment is highly urbanised. The high level of urbanisation across the study area creates many flow obstructions. One of the most significant impediments to overland flow in urban environments is buildings. Available research indicates that buildings have a considerable influence on flow behaviour in urban environments by significantly deflecting flows irrespective of whether the building is flooded inside or remains water tight (Smith et al, 2012). Accordingly, it was considered necessary to include a representation of the buildings in the computer model.

The lower part (i.e., the area between the ground surface and the floor level) of each building located within major overland flow paths was represented as a complete flow obstruction. This is shown conceptually in **Plate 19**. This was implemented by elevating all TUFLOW elevations contained within the building footprint to the floor level of the building. The floor level of each building was defined based on surveyed floor elevations (where available) or were estimated using a "drive by" survey. Further detailed information on the floor level estimation technique is provided in Section 9.2.

Plate 19 Conceptual representation of buildings in TUFLOW model

Once the water level exceeded the floor level of each building, it was allowed to "enter" the building. However, a high Manning's "n" value of 1.0 was adopted to reflect the significant impediment to flow afforded by the many flow obstructions contained with a typical house (e.g., walls, furniture etc). This is also shown conceptually in **Plate 19**.

Other significant structures (e.g., garages, large sheds) were also included in the TUFLOW model. However, the floor levels of these structures were retained at or near the natural ground surface and only the elevated Manning's "n" value was applied to represent the impediment to overland flow.

Plate 20 provides an example of floodwater depth and velocity vectors in the vicinity of buildings (the velocity vectors show the direction and speed of the floodwaters). The velocity vectors show that the raised building footprints provide an appropriate representation of the redirection of flow around buildings at shallow water depths. It also provides a good representation of the increase in flow velocities as water is “squeezed” between buildings, thereby helping to ensure the flood hazard is suitably quantified.

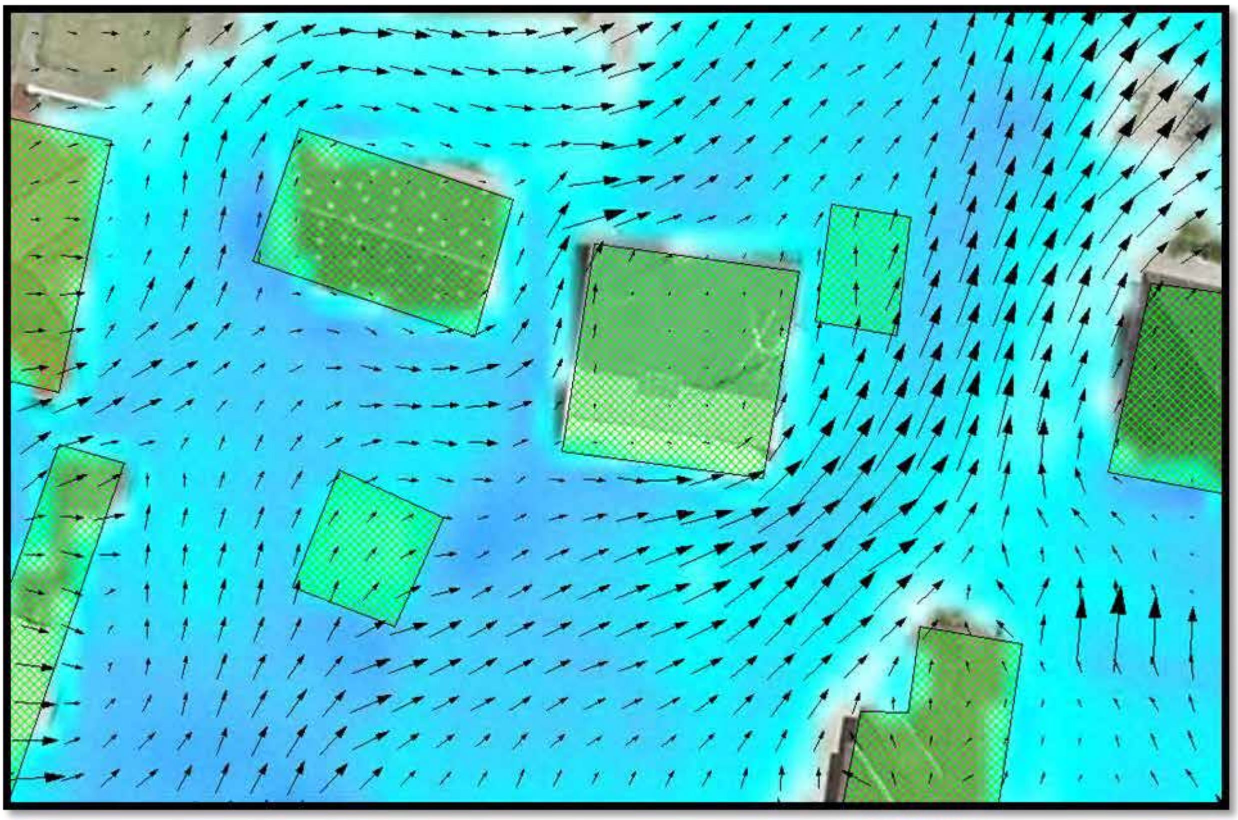


Plate 20 Example of building representation in the vicinity of 24 to 30 Moir Street, Smithfield. Velocity vector arrows (black) show that the elevated building footprints force the water to move around and between buildings

4.2.7 Stormwater System

The stormwater system has the potential to convey a significant proportion of runoff across the study area during relatively frequent rainfall events. Therefore, it was considered important to incorporate the conveyance provided by the stormwater system in the TUFLOW model to ensure the interaction between piped stormwater and overland flows was represented.

The full stormwater system was included within the TUFLOW models as a dynamically linked 1-Dimensional (1D) network. This allowed representation of the conveyance of flows by the stormwater system below ground as well as simulation of overland flows in two dimensions once the capacity of the stormwater system is exceeded.

As discussed in Section 3.5, major stormwater pits and pipes were surveyed in 2010 and 2014 and this information was provided for use as part of the study. This survey information provided a detailed description of the key attributes of major stormwater pits and pipes allowing these stormwater components to be directly included in the TUFLOW model.

Council’s stormwater GIS layer was provided to supplement the surveyed stormwater information. However, as discussed in Section 3.6, some stormwater pits were incorrectly positioned, omitted, or the pit type was not correctly identified in the GIS layer. As a result, the GIS layers did not contain all of the information necessary to fully define the stormwater system in TUFLOW.

Therefore, the missing pit and pipe GIS information was estimated to ensure all required information describing the stormwater system was represented. The missing pipe information was estimated using the following approach:

- Where pipe diameter information was not available, the diameter was interpolated based upon inspection of the upstream and downstream pipe diameters;
- Where pit/pipe locations did not agree / connect with surveyed data, the pits and pipes were manually adjusted;

The missing GIS pit information was populated using the following approach:

- Pit locations were adjusted based upon the 2014 aerial imagery so they were located in the appropriate location.
- All stormwater pits without a type classification were inspected using Google Street View, and the grate and lintel sizes were estimated. This resulted in over 140 new pit type classifications being included in the stormwater system representation.
- Pit invert elevations were linearly interpolated between surveyed pit locations. Where surveyed pit information was not available to assist the interpolation, the pit inverts were estimated using the following equation:
  -> Invert elevation = Ground elevation – 0.5m (cover) – Pipe diameter

A copy of the stormwater pit and pipe database that was developed is provided in **Appendix D**.

Once the all pit types were defined across the catchment, inlet capacity curves were prepared to define the variation in pit inflow capacity with respect to water depth at each pit location. The 'Drains Generic Pit Spreadsheet' (Watercom Pty Ltd, July 2005), was used to develop the inlet capacity curves. The inlet capacity curves were developed to take account of:

- The different pit inlet types (e.g., sag inlets, grated inlets, kerb inlets, combination inlets); and,
- The different pit dimensions and lintel sizes.

A copy of the inlet capacity curves are provided in **Appendix E**.

The extent of the stormwater system included within the TUFLOW models is shown in **Figure 2**. Those pits/pipes defined based upon surveyed information are shown in green and blue. Those pits/pipes that were based upon Council's GIS layer are shown in purple. As shown in **Figure 2**, the surveyed stormwater information covers the main trunk drainage system and the GIS based estimations are generally restricted to smaller diameter "branches".

Culvert and Stormwater Blockage

During most floods, sediment, vegetation and urban debris (e.g., litter, shopping trolleys) from the catchment can become mobilised leading to blockage of culverts and stormwater inlets (refer **Plate 21**). Consequently, these drainage structures will typically not operate at full efficiency during most floods. This can increase the severity of flooding across areas located adjacent to these structures.

Plate 21 View showing partial blockage of a stormwater pit located on Chifley Street, Smithfield

In recognition of this, blockage factors were applied to all structures. The blockage factors were based on the latest available structure blockage information contained in 'Project 11: Blockage of Hydraulic Structures' (Engineers Australia, 2013) as well as information provided by Fairfield City Council.

However, it is very difficult to know the extent of blockage that each structure is going to be subjected to during a particular flood. Therefore, three different blockage scenarios were considered as part of each calibration simulation to reflect the potential variability in structure blockage:

- No Blockage;
- 30% Blockage; and,
- 50% Blockage.

The blockage factors listed above were applied to all stormwater pits as well as culverts within the Smithfield West catchment. Further information on the blockage factors that were applied as part of the design flood simulations is included in Section 6.2.2.

4.2.8 Fences

Fences can also provide a significant impediment to flow in urbanised catchments. Therefore, it was also considered important to include a representation of fences within the TUFLOW model. An automated approach was employed to extract approximate fence alignments across the study area based on information contained in cadastre, roadway and LEP GIS layers.

The fence alignments were then reviewed relative to the 2014 aerial imagery and adjustments to the fence lines were completed by hand, where necessary, to ensure a reliable representation of fences was provided across the study area.

The impediment to flow afforded by fences is influenced by two main factors:

- Fence type; and,
- Debris accumulation on fence.

The large array of fence types and debris blockage potential means that there is likely to be considerable variability in the overall blockage provided by different fence types. Although it can be difficult to quantify the variation in debris accumulation potential across the catchment, the types of fences are more readily identifiable. Therefore, fence types along major overland flow paths were identified as part of the study through field inspections and Google Street View. Specifically, a preliminary 1% AEP simulation was completed and all fences exposed to a water depth of greater than 0.15 metres were included in the fence type delineation. This resulted in fences along major overland flow paths being classified according to one of six fence types. The extent of the different fence types delineated using this approach is shown in **Plate 22**.

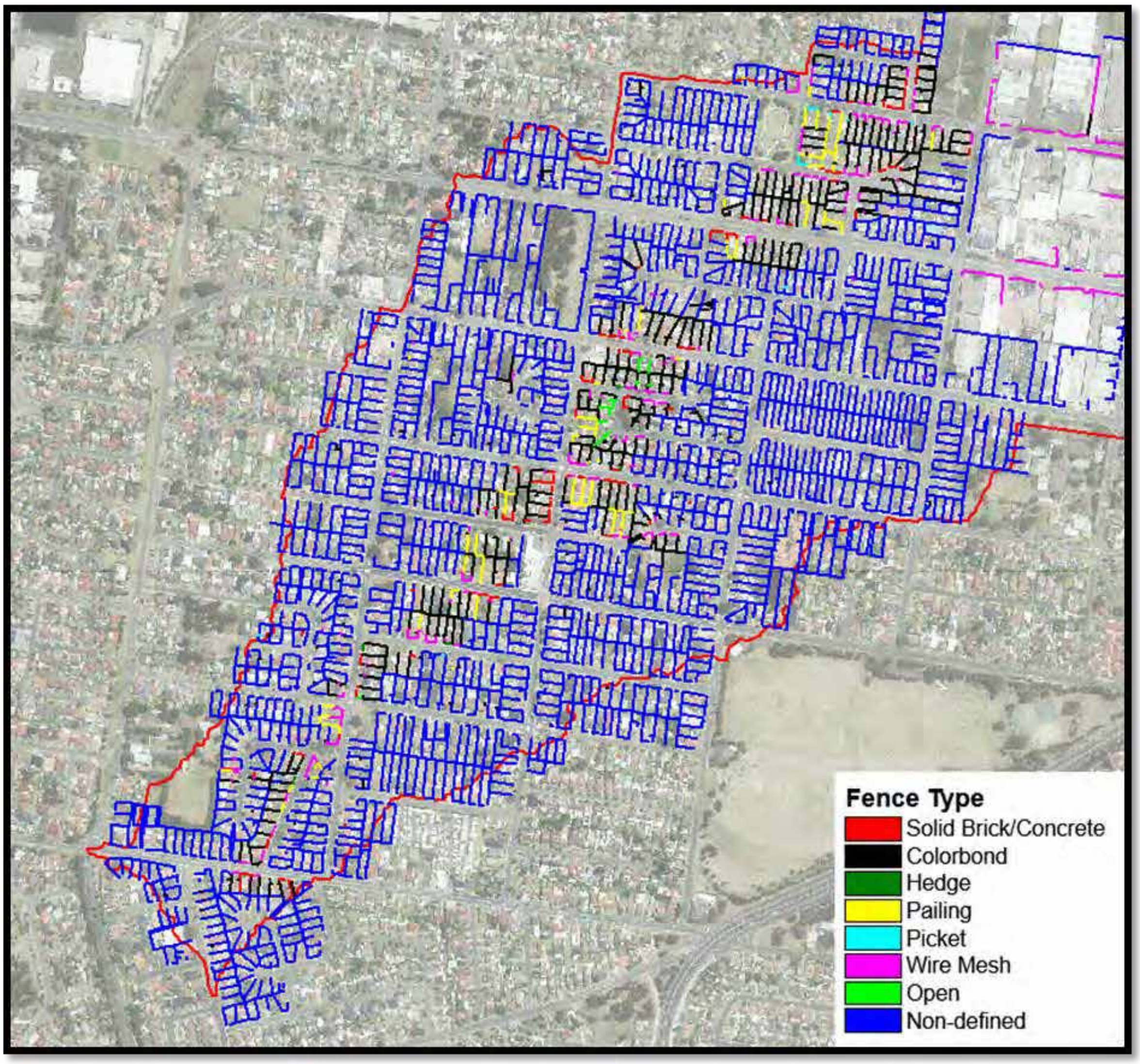


Plate 22 Extent and type of fences included in the TUFLOW model

Plate 22 shows that although there are a variety of fence types located along overland flow paths, the most common fence type is Colorbond^{TM}.

Those fences located outside of major overland flow paths were delineated as “non-defined” fences. These fences are shown in blue in **Plate 22** and were not explicitly categorised as part of the study.

The fences were subsequently included in the TUFLOW model as a “flow constriction” line. This representation allows a blockage factor to be applied to each cell located beneath a fence line to reflect the impediment to flow / reduced conveyance capacity through fences.

Unfortunately, there is little information available describing the blockage afforded by different fence types. The Australian Rainfall & Runoff ‘Project 11: Blockage of Hydraulic Structures’ (Engineers Australia, 2013) suggests that blockage factors of between 50% and 100% would typically be appropriate for fences located in overland flow paths. However, it does not provide fence type specific blockage information.

Therefore, professional judgement was used as the basis for assigning representative blockage factors to each fence type. The adopted blockage factors are summarised in **Table 5**. As shown in **Table 5**, the majority of the adopted blockage factors fall within the 50% to 100% range suggested in the Project 11 document.

Table 5 Adopted Blockage Factors for Various Fence Types in the Smithfield West Catchment

Fence Type	Blockage Factor
Colorbond	90%
Solid Brick/Concrete	100%
Pailing	75%
Wire Mesh	25%
Open	10% (depth <0.4m), 90% (0.4m<depth<1.0m), 0% (depth>1m)
Hedge	80%
Picket Fence	50%
Non-defined	50%

As discussed in Section 3.2.3, open fencing has been installed along major overland flow paths within the Smithfield West catchment in an effort to reduce the severity of overland flooding. As shown in **Plate 23**, the degree of blockage afforded by these fence types will vary considerably with depth. As a result, it was not considered appropriate to adopt a single blockage factor for open fence types. Therefore, the open fences were included in the TUFLOW model as a “layered flow constriction” layer. This allows the blockage factors to be varied with respect to water depth/height, allowing a more realistic representation of the depth variation in blockage to be provided in the model. The adopted blockage values for the open fencing is also summarised in **Table 5**.

It was assumed that all fences were 1 metre high in the TUFLOW model. That is, the blockage factors summarised in **Table 5** were applied for all water depths up to 1 metre. Any water exceeding a water depth of 1 metre was assumed to “overtop” the fence and no blockage was applied. Although it was acknowledged that fences can often exceed 1 metre in height, most fence types will fail once the water depth exceeds 1 metre. As a result, the 1 metre fence height was considered to provide a reasonable "upper limit" of the degree of blockage that can be provided by an average fence without failing.

Plate 23 Example of "open" fencing in Cartela Crescent, Smithfield West

5 COMPUTER MODEL CALIBRATION

5.1 Overview

Computer flood models are approximations of a very complex process and are generally developed using parameters that are not known with a high degree of certainty and/or are subject to natural variability. This includes catchment roughness/vegetation density as well as blockage of hydraulic structures. Accordingly, the model should be calibrated using flow and flood mark information from historic floods to ensure the adopted model parameters are producing reliable estimates of flood behaviour.

Calibration is typically completed by routing recorded rainfall from historic floods through a computer model. Simulated flows and flood levels are extracted from the model results at locations where recorded data are available. Calibration is completed by iteratively adjusting the model parameters within reasonable bounds to achieve the best possible match between simulated and recorded flood flows and flood marks.

Unfortunately, there are no stream gauges located within the study area. Therefore, it is not possible to complete a full calibration of the computer model developed for this study.

However, historic flood marks are available for the 1990 and 2012 events. Therefore, it is possible to complete a 'pseudo-calibration' by routing historic rainfall through the model and comparing simulated water levels against recorded flood mark elevations for these floods.

Verification of the computer model can also be completed by trying to reproduce recorded flood mark elevations for additional historic floods using the same model parameters that were adopted for the calibration simulations. Unfortunately there was an insufficient number of historic floods with associated flood mark information to complete a separate verification simulation. Accordingly, a focus was placed on calibrating the TUFLOW model against the 1990 and 2012 floods. Further details of the TUFLOW model calibration process are provided below.

5.2 February 1990 Flood

5.2.1 Local Catchment Rainfall

The February 1990 flood occurred over a 3 hour period on 10 February 1990. It caused damage to a number of properties and was considered to be a major event close to a 100 year ARI storm (Dalland & Lucas, 1996).

Accumulated daily rainfall totals for each rainfall gauge that was operational during the 1990 event were used to develop a rainfall isohyet map for the 1990 event, which is shown in **Figure 5**. The isohyet map shows that around 108 mm of rain fall across the catchment within a 24 hour period. As there was minimal spatial variation in rainfall during the 1990 event, a uniform rainfall depth of 108 mm was applied to the TUFLOW model.

The temporal (i.e, time-varying) distribution of rainfall was applied based on the closest continuous rainfall gauge. The closest continuous gauge was determined to be the Parramatta North (Masons Drive) gauge (Gauge #66124), which is located approximately 10 kilometres north-east of the Smithfield West Catchment. The location of the gauge is shown in **Figure 3**. A review of the continuous rainfall information indicates that the majority of the rainfall during this event occurred over a 1 hour period.

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Appendix F** and indicates that the 1990 rainfall was close to but slightly more severe that a 1% AEP flood event.

5.2.2 Downstream Boundary Conditions

Hydraulic computer models also require the adoption of a suitable downstream boundary condition in order to reliably define flood behaviour throughout the area of interest. The downstream boundary is typically defined as a known water surface elevation (i.e., stage). The downstream boundary of the Smithfield West catchment coincides with Prospect Creek. Accordingly, the water level across the downstream reaches of the Smithfield West catchment will be driven by the prevailing water level along Prospect Creek at the time of the flood.

To help ensure the variation in flood levels along Prospect Creek was reliably defined, the full length of Prospect Creek that adjoins the Smithfield West catchment was incorporated in the TUFLOW model. The Prospect Creek channel was defined using cross-section information extracted from the TUFLOW model developed / updated as part of the *"Prospect Creek Floodplain Management Plan – Flood Study Review"* (Bewsher Consulting, 2006).

At the downstream end of Prospect Creek a "normal depth" (i.e., Manning's) boundary condition was applied. The normal depth boundary automatically calculates a water level based upon the amount of water travelling across the downstream model boundary and the characteristics of the channel at that location (i.e., channel geometry, slope and roughness). The location of the normal depth boundary condition is shown in **Figure 4**. As shown in **Figure 4**, the TUFLOW model boundary is located approximately 200 metres downstream of the Smithfield West catchment. Therefore, any uncertainties associated with this boundary definition should not impact on results across the Smithfield West catchment (this was subsequently confirmed as part of the sensitivity analysis).

In addition to defining a normal depth boundary condition at the downstream end of Prospect Creek, it is also necessary to define flows entering the upstream end of Prospect Creek as well as flows from the local Holroyd City Council LGA subcatchment located on the northern side of Prospect Creek. Therefore, two inflow boundary conditions were also included in the TUFLOW model to represent inflows along Prospect Creek as well as from the Holroyd City Council LGA subcatchment. The location of the inflow boundary conditions is also shown in **Figure 4**.

The inflow hydrographs for Prospect Creek and the Holroyd City Council LGA were defined using the XP-RAFTS hydrologic model that was updated as part of the *"Prospect Creek Floodplain Management Plan - Flood Study Review"* (Bewsher Consulting, 2006). The XP-

RAFTS model was updated to include a representative description of the 1990 rainfall across the Prospect Creek catchment. This was based on the isohyet map shown in **Figure 5**. The temporal variation in this rainfall was based upon Parramatta North (Masons Drive) gauge (i.e., the same gauge that was used to describe the temporal variation in rainfall across the Smithfield West catchment).

5.2.3 Modifications to Represent Historic Conditions

As the February 1990 flood occurred over 20 years ago, there have been some changes in development/drainage conditions across the Smithfield West catchment. In an attempt to provide a model that was representative of conditions in 1990, the TUFLOW model that was developed to represent "contemporary" conditions was modified in an attempt to reflect historic conditions.

Specifically, 1995 aerial imagery was acquired and was used as the basis for modifying material types and building polygons across the study area so it was more representative of development conditions in 1990.

In addition, the "open fences" discussed in Section 4.2.8 were installed after the 1990 flood. Therefore, these fences were removed and replaced with colorbond^{TM} type fences.

Finally, Council constructed new stormwater pits and pipes along Bourke Street and Moir Street in 2009. Therefore, these pipes and pits were also removed from the TUFLOW model so the drainage system representation more reliably reflected 1990 conditions.

5.2.4 Antecedent Catchment Conditions

A review of rainfall records was completed to ensure a reliable representation of antecedent catchment conditions was provided before undertaking the 1990 flood simulation. This review determined that over 350 mm of rain fell in the vicinity of Smithfield West in the previous 7 days. As a result, the catchment would likely have been saturated before the main rainfall event. *"Australian Rainfall and Runoff – A Guide to Flood Estimation"* (Engineers Australia, 1987) recommends initial rainfall losses of between 10 mm and 30 mm. As a significant amount of rainfall preceded the 1990 event, an initial loss at the lower end of the suggested range (i.e., 10 mm) was adopted for pervious surfaces of the catchment.

5.2.5 Results

Calibration of the TUFLOW hydraulic model was attempted based upon sixteen (16) flood marks for the February 1990 flood. The calibration was undertaken by routing the historic rainfall and inflows through the TUFLOW model and adjusting model parameter values until a reasonable agreement between simulated flood levels and recorded flood marks was achieved.

As discussed in Section 4.2.7, there is considerable uncertainty associated with blockage of hydraulic structures. Therefore, it is difficult to know what drainage structures were subject to what proportion of blockage during the 1990 flood. Therefore, the 1990 flood was simulated with 3 different blockage scenarios in the hope that one of these scenarios would approximate the blockage conditions in 1990.

Peak floodwater depths and velocity vectors were extracted from the results of the no blockage simulation and are included on **Figure 6**. It should be noted that only water depths greater than 0.15 metres are shown in **Figure 6**. Further information on the display of the modelling results and the filtering that has been completed to the raw modelling results is provided in Section 6.3.3.

A comparison between the peak flood levels generated by the TUFLOW model under "no blockage" conditions and the recorded flood mark elevations for the 1990 flood is also provided in **Figure 6**. A comparison between recorded flood mark elevations and simulated flood levels for each blockage scenario is also presented in **Table 6**.

Table 6 Comparison between simulated flood levels and recorded flood marks for 1990 flood

Location	Recorded Flood Mark Elevation (mAHD)	Simulated Flood Level (mAHD)			Difference (m)		
		No Blockage	30% Blockage	50% Blockage	No Blockage	30% Blockage	50% Blockage
76 Dublin St	33.10	33.08	33.08	33.08	-0.02	-0.02	-0.02
823 The Horsley Dr	30.47	30.54	30.54	30.54	0.07	0.07	0.07
32 Moir St	30.05	30.02	30.02	30.02	-0.03	-0.03	-0.03
27 Moir St	29.13	29.09	29.09	29.09	-0.04	-0.04	-0.04
10 Hart St	28.98	28.92	28.92	28.92	-0.06	-0.06	-0.06
19 Moir St	28.94	28.92	28.92	28.92	-0.02	-0.02	-0.02
21 Moir St	28.89	28.87	28.87	28.86	-0.02	-0.02	-0.03
142 Victoria St	27.63	27.66	27.67	27.67	0.03	0.04	0.04
30 Hinkler St	27.01	26.94	26.95	26.94	-0.07	-0.06	-0.07
38 Hinkler St (back yard)	26.43	26.42	26.42	26.42	-0.01	-0.01	-0.01
1 Kingsford St	26.12	26.14	26.14	26.14	0.02	0.02	0.02
38 Hinkler St (front yard)	25.70	25.77	25.78	25.78	0.07	0.08	0.08
31 Hinkler St	25.41	25.42	25.43	25.43	0.01	0.02	0.02
39 Hinker St	25.32	25.30	25.30	25.30	-0.02	-0.02	-0.02
66 Chifley St	23.93	24.00	24.00	24.00	0.07	0.07	0.07
49 Rhondda St	33.10	33.08	33.08	33.08	-0.02	-0.02	-0.02
				Average	-0.003	0.000	-0.001
				Standard Deviation	0.043	0.044	0.045

The flood level comparison provided in **Table 6** shows that the TUFLOW model provides a reasonable reproduction of recorded flood mark elevations. It also shows that the three different blockage scenarios produced similar peak flood level results at each flood mark location. This is likely associated with the stormwater pipe system being "fully charged" during the 1990 flood by the time it reaches the most seriously impacted areas of the catchment. As a result, no additional flow can "fit" into the stormwater system across these areas regardless of the blockage.

The "no blockage" scenario reproduces all historic flood mark elevations to within 0.07 metres. **Table 6** also shows that the average difference between simulated and recorded flood levels for the no blockage scenario is approximately zero. Accordingly, the outcomes of the 1990 simulation indicates that the TUFLOW model is providing a reasonable reproduction of overland flood behaviour.

However, to ensure the TUFLOW model was also providing a reasonable reproduction of sub-surface drainage (i.e., stormwater) conditions, peak stormwater pipe flows were also verified. The verification was completed by comparing peak pipe flows extracted from the TUFLOW model with pipe flow information contained in the *'Smithfield West Drainage Study: Chifley Street to The Horsley Drive'* (Dalland & Lucas, 1996). The pipe flow information documented in this report was generated by an ILSAX hydraulic model that was used to simulate the 1990 flood. This comparison is provided in **Table 7** and indicates that the TUFLOW produces peak pipe flows that are generally within 10% of the ILSAX model.

Table 7 Comparison between simulated flood levels and recorded flood marks for 1990 flood simulation

Location	Peak Pipe Flow (m^3/s)			
	ILSAX	TUFLOW	Difference	
			(m^3/s)	(%)
The Horsley Drive	5.0	5.1	0.1	2%
Moir Street	5.0	5.2	0.2	4%
Victoria Street	5.3	5.8	0.5	9%
Hinkler Street	6.4	6.5	0.1	2%
Chifley Street	7.6	6.8	-0.8	-11%

Overall, the outcomes of the calibration and verification shows that the TUFLOW model is providing a good reproduction of historic flood mark elevations and peak pipe flow information.

5.3 April 2012 Flood

5.3.1 Local Catchment Rainfall

The April 2012 flood occurred as a result of rainfall a 1.5 hour period starting around 1:00pm on the 18th April 2012. The flood did not cause over floor flooding of any residential buildings within the Smithfield West catchment. However, it did inundate a number of garages / sheds.

Accumulated daily rainfall totals for each rainfall gauge that was operational during the 2012 event were used to develop a rainfall isohyet map for the 2012 event, which is shown in **Figure 7**. The isohyet map indicates that there was only a slight spatial variation in rainfall across the catchment during the 2012 event. It indicates that around 76 mm of rain fell across the catchment during the event. Accordingly, this rainfall depth was applied to the TUFLOW model to represent rainfall over the Smithfield West catchment.

The temporal (i.e, time-varying) distribution of rainfall was applied to the TUFLOW model based on rainfall records for the Prospect Dam gauge (Gauge #567083), which is located about 4 km from the Smithfield West catchment. The location of the gauge is shown on **Figure 3**. The rainfall information for this gauge was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Appendix F** and indicates that the 2012 rainfall was approximately equal to a 10% AEP event.

5.3.2 Downstream Boundary Conditions

A normal depth boundary condition was also adopted for the downstream end of Prospect Creek for the 2012 flood simulation. As discussed in Section 4.3.1, the downstream boundary is located a significant distance downstream of the Smithfield West catchment. As a result, any uncertainties associated with the downstream boundary definition should not impact on results across the Smithfield West catchment.

The inflow hydrographs for Prospect Creek and the Holroyd City Council LGA were defined using the 'Prospect Creek Floodplain Management Plan - Flood Study Review' (Bewsher Consulting, 2006) XP-RAFTS model. The XP-RAFTS model was updated to include a representative description of the 2012 rainfall across the Prospect Creek catchment. This was based on the isohyet map shown in **Figure 7**. The temporal variation in this rainfall was based upon Prospect Dam gauge. This is the same gauge that was used to describe the variation in rainfall across the Smithfield West catchment.

5.3.3 Modifications to Represent Historic Conditions

As the 2012 flood occurred relatively recently, no changes to the existing conditions in the TUFLOW model as described in Section 4.2 were completed for the 2012 event.

5.3.4 Antecedent Catchment Conditions

A review of rainfall records was completed to define antecedent conditions before completing the 2012 simulation. This review determined that less than 20 mm of rain fell in 7 days preceding the event. As a result, the catchment may have been "damp" but would not have been saturated like the 1990 event. Therefore, an initial loss value at the middle of the suggested 'Australian Rainfall and Runoff – A Guide to Flood Estimation' (Engineers Australia, 1987) range (i.e., 20 mm) was initially adopted for pervious surfaces of the catchment. This initial loss was subsequently adjusted as part of the model calibration and a final initial loss value of 17 mm was adopted.

5.3.5 Results

Calibration of the TUFLOW hydraulic model was attempted based upon thirteen (13) flood marks for the April 2012 flood. The calibration was undertaken by routing the historic rainfall through the TUFLOW model and adjusting model parameter values until a reasonable agreement between simulated flood levels and recorded flood marks was achieved. As with the 1990 flood simulation, three different blockage scenarios were simulated to reflect the uncertainty associated in stormwater system blockage.

Peak floodwater depths and velocity vectors were extracted from the results of the "no blockage" 2012 simulation and are included on **Figure 8**. It should be noted that only water depths greater than 0.15 metres are shown in **Figure 8**. Further information on the display of the modelling results and the filtering that has been completed to the raw modelling results is provided in Section 6.3.3.

A comparison between the peak flood levels generated by the TUFLOW model for "no blockage" conditions and the recorded flood mark elevations for the 2012 flood is also provided in **Figure 8**. A comparison between recorded flood mark elevations and simulated flood levels for each blockage scenario is also presented in **Table 8**.

Table 8 Comparison between simulated flood levels and recorded flood marks for 2012 flood simulation

Location	Recorded Flood Mark Elevation (mAHD)	Simulated Flood Level (mAHD)			Difference (m)		
		No Blockage	30% Blockage	50% Blockage	No Blockage	30% Blockage	50% Blockage
819 The Horsley Dr	29.86	29.92	29.92	29.91	0.06	0.06	0.05
26 Moir St	29.18	29.13	29.13	29.13	-0.05	-0.05	-0.05
14 Hart St	28.98	28.82	28.82	28.82	-0.16	-0.16	-0.16
25 Moir St	28.89	28.80	28.80	28.80	-0.09	-0.09	-0.09
12 Hart St	28.81	28.72	28.72	28.72	-0.09	-0.09	-0.09
139B Victoria St	28.56	28.54	28.54	28.54	-0.02	-0.02	-0.02
139B Victoria St	28.49	28.50	28.50	28.50	0.01	0.01	0.01
137C Victoria St	28.19	28.19	28.18	28.18	0.00	-0.01	-0.01
128 Victoria St	27.24	27.19	27.19	27.20	-0.05	-0.05	-0.04
130 Victoria St	27.11	27.11	27.11	27.11	0.00	0.00	0.00
33 Hinkler St	25.16	25.11	25.12	25.13	-0.05	-0.04	-0.03
35 Hinkler St	25.05	25.09	25.09	25.11	0.04	0.04	0.06
85 Market St	22.53	22.36	22.36	22.35	-0.17	-0.17	-0.18
				Average	-0.04	-0.05	-0.04
				Standard Deviation	0.07	0.07	0.07

The flood level comparisons provided in **Table 8** indicate that the TUFLOW model provides a reasonable reproduction of recorded flood mark elevations. The "no blockage" scenario reproduces all historic flood marks to within 0.17 metres. **Table 8** also shows that the average difference between simulated and recorded flood levels for the no blockage scenario is -0.04 metres.

Overall, it was considered that the TUFLOW model is providing a good reproduction of historic flood mark elevations for the 1990 and 2012 floods. As a result of the good reproduction of historic flood marks, it was considered that the TUFLOW model was providing a reliable representation of overland flood behaviour. Moreover, as there have been negligible changes across the catchment since the 2012 flood, it was considered that the TUFLOW model used to simulate the 2012 could also be used to simulate design flood behaviour across the Smithfield West catchment for current (i.e., 2015) conditions.

5.4 Quality Review of TUFLOW Model

As discussed above, the TUFLOW computer model provided a good reproduction of historic flood marks. However, to further ensure that the model was appropriately setup and parameterised, an independent review of the model was completed by BMT WBM (developers of the TUFLOW software).

The review focused on the following components of the TUFLOW model:

- Overall model health (e.g., mass balance, instabilities).
- Model schematisation (e.g., 1D/2D links, stormwater system representation).
- Representation of fences.
- Appropriate choice of model parameters (e.g., Manning's 'n', stormwater/culvert loss coefficients).
- Suitability of boundary conditions.
- Any other standard checks considered necessary.

The outcomes of the review are summarised in **Appendix G**. In general, the review found that it is "...a very well built model with innovative methods of classifying land use and properties". Nevertheless, the review recommended several updates to ensure the model performed as required. A summary of the key recommendations arising from the review are also provided in **Table 9**. **Table 9** also provides a summary of the updates that were completed to the model to address each comment.

Table 9 Summary of Quality Review Comments and Actions

#	Comment	Response / Action
1	Revise buildings code Manning's "n" value	Manning's 'n' assigned to buildings increased from 0.1 to 1.0
2	Define 2d_fcsh level data at points rather than on lines for modelling fences	New fence point layer created and used to assigned elevations to the top of the existing fence lines
3	Consider a design flood envelope of flood levels which includes both "with" and "without" fence scenarios	Additional "No Fences" scenario included in design flood simulations and will be used to develop design flood envelope for each design flood
4	Resolve connectivity and negative slope issues in pipe layers	Connectivity issues and adverse pipe slopes have been rectified and checked
5	Consider extending the Prospect Creek boundary west of Rosford Street Reserve embankment	Upstream model boundary extended slightly to west to Rosford Reserve embankment. The revised model results were reviewed and compared with the previous "Prospect Creek Floodplain Management Plan - Flood Study Review" TUFLOW model and it was found that the updated model provided a conservative estimate of flood behaviour along Prospect Creek. Accordingly, this solution was considered suitable.
6	Demonstrate no sensitivity to auto HQ boundary slope assumptions	Sensitivity analysis results show that the adopted model boundary is extended a sufficient distance downstream to not influence flood behaviour across the study area
7	Initialise model to resolve mass balance errors early in the model simulation	"IWL==19" command added and this appears to have rectified early mass balance issues

6 Design Flood Estimation

6.1 General

Design floods are hypothetical floods that are commonly used for planning and floodplain management investigations. Design floods are based on statistical analysis of rainfall and flood records and are typically defined by their probability of exceedance. This is typically expressed as an Annual Exceedance Probability (AEP).

The AEP of a flood level / depth at a particular location is the probability that the flood level / depth will be equalled or exceeded in any one year. For example, a 1% AEP flood is the best estimate of a flood that has a 1% chance of being equalled or exceeded in any one year.

Design floods can also be expressed by their Average Recurrence Interval (ARI). For example, the 1% AEP flood can also be expressed as a 1 in 100 year ARI flood. That is, the 1% AEP flood will occur, on average, once every 100 years.

It should be noted that there is no guarantee that a 1% AEP flood will occur once in a 100 year period. It may occur more than once, or at no time at all in the 100 year period. This is because design floods are based upon a long-term statistical average. Therefore, it is prudent to understand that the occurrence of recent large floods does not preclude the potential for another large flood to occur in the immediate future.

Design floods are typically estimated by applying design rainfall to the computer model and using the model to route the rainfall excess across the catchment to determine design flood level, depth and velocity estimates. The procedures employed in deriving design flood estimates for the Smithfield West catchment are outlined in the following sections.

6.2 Computer Model Setup

6.2.1 Boundary Conditions

Design Rainfall

Design rainfall for the 50%, 20%, 10%, 5%, 1% and 0.2% AEP events were extracted using standard procedures outlined in *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987). This involved extracting base design intensity-frequency-duration values at the centroid of the Smithfield West catchment from Volume 2 of *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987) (refer **Table 10**).

This base design rainfall information was used to interpolate design rainfall for other design rainfall frequencies and durations. Adopted rainfall intensities for each design storm and duration are summarised in **Table 11**. The resulting intensity-frequency-duration (IFD) curves for the Smithfield West catchment are also provided in **Appendix F**. The resulting design rainfall information was also verified against design rainfall extracted using the Bureau of Meteorology's Computerised Design IFD Rainfall System and was found to be consistent.

Table 10 Design IFD Parameters

Parameter	Value
$^{2}I_{1}$	32.2
$^{2}I_{12}$	6.76
$^{2}I_{72}$	1.99
F2	4.29
F50	15.85

Table 11 Design Rainfall Intensities

DURATION	Rainfall Intensity (mm/hr)						
	50% AEP	20% AEP	5% AEP	1% AEP	0.2% AEP	1 in 10,000 Year ARI	PMP
10 mins	80.1	102.0	130.0	168.0	N/A	N/A	N/A
15 mins	67.1	84.9	108.7	139.7	172.4	254.7	676.0
30 mins	47.5	60.1	76.8	98.5	121.4	181.6	486.0
1 hour	32.2	40.7	52.2	67.1	82.2	128.9	355.0
90 mins	25.2	32.0	41.1	53.0	65.3	106.8	304.0
2 hours	21.1	26.8	34.6	44.7	55.3	92.5	267.5
3 hours	16.3	20.9	27.0	35.0	43.6	74.0	217.0
6 hours	10.5	13.6	17.7	23.1	29.0	49.0	143.8
12 hours	6.8	8.8	11.7	15.4	N/A	N/A	N/A
24 hours	4.3	5.8	7.7	10.3	N/A	N/A	N/A
48 hours	2.7	3.7	5.0	6.8	N/A	N/A	N/A
72 hours	2.0	2.7	3.8	5.2	N/A	N/A	N/A

NOTE: N/A indicates a design rainfall is not available for the nominated storm duration

Design rainfall intensities for the 1 in 10,000 year ARI event were established by interpolating between the 1% AEP and PMP rainfall. Further details on the 1 in 10,000 year ARI rainfall interpolation is provided in **Appendix H**.

For all design storms up to and including the 1 in 10,000 year ARI event, the design rainfall was uniformly distributed across the entire study area. That is, there was no spatial variation in design rainfall across the study area. In addition, due to the small size of the catchment, no areal reduction factors were applied to the rainfall.

The design rainfall estimates were used in conjunction with standard design temporal patterns to describe how the design rainfall varies with respect to time throughout each design storm. The temporal pattern for the 1 in 10,000 year ARI event was based on the standard PMP temporal pattern.

Probable Maximum Precipitation (PMP)

As part of the flood study it was also necessary to define flood characteristics for the Probable Maximum Flood (PMF). The PMF is estimated by routing the Probable Maximum Precipitation (PMP) through the computer model. The PMP is defined as the greatest depth of precipitation that is meteorologically possible at a specific location. Accordingly, it is considered the largest quantity of rainfall that could conceivably fall within a particular catchment.

PMP depths were derived for the Smithfield West catchment for a range of storm durations up to and including the 6 hour event based on procedures set out in the Bureau of Meteorology's *'Generalised Short Duration Method'* (GSDM) (Bureau of Meteorology, 2003). The PMP estimates were varied spatially and temporally based on the GSDM approach before application to the TUFLOW model.

The GSDM PMP calculations are included in **Appendix H**. The PMP rainfall intensities are also included in the intensity-frequency-duration curves provided in **Appendix F**.

Downstream Boundary Conditions

As discussed in Section 4.3.1, the full length of Prospect Creek that adjoins the Smithfield West catchment was included in the TUFLOW model to ensure a reliable description of water levels was provided along the downstream boundary of the Smithfield West catchment. A normal depth boundary condition was defined at the downstream end of Prospect Creek. This was combined with upstream inflows for Prospect Creek as well as the Holroyd LGA that were extracted from the *'Prospect Creek Floodplain Management Plan - Flood Study Review'* (Bewsher Consulting, 2006) XP-RAFTS model. It was assumed that the design rainfall across the broader Prospect Creek catchment commenced at the same time and finished at the same time as rainfall across the local Smithfield West catchment.

To ensure consistency with other flood studies that have been completed across the Fairfield City Council LGA, it was assumed that floods of equivalent severity were occurring across the Smithfield West catchment at the same time as across the broader Prospect Creek catchment during all events up to and including the 1% AEP event. The 1% AEP flood was adopted for Prospect Creek during all Smithfield West events greater than the 1% AEP flood (i.e., 1 in 10,000 year ARI and PMF). A summary of the adopted local catchment / Prospect Creek design flood combinations that were considered as part of the study are provided in **Table 12**.

6.2.2 Hydraulic Structure Blockage

As noted during the model calibration, debris from the catchment can become mobilised during floods leading to blockage of the stormwater system as well as culverts. As a result, the stormwater system and culverts will typically not operate at full efficiency during floods. This can increase the severity of flooding across areas located adjacent to these structures. Therefore, it was considered important to incorporate a representation of structure blockage in the model for the design flood simulations.

Table 12 Adopted Prospect Creek Downstream Boundary Conditions for Design Simulations

Smithfield West Design Flood	Prospect Creek Design Flood			
	50% AEP	20% AEP	5% AEP	1% AEP
50% AEP	×			
20% AEP		×		
5% AEP			×	
1% AEP				×
0.2% AEP				×
1 in 10,000 Year ARI				×
PMF				×

Blockage factors were assigned for each design flood simulation based on information provided by Fairfield City Council. This information is reproduced in **Table 13**. It is understood that the blockage factors listed in **Table 13** have been applied in other similar studies across the Fairfield City Council LGA.

Table 13 Adopted Blockage for Design Flood Simulations

Smithfield West Design Flood	Adopted Stormwater Pit / Culvert Blockage			
	0% Blockage	30% Blockage	50% Blockage	100% Blockage
50% AEP		×		
20% AEP		×		
5% AEP		×		
1% AEP	Sensitivity Analysis		×	Sensitivity Analysis
0.2% AEP			×	
1 in 10,000 Year ARI			×	
PMF			×	

As outlined in **Table 13**, 30% blockage was applied to all stormwater pits and culverts for all design floods up to and including the 5% AEP event. 50% blockage was applied for all events in excess of the 5% AEP event. The impact of no blockage as well as complete blockage of pits and culverts on 1% AEP results was assessed as part of the sensitivity analysis (refer Section 8.2.4).

6.2.3 Fences

As noted in Section 4.2.8, fences can provide a significant impediment to overland flood behaviour. More specifically, relatively "solid" fence types such as brick and colorbond will typically cause floodwaters to "build up" upstream of the fences leading to elevated upstream water levels. However, most fence types are not designed to withstand the hydrodynamic forces associated with overland floodwaters. As a result, fences also have the potential to fail during a flood, which would typically result in increased flooding downstream of fences.

As it is not known which fences will or will not fail during a particular flood, it was considered to complete each design flood simulation with and without fences and combine the results to form the final design "flood envelope". The "with fences" simulations retained the same blockage factors used in the calibration simulations, as outlined in **Table 5**.

6.3 Results

6.3.1 Critical Duration

It was recognised that a single storm duration will not necessarily produce the "worst case" flooding across all sections of the study area. An important outcome of this study was to ensure that the "worst case" flooding conditions were defined across the full catchment. Therefore, the TUFLOW model was used to simulate flood behaviour across the Smithfield West catchment for a range of different durations for each design storm (i.e., 30 minutes up to 6 hours). The results from the 1% AEP design flood simulations were subsequently interrogated to determine the "critical" storm duration or durations across the catchment. The outcomes from this assessment are shown graphically in **Plate 24** and are also tabulated in **Table 14**.

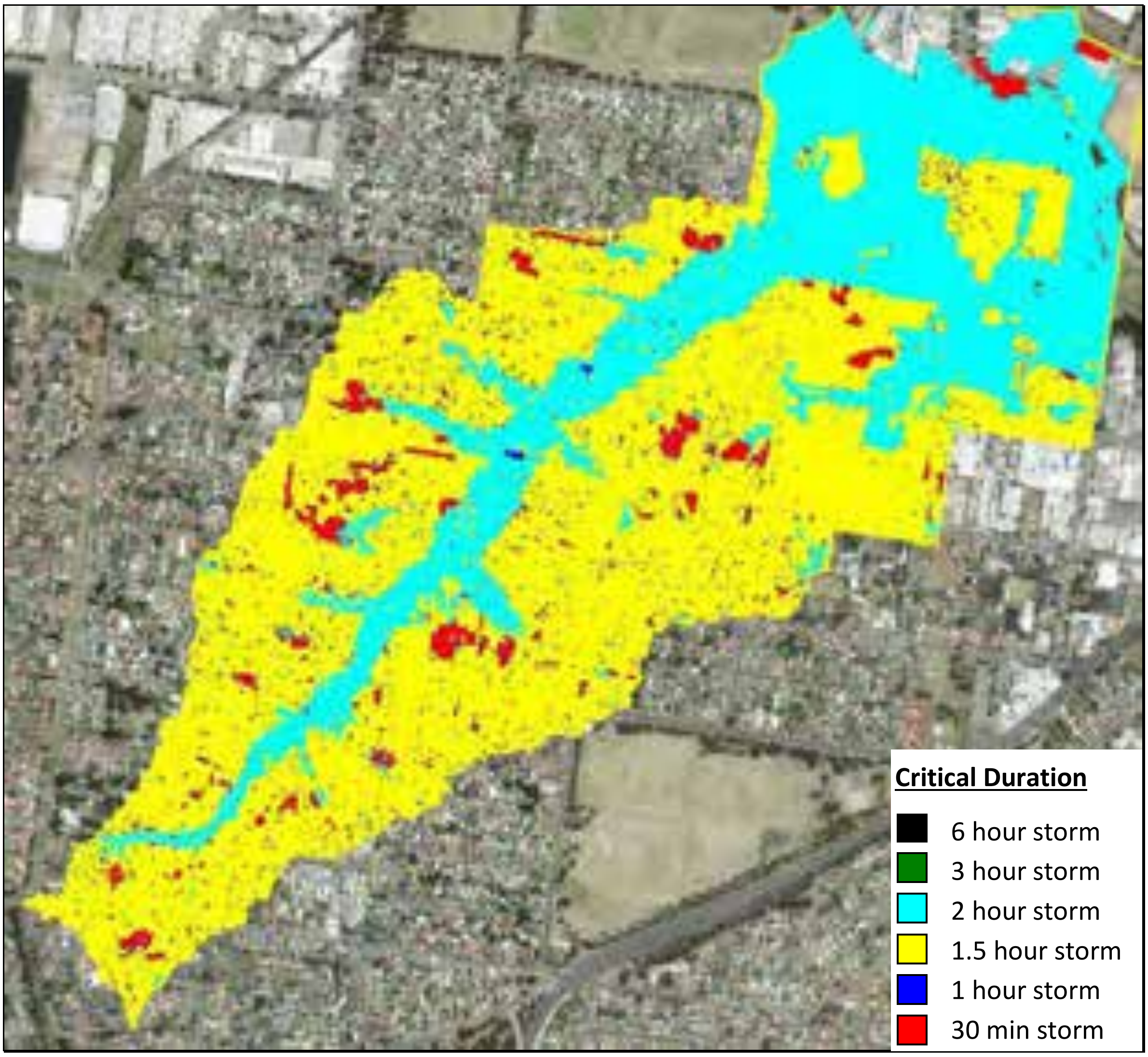


Plate 24 Spatial Variation in Critical Duration for the 1% AEP Storm

The information contained in **Plate 24** shows that the 1.5 hour and 2 hour storm durations produce the highest 1% AEP flood levels across the majority of the catchment. The 1.5 hour storm generally dominates in areas of shallow flow while the 2 hour storm duration dominates along the major overland flow path that runs diagonally through the catchment. This differs slightly from the results documented in the *"Smithfield West Drainage Study: Chifley Street to The Horsley Drive"* (Dalland & Lucas, 1996), which determined that the 1 hour storm was the critical event. The difference may be associated with TUFLOW model including additional "micro-storages" (e.g., behind road embankments, swimming pools) that need to be "filled" before contributing runoff. Consequently, a slightly longer, higher volume storm produces higher peak water levels in the TUFLOW model.

Table 14 Summary of Critical Storm Durations across the Smithfield West catchment

Storm Duration (hours)	Proportion of Catchment Where Storm Duration is Critical	Rank
6	0%	=5
3	0%	=5
2	34%	2
1.5	60%	1
1	2%	4
0.5	4%	3

The 0.5 and 1 hour storms are also critical across small sections of the catchment. The 3 and 6 hour durations were not critical at any location within the catchment. Therefore, they were not represented as part of the design flood simulations.

6.3.2 Design Flood Envelope

As discussed, a range of storm durations were simulated for each design flood to ensure the highest peak flood level was defined across all sections of the catchment. In addition, simulations were completed with and without fences. Consequently, a range of simulations were completed to ensure the worst case flood conditions were represented across all sections of the catchment for each design flood.

Therefore, the results from each simulation for each design flood were interrogated and combined to form a "design flood envelope" for each design flood. It is this "design flood envelope", comprising the worst case depths, velocities and levels at each TUFLOW cell that forms the basis for the results documented in the following sections.

6.3.3 Presentation of Model Results

The adopted modelling approach for the study involves applying rainfall directly to each cell in the computer model and routing the rainfall excess based on the physical characteristics of the catchment (e.g., variation in terrain, stormwater system). Once the rain falling on each grid cell exceeds the rainfall losses, each cell will be "wet". However, water depths across the majority of the catchment will likely be very shallow and would not present a significant flooding problem. Therefore, it was necessary for the results of the computer simulations to be "filtered" to distinguish between areas of significant inundation depth / flood hazard and those areas subject to negligible inundation.

A minimum depth threshold of 0.15 metres has been adopted in other overland flood studies completed across the Fairfield LGA for the following reasons:

- Council's standard kerb height is generally 0.15 metres. Therefore, water depths less than 0.15 metre will typically be contained to roadways and will not travel overland through properties;
- Section 3.1.2.3(b) of the Building Code of Australia (BCA) (2012), requires the floor level of buildings in poorly drained areas to be elevated 0.15 metres above the finished ground level. Accordingly, there is minimal chance of over floor flooding when water depths are less than 0.15 metres.
- Removing areas inundated by more than 0.15 metres typically resulted in many isolated "puddles" and was considered to underestimate the flood risk.

The adoption of a minimum depth threshold of 0.15 metres was also considered appropriate for the current study. That is, flood model results were only presented in the maps/figures where the depth of inundation was predicted to exceed 0.15 metres.

It was noted that application of a depth filter in isolation did still result in a number of isolated "puddles". In reality, each of these "puddles" would likely be linked by areas of shallower flow to create a continuous flow path. Council felt that it was important to also represent these linkages in the mapping. Therefore, the raw flood modelling results were reviewed and "puddles" located in close proximity to each other that appeared to be part of a continuous overland flow path were manually "linked" together. This involved reviewing velocity depth product outputs to determine where the majority of flow is conveyed between the puddles. All depths located within these "linkages" were reinstated to create continuous flow paths.

6.3.4 Design Floodwater Depths, Levels & Velocities

Peak floodwater depths for the 5% and 1% AEP events as well as the Probable Maximum Flood (PMF) were extracted from the results of the TUFLOW model and are presented in **Figures 9** to **11**.

Peak 1% AEP flood levels are provided in **Figure 12** and peak 1% AEP and PMF flow velocities are presented in **Figures 13** and **14**.

6.3.5 Design Discharges

Plot Output (PO) lines were incorporated within the TUFLOW model to allow overland discharges to be extracted for each design flood. This overland discharge information was combined with sub-surface pipe discharges (also extracted from TUFLOW) to allow the total peak discharge to be determined at discreet locations throughout the Smithfield West catchment. The peak discharges that were extracted from the TUFLOW model results are summarised in **Table 15**. The location where the peak discharges were extracted is shown in the figure located on the following page.

Table 15 Design Discharges at Key Roadway Crossings within the Smithfield West Catchment

LOCATION	Peak Discharge (m^3/s)						
	50% AEP	20% AEP	5% AEP	1% AEP	0.2% AEP	1 in 10,000 Year ARI	PMF
Gemoore St	1.5	2.1	2.5	3.0	3.8	4.9	6.7
Charles St	2.9	3.6	3.9	4.6	5.8	7.4	9.8
Brown St	3.1	4.2	4.9	5.8	7.4	9.6	13.5
Brenan St	3.4	4.9	5.6	6.4	8.4	10.7	16.1
Jane St	3.8	5.5	6.3	7.3	9.4	12.6	18.8
Corner Dublin & Neville St	6.8	8.9	10.4	12.4	15.7	20.1	29.1
Canara Pl	7.1	9.5	11.0	13.2	16.8	21.4	31.3
The Horsley Drive	7.6	10.5	12.2	14.6	18.8	23.8	35.1
Moir St	7.8	10.9	12.8	15.2	19.7	25.0	37.0
Victoria St	7.8	11.2	13.0	15.5	20.4	26.1	38.3
Hinkler St	7.3	11.3	13.5	16.1	21.1	27.0	40.9
Corner Chifley Rhondda St	7.7	11.7	13.9	16.7	22.0	28.2	43.2

NOTE: The location where peak discharges were extracted are shown in the figure on the following page

6.3.6 Stormwater Pipe Capacity

The TUFLOW model also produces information describing the amount of water flowing through each stormwater pipe including which pipes are flowing completely full during each design flood. This information can be used to provide an assessment of the capacity of each pipe in the stormwater system. In doing so, it allows identification of where stormwater capacity constraints may exist across the catchment.

The pipe flow results of all design flood simulations were interrogated to determine the capacity of each stormwater pipe in terms of a nominal return period (i.e., AEP). The capacity of the pipe was defined as the largest design event whereby the pipe was not flowing completely full. For example, if a particular stormwater pipe was flowing 80% full during the 5% AEP event, 95% full during the 10% AEP event and 100% full during the 20% AEP event, the pipe capacity would be defined as "10% AEP".

The pipe capacity is presented in **Figure 15**.

6.4 Results Verification

The TUFLOW model developed as part of this study was calibrated against two historic floods. In general, the model was found to provide a good reproduction of historic flood mark elevations. However, the outcomes of the calibration only provides evidence that the model is providing a reliable representation of flood behaviour at isolated locations (i.e., at recorded flood mark locations).

Therefore, additional validation of the TUFLOW model was completed by comparing the results generated by the TUFLOW model against past studies as well as alternate computer modelling approaches.

Further details on the outcome of the TUFLOW model verification is presented below.

6.4.1 Past Studies

The only past study where design flood level information is documented for the Smithfield West catchment is the *'Smithfield West Drainage Study: Chifley Street to The Horsley Drive'* (Dalland & Lucas, 1996). Therefore, peak 1% AEP were extracted from Appendix B of the 1996 Drainage Study and were compared against peak 1% AEP flood levels extracted from the TUFLOW model. This comparison is provided in **Table 16**.

It was noted that the TUFLOW model shows a significant water level gradient across some properties. As the 1996 Drainage Study only lists a single flood level for each property, the location where the flood level comparison is made can have a significant impact on the results of the comparison. It was noted that the 1996 study lists peak 1% AEP flood levels alongside building floor levels. Therefore, it was assumed that the 1996 study flood levels were extracted near the front entrance to each building.

The comparison provided in **Table 16** shows that the TUFLOW model generally produces 1% AEP water levels that are within 0.1 metres of the 1996 Drainage Study. Some larger discrepancies do occur (e.g., 30 Moir Street and 39 Hinkler Street), however, they tend to be isolated. These differences may be associated with localised differences in the modelling approaches (e.g., the inclusion of fences in the TUFLOW model) or the flood level comparison not occurring at the exact same location (refer previous paragraph). Nevertheless, the outcomes of the comparison shows that the TUFLOW model is producing comparable 1% AEP water level estimates between The Horsely Drive and Rhondda Street.

Peak 1% AEP discharges documented in Appendix C of the 1996 Drainage Study were also compared against peak 1% AEP discharges extracted from the TUFLOW model at select locations. This comparison is provided in **Table 17**.

The comparison in **Table 17** shows that the TUFLOW model produces lower peak 1% AEP discharges relative to the ILSAX model used in the 1996 Drainage Study. This is likely associated with the TUFLOW model including additional "micro" storage (e.g., storage behind road embankments) that is typically not included in lumped models such as ILSAX. "Lumped" models are those models where the hydrologic properties (e.g., slope) are averaged across subcatchments. As a result, they do not always account for local variations in hydrologic properties that may influence rainfall-runoff behaviour.

Table 16 Comparison between TUFLOW and 1996 Drainage Study 1% AEP Water Levels

Location	Peak Water Level (mAHD)		
	1996 Study	Current Study	Difference
823 The Horsley Drive	30.54	30.59	0.05
819 The Horsley Drive	30.54	30.57	0.03
30 Moir Street	29.85	29.72	-0.13
29 Moir Street	29.30	29.26	-0.04
22 Moir Street	29.17	29.20	0.03
23 Moir Street	29.10	29.02	-0.08
25 Moir Street	29.10	29.04	-0.06
10 Hart Street	29.01	29.04	0.03
14 Hart Street	29.01	29.09	0.08
139B Victoria Street	28.70	28.60	-0.10
144 Victoria Street	27.69	27.74	0.05
128 Victoria Street	27.24	27.15	-0.09
124 Victoria Street	27.00	27.10	0.10
1 Kingsford Street	26.48	26.39	-0.09
1 Shamrock Street	26.10	26.15	0.05
28 Hinkler Street	27.06	27.02	-0.04
32 Hinkler Street	26.90	26.87	-0.03
27 Hinkler Street	25.90	25.97	0.07
39 Hinkler Street	25.45	25.59	0.14
64 Chifley Street	25.23	25.18	-0.05
56 Chifley Street	24.55	24.53	-0.02
52 Chifley Street	24.50	24.49	-0.01
49 Rhondda Street	24.10	24.17	0.07
47 Rhondda Street	24.10	24.18	0.08
		Average	0.00
		Standard Deviation	0.07

Table 17 Comparison between TUFLOW and 1996 Drainage Study 1% AEP Peak Discharges

Location	Peak Discharge (m^3/s)		
	1996 Study	Current Study	Difference
The Horsley Drive	20.9	18.8	-2.1
Moir Street	20.8	19.7	-1.1
Victoria Street	21.2	20.4	-0.8
Hinkler Street	22.3	21.1	-1.2
Chifley Street	23.4	22.0	-1.4
		Average	-1.32
		Standard Deviation	0.44

6.4.2 Alternate Calculation Approaches

XP-RAFTS Hydrologic Model

The ability of the TUFLOW model to represent rainfall-runoff processes was validated relative to a hydrologic model of the Smithfield West catchment that were established using the XP-RAFTS software. Detailed information on the XP-RAFTS model setup is provided in **Appendix I**.

The XP-RAFTS model was subsequently used to simulate the 1% AEP flood using the same hydrologic inputs as the TUFLOW model (i.e., design rainfall, rainfall losses, impervious proportion etc). Peak 1% AEP flows were extracted from the XP-RAFTS model at key subcatchments for the 2 hour storm duration and are presented in **Table 18**. The corresponding TUFLOW 1% AEP flow at each subcatchments is also provided **Table 18** for comparison. Full discharge hydrographs showing the time variation in flows at discreet locations throughout the catchment were also extracted from the XP-RAFTS and TUFLOW model results and are included in **Appendix I**.

Table 18 Verification of TUFLOW 1%AEP Peak Discharges against alternate calculation approaches

XP-RAFTS Subcatchment	Peak 1% AEP Flow (m^3/s)				
	TUFLOW	XP-RAFTS		PRM	
		Peak Flow	Difference	Peak Flow	Difference
47	3.2	3.0	0.2	2.8	-0.4
41	7.2	7.0	0.2	6.3	-0.9
31	12.9	12.9	0.0	11.7	-1.2
22	14.8	14.4	0.4	13.1	-1.7
Victoria St	18.9	18.3	0.6	17.1	-1.8
4	19.2	19.6	-0.4	19.0	-0.2

The comparison provided in **Table 18** shows that, in all cases, the TUFLOW model reproduces the XP-RAFTS peak discharges to within 0.6 m^3/s. The hydrograph comparison provided in **Appendix I** also shows that the TUFLOW model generally provides a good reproduction of the

XP-RAFTS hydrograph shape, with the volume of runoff (indicated by the area under the hydrograph) and timing of the peak discharge being well replicated.

This outcome shows that the TUFLOW model is providing a reasonable reproduction of the XP-RAFTS model discharges and indicates the TUFLOW model is providing a reasonable representation of the hydrologic processes across the Smithfield West catchment.

Probabilistic Rational Method

Additional verification of the 1% AEP discharges generated by the TUFLOW model was completed by comparing peak 1% AEP discharges against peak discharges calculated using the Probabilistic Rational Method (PRM). The outcomes of the comparison is provided in **Table 18** at select locations across the Smithfield West catchment.

In general, the TUFLOW and PRM discharges provided in **Table 18** show a good correlation with discharges typically agreeing to within 15%. The PRM typically predicts lower discharges relative to the TUFLOW model. This is not unexpected as the PRM is designed for application in rural catchments so fails to account for the increased runoff potential across impervious sections of the catchment.

Nevertheless, the outcomes of the peak discharge comparison indicates that the TUFLOW model is producing realistic 1% AEP peak discharges.

7 Flood Hazard and Hydraulic Categories

7.1 Flood Hazard

7.1.1 Overview

Flood hazard defines the potential impact that flooding will have on development and people across different sections of the floodplain.

The determination of flood hazard at a particular location requires consideration of a number of factors, including (NSW Government, 2005):

- depth and velocity of floodwaters;
- size of the flood;
- effective warning time;
- flood awareness;
- rate of rise of floodwaters;
- duration of flooding; and
- potential for evacuation.

Consideration of the depth and velocity of floodwater in isolation is referred to as the *hydraulic* or *provisional* flood hazard. The provisional flood hazard at a particular area of a floodplain can be established from Figure L2 of the *"Floodplain Development Manual"* (NSW Government, 2005). This figure is reproduced on the right.

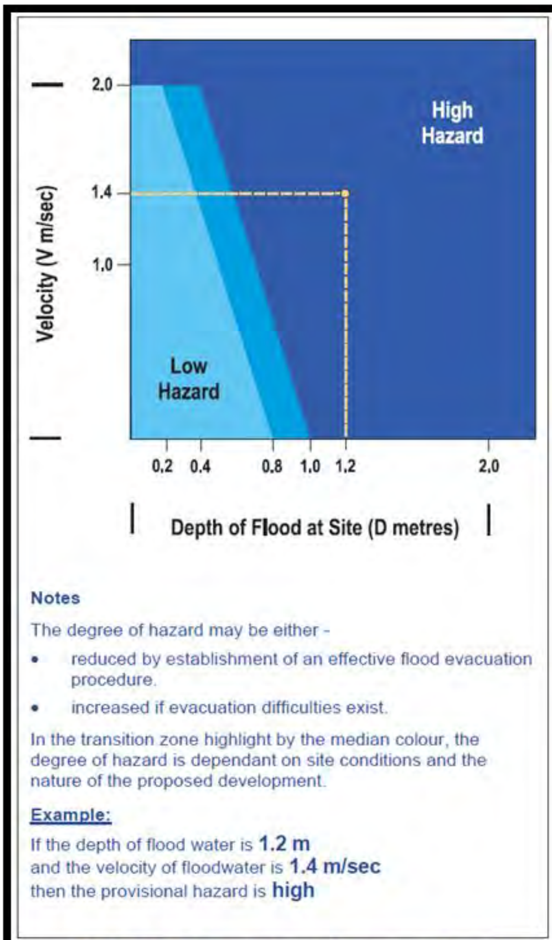


FIGURE L2 - Provisional Hydraulic Hazard Categories

As shown in Figure L2, the *"Floodplain Development Manual"* (NSW Government, 2005) divides provisional hazard into two categories, namely high and low. It also includes a *transition zone* between the low and high hazard categories. Sections of the floodplain located in the "transition zone" may be classified as either high or low depending on site conditions or the nature of any proposed development.

7.1.2 Provisional Flood Hazard

The TUFLOW hydraulic software was used to automatically calculate the variation in provisional flood hazard across the Smithfield West catchment based on the criteria shown in Figure L2 for the 1% AEP flood as well as the PMF. These hazard category maps are shown in **Figures 16** and **17**.

It needs to be reinforced that the hazard represented in this mapping is provisional only. This is because it is based only on an interpretation of the flood hydraulics and does not reflect the effects of other factors that influence flood hazard. Refinement of the provisional hazard categories to include consideration of these other factors will be completed as part of the future floodplain risk management study.

7.2 Flood Emergency Response Planning Classifications

The provisional hazard mapping presented in **Figures 16** and **17** can provide an indication of the risk to life and property across different sections of the floodplain based on the depth and the velocity of floodwaters. Those areas subject to a low flood hazard can, if necessary, be evacuated by trucks and able-bodied adults would have little difficulty wading to safety (NOTE: evacuation by car may <u>not</u> be possible). Those areas of the floodplain exposed to a high flood hazard would have difficulty evacuating by trucks, there is potential for damage to buildings and there is possible danger to personal safety (i.e., evacuation by wading may not be possible).

Accordingly, the provisional hazard categories provide an initial appraisal of the variation in flood hazard across the Smithfield West catchment based on the depth and velocity of floodwaters. However, a number of other factors need to be considered to determine the potential vulnerability of the community during floods.

In an effort to quantify the other factors that impact on flood hazard, the Office of Environment and Heritage (formerly Department of Environment and Climate Change), in conjunction with the State Emergency Service (SES) developed the *"Flood Emergency Response Planning Classification of Communities" (2007)*. The guideline was also developed to assist the SES in planning and implementing response strategies for different sections of the floodplain.

The guideline provides a basis for the categorisation of floodplain communities into various Emergency Response Planning (ERP) classifications. The ERP classifications are summarised in

Table 19 and can be used to provide an indication of the type of emergency response required across different sections of the floodplain.

Each allotment within the Smithfield West catchment was classified based upon the flow chart provided in the ERP guideline for the 1% AEP flood as well as the PMF. This was completed using the TUFLOW model results, DEM and a road network GIS layer in conjunction with proprietary software that considered the following factors:

- whether evacuation routes/roadways get "cut off" and the depth of inundation (a 200mm depth threshold was used to define a "cut" road);
- whether evacuation routes continuously rise out of the floodplain;
- whether an allotment gets inundated during the nominated design flood and whether evacuation routes are cut or the lot becomes completely surrounded (i.e., isolated) by water before inundation;

- if evacuation by car was not possible, whether evacuation by walking was possible (a 500mm depth threshold was used to define when a route could not be traversed by walking).

Table 19 Response Required for Different Flood ERP Classifications (Department of Environment & Climate Change, 2007)

Classification	Response Required		
	Resupply	**Rescue/Medivac**	**Evacuation**
High Flood Island	Yes	Possibly	Possibly
Low Flood Island	No	Yes	Yes
Area with Rising Road Access	No	Possibly	Yes
Area with Overland Escape Routes	No	Possibly	Yes
Low Trapped Perimeter	No	Yes	Yes
High Trapped Perimeter	Yes	Possibly	Possibly
Indirectly Affected Areas	Possibly	Possibly	Possibly
Not Flood Effected	No	No	No

The resulting ERP classifications for the 1% AEP flood as well as the PMF are provided in **Figures 18** and **19**. A range of other datasets were also generated as part of the classification process to assist Council and the SES. This includes the locations where roadways first become cut by floodwaters, the time at which the roadways first become cut, the length of time the roadways are cut as well as the maximum depth of inundation. The location where roads first get cut, the time roads first get cut and the duration of inundation selection of this information is also included in **Figures 18** and **19**.

It should be noted that the automated application of the Flood Emergency Response Classification Flow Chart at allotment scales is a technique still under current research and development. For more information, please refer to the paper, Emergency Response Planning Classification at Sub-Precinct Scales (Ryan et al, 2014).

7.3 Hydraulic Categories

7.3.1 Overview

The NSW Government's *'Floodplain Development Manual'* (NSW Government, 2005) also characterises flood prone areas according to the hydraulic categories presented in **Table 20**. The hydraulic categories provide an indication of the potential for development across different sections of the floodplain to impact on existing flood behaviour and highlights areas that should be retained for the conveyance of floodwaters.

7.3.2 Adopted Hydraulic Categories

Unlike provisional hazard categories, the *"Floodplain Development Manual"* (NSW Government, 2005) does not provide explicit quantitative criteria for defining hydraulic categories. This is because the extent of floodway, flood storage and flood fringe areas are typically specific to a particular catchment.

Table 20 Qualitative and Quantitative Criteria for Hydraulic Categories

Hydraulic Category	Floodplain Development Manual Definition	Adopted Criteria*
Floodway	• those areas where a significant volume of water flows during floods • often aligned with obvious natural channels and drainage depressions • they are areas that, even if only partially blocked, would have a significant impact on upstream water levels and/or would divert water from existing flowpaths resulting in the development of new flowpaths. • they are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.	Area where 80% of the total flow is conveyed
Flood Storage	• those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood • if the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. • substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.	Areas that are not floodway and where the depth of inundation is greater than 0.15 metres
Flood Fringe	• the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. • development (e.g., filling) in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.	Areas that are not floodway where the depth of inundation is less than 0.15 meters

In an effort to provide quantitative criteria for establishing floodway extents, Thomas & Golaszewski (2012), suggested that floodways can be defined as areas where 80% of the total flow is conveyed. Accordingly, this criteria was used as the basis for establishing the extent of floodways across the Smithfield West catchment.

To enable this criteria to be applied, the alignment of major flow paths across the catchment were first delineated by hand based upon TUFLOW depth and velocity outputs. Cross-sections were subsequently extracted perpendicular to each flow path alignment and the total discharge at each cross-section was calculated based on the design water depth and velocity outputs (a depth threshold of 0.15 metres was adopted to define the end of each cross-section). The cross-section was progressively truncated until 80% of the total flow was captured within the truncated cross-section limits. This was defined as the floodway extent at that cross-section location. This process was repeated at 2 metre increments along each flow path alignment to create a continuous floodway.

As discussed in Section 6.3.3, "filtering" of the raw modelling results was completed to remove areas of insignificant inundation from the flood mapping (i.e., areas where the depth of inundation was less than 0.15 metres). It was considered that the areas that were removed from the flood mapping would fall under the "flood fringe" hydraulic category. Accordingly, it is suggested that those areas where no depth / hydraulic category mapping is presented would be considered flood fringe.

Flood storage areas were then defined as those areas located outside of floodways but where the depth of inundation was greater than 0.15 metres. This aimed to identify areas where a significant amount of flow was not necessarily conveyed, however, the depths of water indicate a significant amount of storage capacity was being provided.

The resulting hydraulic category maps for the 1% AEP flood and PMF are shown in **Figures 20** and **21**.

7.4 Flood Risk Precincts

Fairfield City Council subdivides each floodplain within their LGA into Flood Risk Precincts. The Flood Risk Precincts are used as the basis for defining the variation in flood risk across the Fairfield City Council LGA and are used as the basis for determining what development controls apply to land within the floodplain to ensure the flood risk is suitably managed.

Chapter 11 of the *'Fairfield City Wide Development Control Plan'* (Fairfield City Council, 2014) provides definitions for three Flood Risk Precincts (i.e., Low, Medium and High). The definition for each precinct is reproduced in **Table 21**.

Table 21 Flood Risk Precinct Definitions

Flood Risk Precinct	Definition
High	Land below the 1% AEP flood that is either subject to a high hydraulic hazard or where there are significant evacuation difficulties
Medium	Land below the 1% AEP flood that is not subject to a high hydraulic hazard and where there are no significant evacuation difficulties
Low	This has been defined as all other land within the floodplain (i.e. within the extent of the probable maximum flood) but not identified within either the High Flood Risk or the Medium Flood Risk Precinct.

As shown in **Table 21**, land where there are significant evacuation difficulties fall under the "High" Flood Risk Precinct classification. For the purposes of this study, any property that was categorised as a "low flood island" during the 1% AEP flood as part of the Flood Emergency Response Precinct classifications (refer Section 7.2 and **Figure 18**) was classified as having significant evacuation difficulties.

The Flood Risk Precinct Map that was developed for the Smithfield West catchment is shown in **Figure 22**.

The number of properties/lots across the catchment falling within each flood risk precinct was also calculated. The results of this analysis are provided in **Table 22**. For properties that were exposed to greater than one flood risk precinct classification, the "worst case" flood risk precinct category was adopted.

Table 22 Number of properties within each Flood Risk Precinct

Flood Risk Precinct	Number of Lots Within Each Precinct
High	68
Medium	193
Low	258
No classification	956

8 Sensitivity Analysis

8.1 General

Computer flood models require the adoption of several parameters that are not necessarily known with a high degree of certainty. Each of these parameters can impact on the results generated by the model.

As outlined in Section 5, computer models are typically calibrated using recorded rainfall, stream flow and/or flood mark information. Calibration is achieved by adjusting the parameters that are not known with a high degree of certainty until the computer model is able to reproduce the recorded flood information. Calibration is completed in an attempt to ensure the adopted model parameters are generating realistic estimates of flood behaviour.

As discussed in Sections 5 and 6.4, the TUFLOW model was calibrated and verified using historic flood information, alternate calculation approaches as well as design flood results documented in previous studies. In general, the model was found to provide a reasonable reproduction of past floods and studies.

Nevertheless, it is important to understand how any uncertainties in model input parameters may impact on the results produced by the model. Therefore, a sensitivity analysis was undertaken to establish the sensitivity of the results generated by the computer model to changes in model input parameter values. The outcomes of the sensitivity analysis are presented below.

8.2 Model Parameter Sensitivity

8.2.1 Initial Loss / Antecedent Conditions

An analysis was undertaken for the 1% AEP storm to assess the sensitivity of the results generated by the TUFLOW model to variations in antecedent wetness conditions (i.e., the dryness or wetness of the catchment prior to the design storm event). A catchment that has been saturated prior to a major storm will have less capacity to absorb rainfall. Therefore, under wet antecedent conditions, there will be less "initial loss" of rainfall and consequently more runoff.

The variation in antecedent wetness conditions was represented by increasing and decreasing the initial rainfall losses in the TUFLOW model. Specifically, initial losses were changed from the "design" values of 10mm/1mm (for pervious/impervious areas respectively) to:

- "Wet" catchment: 0mm for pervious and impervious areas; and,
- "Dry" catchment: 20mm for pervious areas and 2mm for impervious areas

The TUFLOW model was used to re-simulate the 1% AEP event with the modified initial losses. Peak water levels were extracted from the results of the modelling and were compared against peak water flood levels for "base" design conditions. This allowed water level difference mapping to be prepared showing the magnitude of any change in water levels associated with the change in initial loss values. A sample difference map for the “dry” catchment simulation is provided in **Plate 25** (decreases in flood level relative to the “design” flood levels documented in **Figure 12** are shown in blue).

Plate 25 Sample flood level difference map for the “dry” catchment sensitivity simulation

The difference mapping was statistically analysed to determine the magnitude of changes in peak 1% AEP water levels across areas of significant inundation depth (i.e., >0.15 metres). The outcomes of this statistical assessment are shown in **Table 23**.

The results documented in **Table 23** show that changing the initial losses alters peak 1% AEP flood levels by between 0.01 and -0.02 metres, on average. The maximum change in peak 1% AEP water level is 0.14 metres and occurs between Victoria Street and Hinkler Street where there are relatively narrow overland flow paths between buildings (refer **Plate 26**). The reduced conveyance provided through these narrow building gaps magnifies the flood level differences at this location relative to other areas.

Overall, it can be concluded that the model is relatively insensitive to changes in the adopted initial losses. *'Australian Rainfall & Runoff'* (Engineers Australia, 1987) suggests adopting an initial loss of between 10 mm and 30 mm for design flood estimation. The adopted initial loss of 10 mm is at the lower end of the suggested range and would, therefore, provide reasonably conservative design flood estimates.

Table 23 Model Parameter Sensitivity Analysis Results

Sensitivity Analysis		Change in "base" 1% AEP flood levels (metres)			
		Max. Decrease	Max. Increase	Average Change	Std Deviation
Initial Loss	Perv IL = 0 mm Imperv. IL = 0 mm	-0.00	0.08	0.01	0.00
	Perv IL = 20 mm Imperv. IL = 2 mm	-0.14	0.01	-0.02	0.01
Continuing Loss Rate	Perv CL = 1.5 mm/hr Imperv. CL = 0 mm/hr	-0.01	0.03	0.00	0.00
	Perv CL = 3.5 mm/hr Imperv. CL = 1.0 mm/hr	-0.05	0.00	0.00	0.00
Manning's 'n'	-10%	-0.12	0.04	0.01	0.01
	+10%	-0.03	0.08	0.01	0.01
Stormwater & Culvert Blockage	No Blockage	-0.04	0.02	0.00	0.00
	Complete Blockage	-0.44	0.24	0.04	0.05
Design Rainfall	New IFD	-0.27	0.00	-0.03	0.03
Downstream Boundary	Normal Depth Slope - 10%	-0.00	0.02	0.00	0.00
	Normal Depth Slope + 10%	-0.02	0.00	0.00	0.00
	Inflow - 10%	-0.13	0.01	0.02	0.03
	Inflow + 10%	-0.03	0.17	0.03	0.03

Plate 26 Relatively narrow gaps between buildings near Hinkler Street where flood level differences are magnified

8.2.2 Continuing Loss Rate

An analysis was also undertaken to assess the sensitivity of the results generated by the TUFLOW models to variations in the adopted continuing loss rates. Accordingly, the continuing loss rates were changed from the "design" values of 2.5 mm/hr (pervious areas) and 0 mm/hr (impervious areas) to:

- Increased Continuing Loss Rates: 3.5mm/hr for pervious areas and 1mm/hr for impervious areas.
- Decreased Continuing Loss Rates: 1.5mm/hr for pervious areas and 0mm/hr for impervious areas.

The TUFLOW model was used to re-simulate the 1% AEP flood with the modified continuing loss rates. Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping. The difference maps were statistically analysed and the outcomes of the analysis are presented in **Table 23**.

The results of the sensitivity analysis show that the TUFLOW model is relatively insensitive to changes in continuing loss rates. More specifically, the average change in water level for both scenarios is predicted to be zero, although some localised increases of up to 0.05 metres are predicted. The largest changes in peak 1% AEP water levels again occur between Victoria and Hinkler Streets.

Therefore, it can be concluded that any uncertainties associated with the adopted continuing loss rates are not predicted to have a significant impact on the results generated by the TUFLOW model.

8.2.3 Manning's 'n'

Manning's' 'n' roughness coefficients are used to describe the resistance to flow afforded by different land uses / surfaces across the catchment. However, they can be subject to variability (e.g., vegetation density in the summer would typically be higher than the winter leading to higher Manning's 'n' values). Therefore, additional analyses were completed to quantify the impact that any uncertainties associated with Manning's 'n' roughness values may have on predicted design flood behaviour.

The TUFLOW model was updated to reflect a 10% increase and a 10% decrease in the adopted design Manning's 'n' values and additional 1% AEP simulations were completed with the modified 'n' values. Flood level difference mapping was prepared based on the results of the revised simulations and the results are presented in **Table 23**.

The results listed in **Table 23** show that changing the Manning's 'n' values by 10% will alter peak 1% AEP flood levels by 0.01 metre, on average. Some increases of more than 0.1 metres are predicted at isolated locations. The largest changes in flood level tend to be concentrated in areas where the vegetation density (and, therefore, Manning's "n" values) are highest (e.g., Prospect Creek and the open channel in the Market Gardens).

But overall, it is considered that the model is relatively insensitive to changes in Manning's 'n' values.

8.2.4 Stormwater and Culvert Blockage

As discussed in Section 6.2.2, blockage factors ranging between 30% and 50% were applied to all culverts and stormwater inlets as part of the design flood simulations. However, as it is not known which structures will develop what percentage of blockage during any particular flood, additional TUFLOW simulations were completed to determine the impact that alternate blockage scenarios would have on simulated flood behaviour. Specifically, additional simulations were undertaken with no blockage as well as complete blockage of all stormwater inlets and culverts. Flood level difference mapping was also prepared and interrogated to quantify the impact that variations to the adopted blockage factors would have on 1% AEP flood levels. The outcomes of the difference mapping assessment are presented in **Table 23**.

The results documented in **Table 23** show that the TUFLOW model is sensitive to increases in blockage factors. Specifically, complete blockage of drainage structures has the potential to increase 1% AEP flood levels by over 0.2 metres and decrease 1% AEP flood levels by over 0.4 metres. However, these increases occur at an isolated location (i.e., in the immediate vicinity of the culvert contained within the Market Gardens) and quickly dissipate. Compete blockage of drainage structures is predicted to cause an average change in 1% AEP flood level of 0.04 metres.

The results included in **Table 23** also show that removal of all blockage is only predicted to cause 1% AEP flood levels to change by 0.01 metres, on average. This comparative lack of sensitivity with no blockage is likely associated with the limited pipe capacity across the Smithfield West catchment. As discussed in Section 6.3.6 most pipes within the catchment have a limited capacity (i.e., much less than a 1% AEP capacity). As a result, the pipe system is generally "*fully charged*" during the 1% AEP flood regardless of the blockage that is applied to the stormwater pits (i.e., the limited stormwater capacity is governed by the pipe capacity rather than the stormwater inlet/pit capacity and any associated pit blockage).

Overall, it is considered that the TUFLOW model is sensitive to variations in blockage in the immediate vicinity of drainage structures, particularly if blockage increases above that which has been adopted as part of the design simulations. This outcome emphasises the need to ensure key drainage infrastructure and culverts are well maintained (i.e., debris is removed on a regular basis).

8.2.5 Design Rainfall

Design rainfall was applied to the TUFLOW model based upon information and standard procedures documented in *"Australian Rainfall and Runoff – A Guide to Flood Estimation"* (Engineers Australia, 1987). However, at the time this study was being prepared, a new version of *"Australian Rainfall and Runoff"* was in the process of being prepared. The revised version includes new intensity-frequency-duration (IFD) information that takes advantage of over 20 years of additional rainfall information. Although the revised IFD data has been released, the Bureau of Meteorology and Engineers Australia outlines that the revised IFD data should not be used for design flood estimation until the full suite of revised techniques is released as part of the new version of *"Australian Rainfall and Runoff"*. Nevertheless, the

Bureau of Meteorology and Engineers Australia recommends that the revised IFD data be used as part of sensitivity testing.

Therefore, revised 1% AEP simulations were completed with the revised design rainfall across the Smithfield West catchment. Design temporal patterns documented in the 1987 version of *"Australian Rainfall and Runoff – A Guide to Flood Estimation"* (Engineers Australia, 1987) were retained as updated temporal patterns are yet to be released for the revised IFD data.

As discussed, Prospect Creek adjoins the downstream boundary of the Smithfield West catchment. Therefore, any changes in design rainfall across the broader Prospect Creek catchment also has the potential to impact on results across the downstream sections of the Smithfield West catchment. Therefore, the XP-RAFTS model of Prospect Creek was also updated to reflect the revised design rainfall information across the upper Prospect Creek catchment and was used to define revised 1% AEP inflow hydrographs for Prospect Creek.

Flood level difference mapping was prepared based on the outcomes of the 1% AEP simulation with the revised IFD values. The results of the difference mapping assessment are presented in **Table 23**.

The results presented in **Table 23** shows that the new IFD rainfall information will reduce peak 1% AEP flood levels across the Smithfield West catchment by up to 0.27 metres. The largest decreases in peak 1% AEP flood level are predicted to occur between Victoria Street and Hinkler Street where the relatively narrow gaps between buildings accentuate the flood level differences.

The average reduction in peak 1% AEP flood level is predicted to be 0.03 metres. Accordingly, the model is considered to be relatively sensitive to a change in IFD information. However, it should be noted that a conclusive sensitivity assessment of the old (i.e., 1987) versus new IFD data cannot be completed until the full suite of revised procedures is released as part of the new version of *"Australian Rainfall and Runoff"* (e.g., revised temporal patterns). As a result, the current 1% AEP water levels should be used until the full suite of information is released as part of the new version of *"Australian Rainfall and Runoff"*.

8.2.6 Downstream Boundary

General

The Smithfield West catchment drains into Prospect Creek, which forms the downstream boundary of the Smithfield West catchment. Accordingly, any uncertainties in the definition of flood behaviour along Prospect Creek has the potential to impact on results across the downstream sections of the Smithfield West catchment.

Therefore, two separate analyses were completed to quantify how uncertainty in Prospect Creek boundary conditions may impact on flood behaviour across the Smithfield West catchment, namely:

- Variations in normal depth boundary definition at the downstream end of Prospect Creek; and,
- Variations in design inflow hydrographs at the upstream end of Prospect Creek.

Normal Depth Boundary
As discussed in Section 6.2.1, the downstream boundary condition for Prospect Creek was defined in the TUFLOW model using an automatic normal depth (i.e., Manning's "n") calculation. This involved defining a bed slope at the downstream model boundary. A bed slope of 0.01 was adopted for this purpose based on a review of surveyed cross-sections and available LiDAR information.

Therefore, additional 1% AEP simulations were completed with modified downstream bed slopes. More specifically, revised simulations were completed with the bed slope increased and decreased by 10%. Flood level difference mapping was prepared based on the results of the revised simulations and the results are summarised in **Table 23**.

The results documented in **Table 23** shows that altering the normal depth bed slope changes peak 1% AEP by up to 0.02 metres (the maximum difference occurs at the downstream model boundary) with an average change of zero. Therefore, it can be concluded that the model is relatively insensitive in changes in Prospect Creek bed slope.

Inflow Boundary
Additional simulations were also completed to quantify the impact of uncertainties in Prospect Creek inflows on flood behaviour across the downstream sections of the Smithfield West catchment. 1% AEP Prospect Creek inflows were increased and decreased by 10% and revised 1% AEP simulations were completed. Flood level difference mapping was prepared based on the revised simulations and the results are summarised in **Table 23**.

The results documented in **Table 23** shows that changing Prospect Creek inflows has the potential to change peak 1% AEP flood levels by nearly 0.2 metres. However, this maximum change occurs at the upstream boundary of Prospect Creek and quickly dissipates to less than 0.1 metres. Along the remainder of Prospect Creek, changes in 1% AEP flood levels are typically around 0.05 metres with the average difference being 0.03 metres.

Overall, the results of the inflow sensitivity analysis show that the model is relatively sensitive to changes in Prospect Creek inflows along Prospect Creek and the immediate floodplain (including the downstream sections of the Smithfield West catchment). However, across the majority of the Smithfield West catchment, Prospect Creek inflows are not predicted to impact on peak 1% AEP flood level estimates.

8.3 Climate Change

8.3.1 Overview

The *'Practical Consideration of Climate Change'* (Department of Environment and Climate Change, 2007) guideline states that rainfall intensities are predicted to increase in the future. The NSW Government's *'Climate Change in the Sydney Metropolitan Catchments'* (CSIRO, 2007) elaborates on this further and suggests that annual rainfall is likely to decrease, however, extreme rainfall events are likely to be more intense. It is anticipated that extreme rainfall intensities could increase by between 2% and 24% by 2070 (Department of Environment and Climate Change, 2007).

Due to the wide potential variability of future rainfall intensities, the '*Practical Consideration of Climate Change*' (Department of Environment and Climate Change, 2007) provides guidelines for quantifying the potential impacts of these changes. The guideline states that additional simulations should be completed with 10%, 20% and 30% increases in rainfall intensities to quantify the potential impacts associated with climate change.

8.3.2 Rainfall Intensity Increases

The TUFLOW model was used to perform additional simulations incorporating increases in 1% AEP design rainfall intensity of 10%, 20% and 30% in accordance with the Department of Environment and Climate Change guideline.

Peak floodwater levels were extracted from the results of the modelling and were compared against peak water flood levels for 'base' conditions. This allowed water level difference mapping to be prepared showing the magnitude of any change in water levels associated with increases in rainfall intensity. The difference mapping was interrogated to determine the magnitude of changes in peak water levels associated with increases in rainfall intensity. A summary of this assessment is provided in **Table 24**.

Table 24 Climate Change Sensitivity Analysis Results

Climate Change Scenario	**Change in "base" 1% AEP flood levels** (metres)			
	Max. Decrease	**Max. Increase**	**Average Change**	**Std Deviation**
10% increase in 1% AEP rainfall	-0.00	0.15	0.02	0.02
20% increase in 1% AEP rainfall	-0.00	0.19	0.04	0.03
30% increase in 1% AEP rainfall	-0.00	0.28	0.06	0.05

The results provided in **Table 24** show that increases in rainfall intensity will increase peak 1% AEP water levels by nearly 0.3 metres at some locations. However, the average change in water level is predicted to be less than 0.10 metres under all three rainfall increase scenarios.

Accordingly, if climate change was to increase rainfall intensities in the future it has the potential to increase the severity of flooding across the study area. The most significant impacts are predicted to occur in the vicinity of the Market Gardens culvert and some stormwater pits/pipes where many of these structures are not currently sized to convey large events, such as the 1% AEP flood. As a result, the impacts of increases in rainfall intensity are magnified at these locations.

As noted in Section 8.2.5, a revised version of *"Australian Rainfall and Runoff"* is being prepared. The outcomes of the sensitivity testing showed that revised the IFD data has the potential to reduce peak 1% AEP flood levels relative to current design rainfall information. Therefore, it is recommended that the climate change analysis be revisited when the revised *"Australian Rainfall and Runoff"* procedures are released to determine if climate change induced increases in rainfall are potentially offset by reductions in the revised "base" design IFD / rainfall information.

8.4 Computer Model Confidence Limits

As discussed, the development of computer models requires the specification of parameters that are not always known with a high degree of certainty. The computer model that was created as part of this study was developed based upon best estimates of model parameters. The model was subsequently shown to produce realistic results relative to available historic flood information as well as other flooding investigations. Accordingly, the computer model is considered to provide a reasonable estimate of design flood behaviour across the catchment for existing conditions.

However, the outcomes of the climate change assessment and sensitivity analysis indicate that the design flood level estimates may be subject to variations if one or more of the input variables change (e.g., stormwater/culvert blockage, rainfall intensities, hydraulic roughness, initial/continuing losses). Accordingly, the model input parameters and design flood level estimates presented in this report are subject to some uncertainty.

In recognition of this uncertainty, additional statistical analyses were completed based upon the outcomes of the various sensitivity and climate change analyses in an attempt to assign "confidence limits" to the peak 1% AEP flood level estimates. In order to reliably define confidence limits to the 1% AEP results, it would be necessary to undertake thousands (potentially tens of thousands) of simulations to reflect the numerous combinations/permutations of potential parameter estimates and provide a sufficiently large population to enable meaningful statistical analysis. Unfortunately, the long simulation times only permit a limited number of parameter scenarios to be investigated (as outlined in Sections 8.2 and 8.3).

In instances where a sufficiently large "population" of results is not available to enable a meaningful statistical analysis, it is still possible to derive confidence limits using the Student's t-test (Ying Zhang, 2013). This approach involves interrogating peak flood level estimates from all 1% AEP simulations (i.e., design, sensitivity and climate change simulations) at each TUFLOW grid cell. This information is used to calculate a mean water level and standard deviation at each grid cell. This information can then be combined with the population size to develop 99% confidence limit estimates at each TUFLOW grid cell.

The resulting "99% Confidence Limit" grid is shown in **Plate 24**. The yellow areas indicate small confidence intervals (i.e., more confidence in results), while red areas indicate higher confidence intervals (i.e., less confidence in results).

The confidence limit grid shows that the model confidence limits across most of the study area is low (i.e. <0.01 metres), indicating a relatively high degree of confidence in the model results. The areas of highest confidence tend to coincide with areas where the depth of inundation is shallow.

The confidence limits along major overland flow paths are higher (i.e., >0.05 metres) indicating reduced confidence in the model results. Nevertheless, the model confidence limits do not exceed 0.15 metres at any location. The highest levels of uncertainty occur along the open channel in the Market Gardens and between Victoria Street and Hinkler Street where the relatively narrow gaps between buildings accentuate flood level differences (refer discussion in Section 8.2.1).

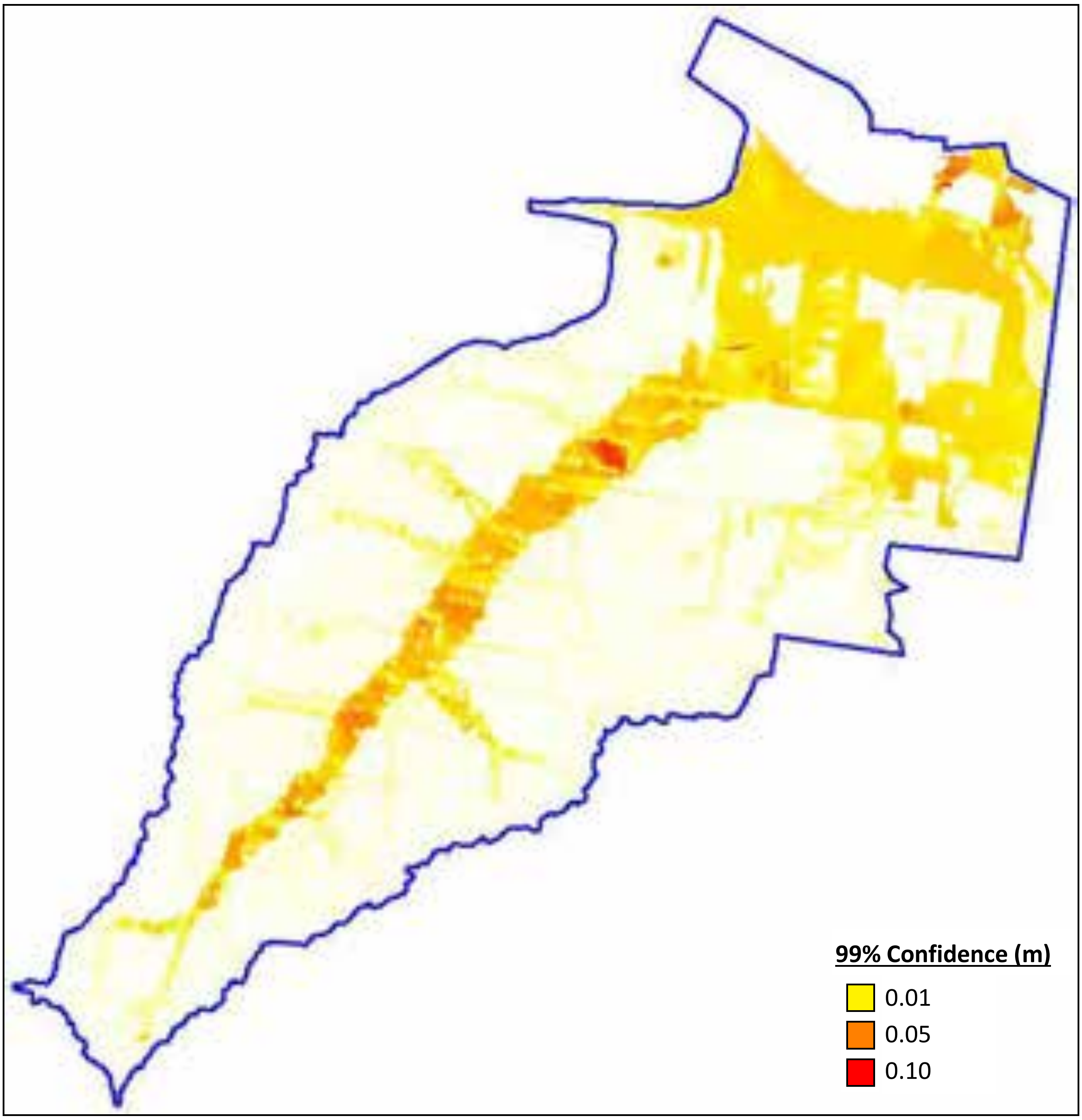


Plate 27 99% Confidence Interval Map for the 1% AEP Flood

8.5 Suitability of Freeboard

Freeboard is a factor of safety that is used to account for uncertainties in computer modelling results. The freeboard is typically used in conjunction with 1% AEP flood level estimates to derive the flood planning level for a particular location.

Fairfield City Council currently has a 0.5 metre freeboard adopted for all flood study areas. Council intends to modify this freeboard height for overland study areas to 0.3 metres. This change is required to Council's Local Environmental Plan 2013, which has been formally gazetted by the NSW State Government. This process is currently underway. Accordingly, Council requested that the suitability of adopting a 0.3 metre freeboard across the Smithfield West catchment also be investigated for consideration in reviewing its planning controls related to floodplain risk.

The freeboard is generally used to account for the following uncertainties:

- Model parameter uncertainty;
- Climate change;
- "Local" factors that can't be explicitly represented in the computer modelling (e.g., small flow paths less than the model grid size);
- Wave action (e.g., wind, boat or car induced waves);

As discussed, the result of the sensitivity / climate change assessment and subsequent confidence limit grid (refer **Plate 27)** shows that model parameter uncertainty and climate change may cause changes in 1% AEP flood levels of up to 0.12 metres.

Unfortunately, the uncertainty associated with the remaining factors cannot be as readily quantified. However, as the wind fetch length is small, water depths are generally shallow and any boats or cars would typically be operating at low speeds it is unlikely that any changes in flood levels associated with wave action would exceed 0.15 metres . As a result, it is considered that a freeboard that accounts for the following uncertainties would be approriate:

- Modelling uncertainty = 0.12 metres
- *'Other'* uncertainty = 0.15 metres

Accordingly, a minimum freeboard of 0.12 metres + 0.15 metres = approximately 0.3 metres is considered reasonable. This freeboard height may be considered for future floodplain risk planning controls.

9 Flood Impacts

9.1 General

The Smithfield West catchment has been impacted by flooding on a number of occasions in the past. The level of impact has ranged from roadways being cut by floodwaters through to yards, garages and dwellings being inundated.

In an effort to quantify the impact that flooding has on the Smithfield West catchment, the number of properties subject to over floor flooding and the likely flood damage that would be incurred during the full range of design floods was calculated. The impact of flooding on key infrastructure and transportation links was also reviewed and is presented in the following sections.

9.2 Above Floor Flooding

9.2.1 Building Floor Levels

It is necessary to have information describing the floor height / level of every building within the PMF extent to enable the number of properties subject to above floor flooding to be estimated. As discussed in Section 4.2.6, the floor levels were defined using either surveyed floor level information or were estimated using a "drive by" survey. The surveyed floor levels were generally extracted from the *'Smithfield West Drainage Study: Chifley Street to The Horsley Drive'* (Dalland & Lucas, 1996). Survey floor levels were available for 105 of the 534 properties located within the PMF extent.

Where surveyed floor levels were not available, the floor levels were estimated using the following "drive by" survey process:

1. Google Street View was used to estimate how high the floor level of each building was elevated above the adjoining ground;
2. The ground level at the point where the floor height was estimated was extracted from the available LiDAR data;
3. The floor level was subsequently estimated by adding the floor height (calculated in step 1) to the ground elevation (calculated in step 2).

9.2.2 Flood Level Estimates

The number of properties subject to above floor flooding during each design flood can be estimated by comparing the building floor levels against peak design flood levels at each building. However, the adopted modelling approach for the study involved applying rainfall directly to the TUFLOW model (including building footprints). As a result of this modelling approach, all buildings will be "wet" even though they may not be subject to over floor flooding resulting from flow entering the building from the upstream catchment.

As discussed in Section 6.3.3, a "filter" was applied to the raw modelling results to distinguish between areas of negligible and more significant overland flooding. Accordingly, this process should remove inundation from the majority of buildings where inundation from the direct rainfall approach dominates. However, it may also remove design flood level estimates from buildings subject to actual over floor flooding where the depth of inundation above floor level is less than 0.15 metres. Therefore, it was considered important to reinstate design flood level estimates across building footprints to ensure all buildings that have the potential to be inundated above floor level are identified. However, reinstatement of the "raw" modelling results was not considered to be appropriate as it would re-introduce the original direct rainfall problem.

Therefore, a revised flood level results surface was developed for each design flood using the following approach:

- The "filtered" water level results were removed from within each building footprint;
- Water levels were then reinstated across each building footprint using an inverse distance weighted interpolation. This creates a more realistic water level estimates based on the water levels surrounding each building;

9.2.3 Results

The interpolated water level at the centroid of each building was compared against the building floor level to determine the potential for over flood flooding during each design flood.

Table J1 in **Appendix J** lists the number of buildings subject to above floor flooding during each design events (grouped according to street). **Tables J2** to **J4** provides a more detailed breakdown of the number of residential, industrial or commercial properties within each street subject to over floor flooding. **Table J5** lists the number of properties on each street that are predicted to be inundated during each design flood (regardless of whether above floor flooding is predicted or not).

The results presented in **Table J1** shows that during the 1% AEP flood, 22 buildings within the Smithfield West catchment are predicted to be inundated above floor level. **Table J6** also indicates that a further 30 buildings have less than 0.3 metres freeboard. This outcome is generally consistent with the results documented in the *"Smithfield West Drainage Study: Chifley Street to The Horsley Drive"* (Dalland & Lucas, 1996) where 18 properties were predicted to be inundated above floor level during the 1% AEP flood. The slightly lower number of properties documented in the 1996 Drainage Study is likely associated with this study only considering the catchment area between Chifley Street and The Horsley Drive.

Table J1 shows that Victoria Street has the highest number of buildings inundated above floor level during the 1% AEP floods (7 buildings), followed by Hart Street (4 buildings) and Moir Street (2 buildings). **Tables J2** to **J4** also shows that residential properties are the most significantly impacted building category within the catchment, accounting for 95% of the properties subject to over floor flooding.

An assessment was also undertaken to quantify the impact that climate change induced increases in rainfall intensity may have on the number of properties subject to over floor flooding during the 1% AEP flood. The outcomes of this assessment are presented in **Tables J7** to **J10** and shows that:

- 8 additional buildings are likely to be inundated above floor level if a 10% increase in rainfall intensity occurs,
- 14 additional buildings are likely to be inundated above floor level if a 20% increase in rainfall intensity occurs; and,
- 20 additional buildings are likely to be inundated above floor level if a 30% increase in rainfall intensity occurs.

This outcome shows that climate change has the potential to cause a significant increase in the number of properties that may be subject to over floor flooding.

9.3 Flood Damage Costs

9.3.1 General

As outlined in the previous section, a number of properties have the potential to be inundated during each of the simulated design floods. This is likely to cause a significant inconvenience to those living and working in the catchment, but also has the potential to impose a significant financial burden if buildings and contents are inundated / damaged by floodwaters. In order to quantify the potential cost of flooding across the Smithfield West catchment, flood damage calculations were prepared.

The costs associated with flooding can be broken down into a number of categories, as shown in **Plate 28**. However, broadly speaking, flood damage costs fall under two major categories;

- tangible damages; and
- intangible damages.

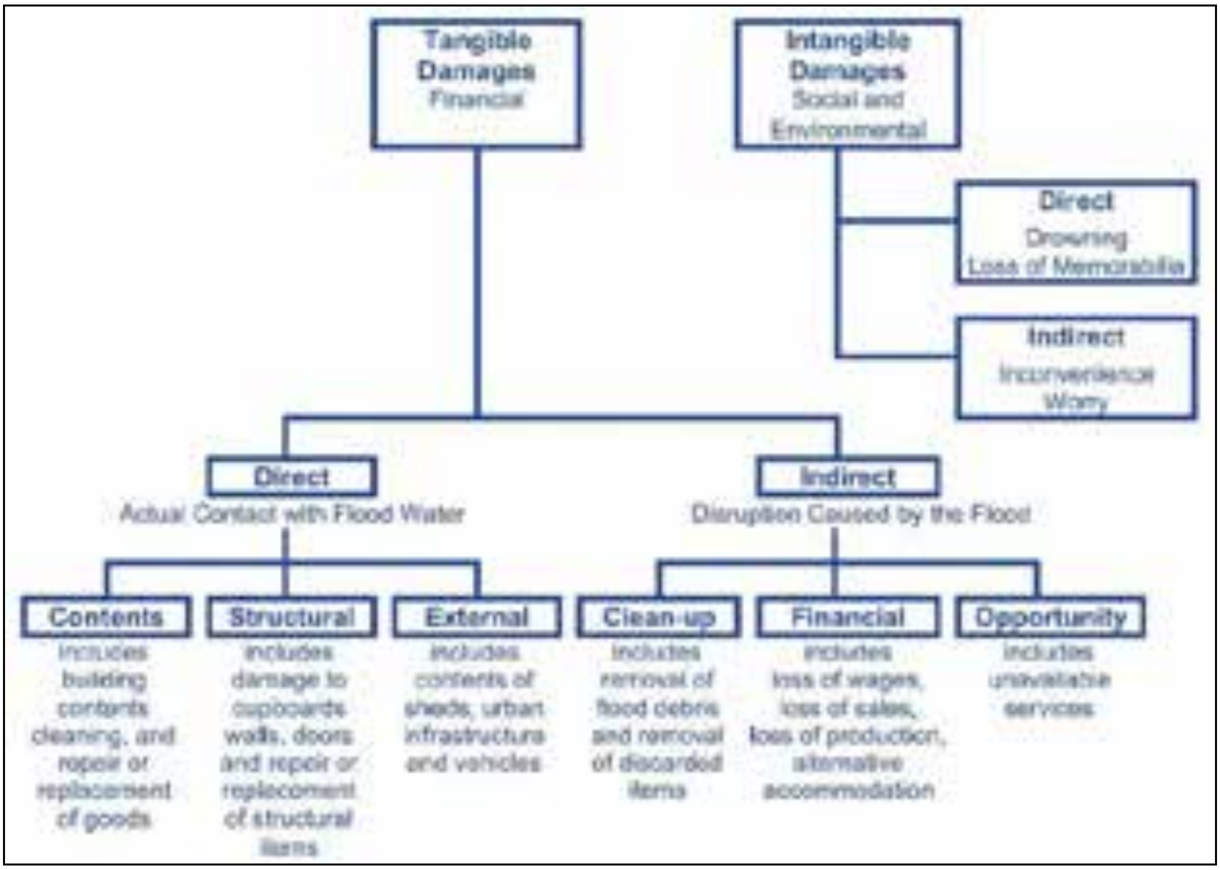


Plate 28 Flood Damage Categories (NSW Government, 2005)

Tangible damages are those which can be quantified in monetary terms (e.g., cost to replace household items damaged by floodwaters). Intangible damages cannot be as readily quantified in monetary terms and include items such as inconvenience and psychological stress.

Tangible damages can be further broken down into direct and indirect damage costs. Direct costs are directly related to the impact of the floodwaters such as replacement of contents, as well as damage to structures such as houses and any external items such as garden sheds and vehicles. Direct costs are typically calculated using depth-damage curves.

Indirect flood damage costs are costs incurred following the flood. This includes loss of wages, loss of trade (for commercial/industrial properties), and/or alternate accommodation costs.

Due to the difficulty associated with quantifying intangible flood damages, Council only requested that tangible flood damages be quantified as part of the flood study. It is understood that the flood damage calculations will be revisited during the floodplain risk management study.

9.3.2 Property Database

A property database was developed as part of the study to enable flood damages calculations to be completed. The database was developed in GIS and included all habitable (i.e., residential, commercial and industrial) buildings located within the PMF extent. The following information was included as additional fields within the database for each building:

- Generic property type (i.e., residential, commercial or industrial);
- Building floor level – refer to Section 9.2.1 for floor level estimation technique;
- Building floor area;
- Residential building type (i.e., two story, single level high set or single level low set); and,
- Commercial or industrial property contents value (low, medium or high value).

The information contained in the property database was used with the design flood level information and depth-damage curves to establish a tangible flood damage estimate for each building located within the Smithfield West catchment for each design flood. Further information on how the depth-damage curves were derived for residential, commercial and industrial properties is provided below.

9.3.3 Flood Damage Calculations

Residential

The NSW Office of Environment and Heritage (OEH) has prepared a spreadsheet that provides a standardised approach for deriving depth-damage curves for residential properties (version 3.00, October 2007). The spreadsheet requires a range of default parameters to be defined to enable a meaningful damage estimate to be derived that is appropriate for the local catchment. The default parameters that were adopted for the Smithfield West catchment are summarised in **Table J11** in **Appendix J**.

It was noted that the resulting depth-damage curves incorporate a damage allowance for negative depths. This is intended to reflect the fact that property damage can be incurred when the water level is below floor level (e.g., damage to fences, sheds, belongings stored below the building floor). The damage curves for 'single storey low set' and 'two storey' properties commence at -0.5 metres, which was considered to be appropriate for the catchment. However, the 'single storey high set' damage curves commenced at -5 metres, which was considered to be too high for the catchment. In order to verify this, single storey high set building floor levels within the PMF extent were compared against the minimum ground elevation within each lot (i.e., the minimum elevation within each lot at which inundation will first occur). This determined that the median difference between the building floor level and minimum ground level within the corresponding lot was 1.22 metres. Accordingly, the 'single-storey high set' damage curves were adjusted so that damage commenced when the flood level was less than 1.3 metres below the floor level.

As noted in Section 9.2.2, building floor areas were calculated. The building floor area serves as one of the residential damage curve inputs. Typically a single representative floor area is used to derive representative residential damage curves. However, an inspection of the floor areas showed that it was difficult to assign a single, representative building floor area due to large array of floor sizes. Therefore, a statistical analysis was performed on the floor areas and it was determined a more meaningful damage estimate could be developed by grouping the buildings into either a 120 m^2 or 190 m^2 floor area group. Accordingly, separate damage curves were developed for both the 120 m^2 and 190 m^2 floor area groups.

The OEH flood damage calculation spreadsheet includes allowances for the following flood damage components:

- Damage to building contents (direct cost);
- External damage (e.g., cars, sheds, fences, landscaping) (direct cost);
- Clean up costs (indirect cost); and,
- Alternate accommodation costs while clean up occurs (indirect cost).

As outlined above, the OEH residential depth-damage curves include allowances for both direct and indirect flood damage costs. However, the indirect damage costs are included as 'lump sum' values and do not make an allowance for the area of land subject to inundation or the likely variation in the length of time required in alternate accommodation across the catchment. For example, a property that is subject to minimal inundation and with no over floor flooding is likely to incur less clean-up costs and not require significant time in alternate accommodation relative to a property that is completely inundated and subject to over floor flooding. Accordingly, it was considered that a more reliable description of the flood damage costs could be prepared by:

- Calculating direct flood damage costs using the OEH spreadsheet; and,
- Calculating indirect flood damage costs using the TUFLOW model outputs in conjunction with GIS analysis of building and property information.

Accordingly, the indirect damage components were removed from the OEH spreadsheet (the resulting 'direct' depth-damage curves are presented in **Figure J1** in **Appendix J**). The indirect damage components were calculated using the following approach:

- Properties that were <u>completely</u> inundated were assigned a clean-up cost of $5,600 (adjusted to 2015 dollars from the $4,000 suggested in the OEH spreadsheet). The clean-up cost for properties not subject to complete inundation were adjusted based on the proportion of the property that was inundated. For example, a property that was subject to 50% inundation would be assigned a clean-up cost of $5,600 x 0.5 = $2,800.
- Loss of wages / sales associated with the need to stay at home and assist with cleaning were calculated assuming an average weekly household income of $1,040 (ABS, 2015) and:
  - 1 week required to clean a property where no over floor flooding was experienced;
  - 2 weeks required to clean a property where over floor flooding was experienced.
- If over floor flooding was predicted, it was assumed that alternate accommodation costs would be incurred for 1 week. 1 week of alternate accommodation was valued at $310, based on Australian Bureau of Statistics property rental statistics for Smithfield (ABS, 2015).

Commercial

Unlike residential flood damage calculations, there are no standard damage curves available for estimating commercial flood damages in NSW. Commercial property types include offices, retailers and shops.

To help ensure consistency with commercial flood damage estimates derived for other catchments within the Fairfield LGA, depth-damage curves developed as part of the *'Georges River Floodplain Management Study and Plan'* (Bewsher Consulting, 2004a) and *'Cabramatta Creek Floodplain Management Study and Plan'* (Bewsher Consulting, 2004b) and subsequently used in the *'Prospect Creek Floodplain Management Plan Review'* (Bewsher Consulting, 2010) were used to define commercial flood damages for the Smithfield West catchment. However, depth-damage curves were updated from 2004 dollars to 2015 dollars using Consumer Price Index (CPI) values published by the Australian Bureau of Statistics (ABS) before application to the Smithfield West catchment.

As noted in Section 9.3.2, each commercial property was classified according to the value of the contents (i.e., low, medium and high damage potential). This is intended to reflect the fact that the damage incurred across commercial properties is likely to be directly related to the value of its contents. **Table 25** provides a summary of the different commercial property types and the associated contents value that each would fall under.

Table 25 Content Value Categories for Commerical Property Types

Low Value Contents	Medium Value Contents	High Value Contents
Small cafes	Food stores	Electrical shops
Florists	Grocers	Chemists
Offices	Corner stores / mixed business	Shoe Shops
Consulting rooms	Take away food	Clothing stores
Post office	Cake shops	Bottle shops
Pet shops	Hairdressers	Bookshops
Churches	Banks	Newsagents
Laundrettes	Dry cleaners	Sporting goods
Public halls	Professions (e.g., solicitors)	Furniture
	Small hardware	DVD rental
	Small retail	Kitchenware
		Restaurants
		Schools

The adopted commercial depth-damage curves are presented in **Figure J2** in **Appendix J**.

No specific allowance is included in the commercial damage curves for indirect losses, such as clean-up costs and loss of income while clean-up occurs. The *'Prospect Creek Floodplain Management Plan Review'* (Bewsher Consulting, 2010) estimated indirect commercial damage costs as 20% of direct damage costs. To ensure consistency with these previous studies, the same factor was adopted for the current study.

Industrial

As for commercial properties, no standard depth-damage curves are available for industrial properties (e.g., warehouses, automotive repairs) in NSW. The industrial depth-damage curves developed for the *'Georges River Floodplain Management Study and Plan'* (Bewsher Consulting, 2004a) and *'Cabramatta Creek Floodplain Management Study and Plan'* (Bewsher Consulting, 2004b) were initially investigated for application to the Smithfield West catchment. However, it was noted that the industrial damage curves developed for the previous studies were only intended for application to industrial buildings up to 1,000 m^2 in size. The Smithfied West catchment comprises some industrial buildings that exceed

5,000 m^2. Although consistency with the previously adopoted studies was desired it was considered that industrial buildings greater than 1,000 m^2 in size would have the potential to incur higher flood damage costs relative to smaller industrial properties. Accordingly, a modified approach for estimating industrial flood damages was derived that takes into account the size of building.

Specifically, for industrial buildings up to 1,000 m^2 in size, the industrial depth-damage curves developed and adopted as part of the previous studies were retained (but were adjusted to 2015 dollars). For those buildings in excess of 1,000 m^2, the depth-damage curves were pro-rated based on the floor area (i.e., a building with a 2,000 m^2 floor area would incur twice as much damage as a building with a 1,000 m^2 area for an equivalanet above floor water depth). The pro-rated cost per unit floor area relationships that were adopted are sumamrised in **Table 26**.

Table 26 Flood depth damage relationships for industrial properties

Depth	Total Direct Costs (for floor areas up to 1,000m^2)			Cost / m^2 * (for floor areas over 1,000m^2)		
	Low Value	**Med. Value**	**High Value**	**Low Value**	**Med. Value**	**High Value**
0	$0	$0	$0	$0.00	$0.00	$0.00
0.20	$23,460	$48,300	$96,600	$23.46	$48.30	$96.60
0.25	$28,980	$56,580	$111,780	$28.98	$56.58	$111.78
0.30	$31,740	$63,480	$128,340	$31.74	$63.48	$128.34
0.50	$48,300	$96,600	$191,820	$48.30	$96.60	$191.82
0.60	$56,580	$111,780	$223,560	$56.58	$111.78	$223.56
0.75	$71,760	$136,620	$271,860	$71.76	$136.62	$271.86
0.90	$80,040	$160,080	$320,160	$80.04	$160.08	$320.16
1.00	$88,320	$176,640	$343,620	$88.32	$176.64	$343.62
1.20	$96,600	$200,100	$400,200	$96.60	$200.10	$400.20
1.25	$103,500	$208,380	$416,760	$103.50	$208.38	$416.76
1.50	$111,780	$223,560	$440,220	$111.78	$223.56	$440.22
1.75	$120,060	$231,840	$463,680	$120.06	$231.84	$463.68
2.00	$128,340	$240,120	$480,240	$128.34	$240.12	$480.24
>2.00	$131,100	$241,500	$483,000	$131.10	$241.50	$483.00

NOTE: * Cost per square metre was calculated by dividing the total cost by a floor area of 1000m^2

As with commercial properties, the industrial properties must be classified according to the value of the building contents (i.e., low, medium and high value). **Table 27** provides a summary of the different industrial property types and the associated contents value that each would fall under.

Table 27 Content Value Categories for Industrial Property Types

Low Value Contents	Medium Value Contents	High Value Contents
Automotive repairs	Equipment hire	Smash repairs
Sand, gravel & cement	Food distribution	Panel beating
Storage	Leather & upholstery	Car yard sales
Transport & couriers	Carpet warehouses	Vehicle showrooms
Paving & landscaping	Agricultural equipment	Service stations
Fuel depots	Truck yards	
Council & Governments depots	Vacant factories	
Chemical storage		
Pool products		
Sale yards		
Plumbing supplies		

The final industrial depth-damage curves are presented in **Figure J3** in **Appendix J**.

Indirect flood damage costs for industrial properties were calculated as 20% of direct costs in line with the approach adopted for the *"Prospect Creek Floodplain Management Plan Review"* (Bewsher Consulting, 2010).

Infrastructure Damage

Infrastructure damage refers to damage to public infrastructure and utilities such as roads, water supply, sewerage, gas, electricity and telephone. Infrastructure damage has been estimated at 15% of the total direct residential, commercial and industrial damages. This value was extracted from the *"Prospect Creek Floodplain Management Plan Review"* (Bewsher Consulting, 2010).

Potential versus Actual Flood Damages

The flood damage calculations outlined above are damages based on a 'do nothing' scenario. However, building occupants may be able undertake measures to minimise flood damage if they are provided with sufficient advance warning of an impending flood (and assuming they are home at the time of flood). Flooding across the Smithfield West catchment is typically associated with relatively short rainfall bursts with little warning time. As a result, it was considered that there would be limited opportunity for residents and business owners to minimise damages and no adjustment was taken to adjust the potential flood damages to actual flood damages.

9.3.4 Summary of Flood Damage Costs

Flood damages were calculated using the interpolated flood level surfaces for each design flood in conjunction with the appropriate depth-damage curves and floor levels for each building. Calculated flood damages for each design flood are summarised in **Table 28** (note that damages are provided in thousands of dollars). Total direct flood damages have also been accumulated for each street in the study area and are provided in **Table J15** in **Appendix J**.

Table 28 Summary of Flood Damages for Existing Conditions

Flood Damage Component		Flood Damages (thousands of 2015 dollars)							
		50% AEP	20% AEP	10% AEP	5% AEP	1% AEP	0.2% AEP	1 in 10,000 ARI	PMF
Residential	Direct	$288	$775	$1,123	$2,036	$2,826	$3,772	$5,517	$12,356
	Indirect	$290	$529	$613	$739	$978	$1,124	$1,428	$2,543
Commercial.	Direct	-	-	-	-	$1	$9	$117	$1,526
	Indirect	-	-	-	-	-	$2	$23	$305
Industrial	Direct	-	-	-	-	-	$11	$34	$115
	Indirect	-	-	-	-	-	$2	$7	$23
Infrastructure		$43	$116	$168	$305	$424	$569	$850	$2,100
TOTAL		$622	$1,421	$1,904	$3,080	$4,229	$5,488	$7,977	$18,968

The results presented in **Table 28** shows that a 1% AEP flood has the potential to cause over $4 million dollars of damages. The majority of the damage costs occur across residential properties with Hinkler Street, Canara Place, Cartella Crescent and Moir Street incurring the highest damage cost per inundated property.

The total flood damages for each design flood were plotted on a chart against the probability of each flood occurring (i.e., AEP) (refer **Plate 29**). The chart was then used as the basis for calculating the average annual damages (AAD) for the Smithfield West catchment (areas under the curve). The AAD provides an estimate of the average annual cost of flooding across the catchment over an extended timeframe. The AAD for the Smithfield West catchment was determined to be $920,000.

A breakdown of the relative contribution of each damage component to the AAD is provided in **Plate 30**. It shows that direct and indirect residential damages are the largest contributor to the average annual damage cost.

Flood damage calculations were also completed to quantify the potential impact that climate change induced rainfall increases may have on 1% AEP flood damages in the future. The outcomes of this assessment are presented in **Tables J16** and **J17** in **Appendix J** and indicates that:

- A 10% increase in rainfall is likely to cause a 9% increase in 1% AEP flood damages
- A 20% increase in rainfall is likely to cause an 18% increase in 1% AEP flood damages
- A 30% increase in rainfall is likely to cause a 27% increase in 1% AEP flood damages

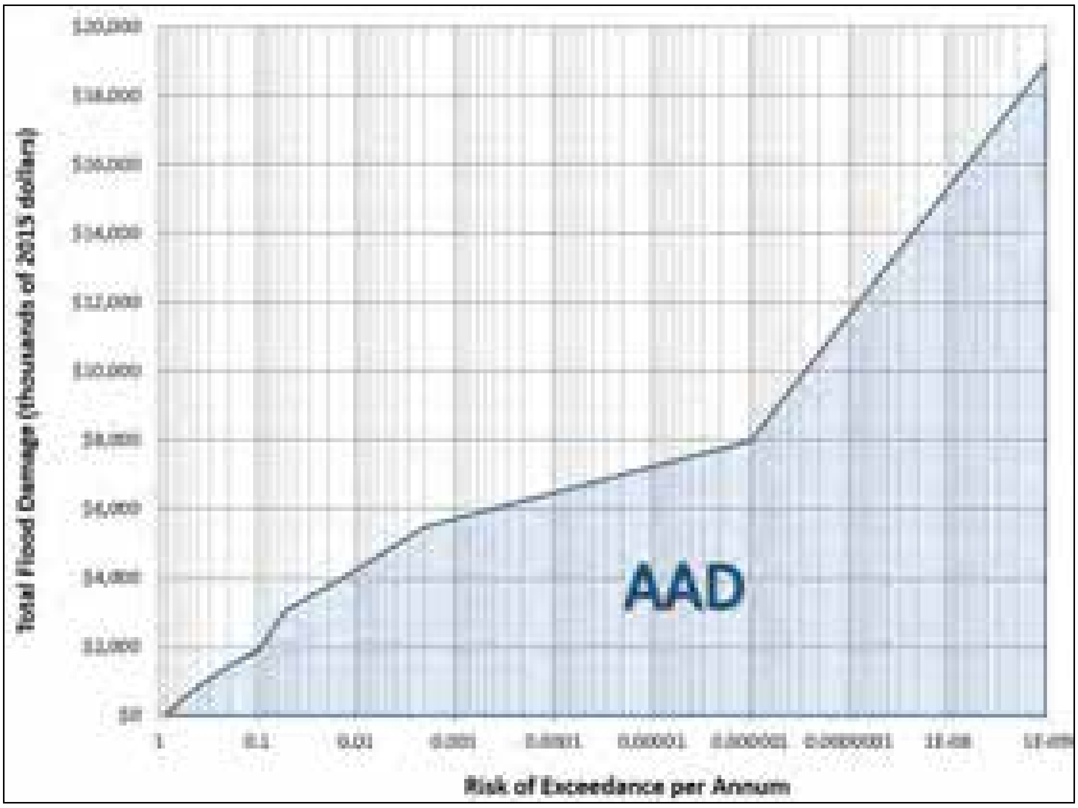


Plate 29 Average Annual Damages (AAD) for the Smithfield West Catchment

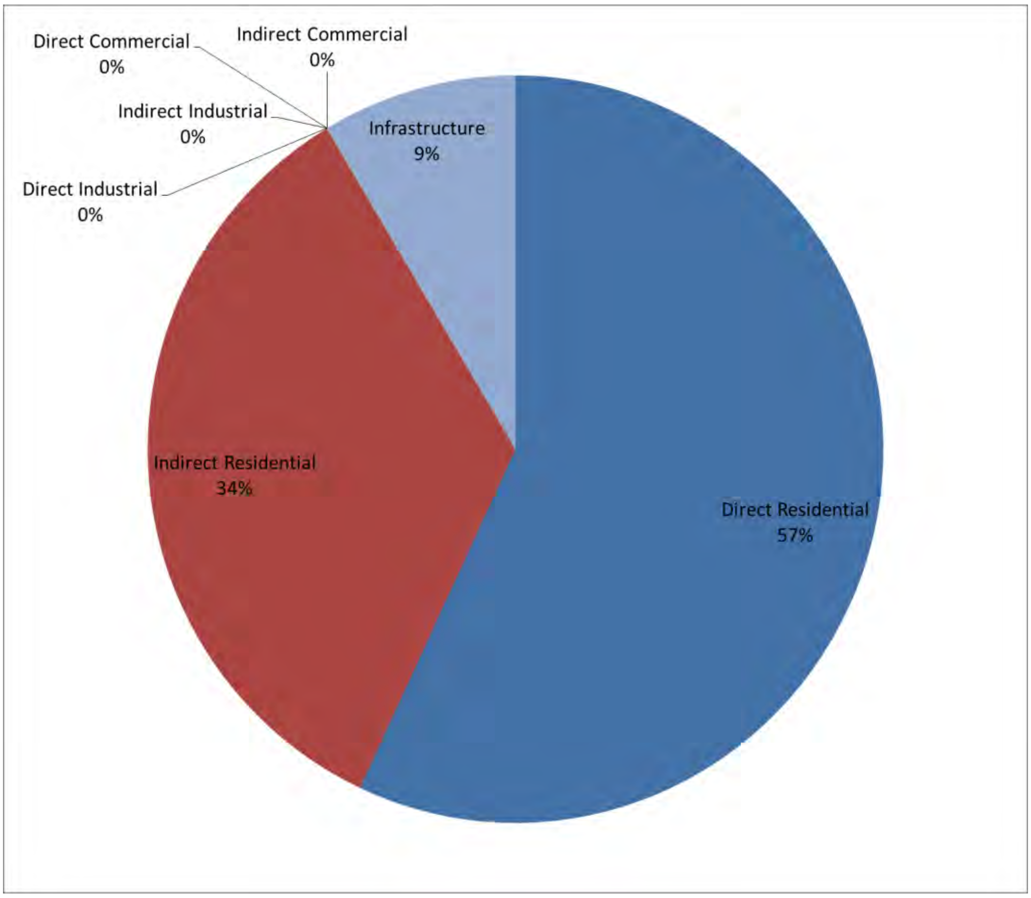


Plate 30 Component breakdown of the Average Annual Damages (AAD) for the Smithfield West Catchment

9.4 Impact of Flooding on Key Facilities

9.4.1 Key Infrastructure

The majority of the Smithfield West catchment comprises residential properties. Nevertheless, there is some key infrastructure that may play an important role in emergency response management during floods. Furthermore, there are some critical facilities located within the catchment that may require special emergency assistance during future floods (e.g., schools). As such, it was considered important to assess the impact of flooding on these facilities to determine their suitability for use during floods and/or determine if special emergency assistance may be required.

Key facilities located within the Smithfield West catchment and the associated flood impacts are listed below and in **Table 29**:

- **Fire Stations**:
  - ***Smithfield Fire Station*** (875 The Horsley Drive, Smithfield): adjoins the western boundary of the Smithfield West catchment (near Gipps Street). The property remains flood free in all design events. However, access roads around the area are cut during a range of design events, including The Horsley Drive which may prevent access to/from the fire station (refer Section 9.4.2 for further information).
- **Schools**:
  - ***Smithfield West Public School*** (Wetherill Street, Wetherill Park): is located in the south-western corner of the Smithfield West catchment. Approximately half of the school (generally containing open space) is contained within the Smithfield West catchment. The property experiences some inundation in the PMF, however, this is restricted to the eastern boundary of the school and is not predicted to impact on any school buildings. As a result, it should be possible to evacuate from the school via Wetherill Street, which is located near to the catchment boundary, if needed.

Table 29 Impact of Flooding on Key Infrastructure

Key Infrastructure		1% AEP Flood		PMF	
		Inundated ?	Access Cut?	Inundated ?	Access Cut?
Fire Stations	*Smithfield Fire Station* (875 The Horsley Drive, Smithfield)	-	-	-	-
Police Stations		There are no police stations located within the catchment			
State Emergency Service		There are no SES buildings located within the catchment			
Ambulance Stations		There are no ambulance stations located within the catchment			
Hospitals		There are no hospitals located within the catchment			
Schools	*Smithfield West Public School* (Wetherill Street, Wetherill Park)	-	-	☑	-
Aged Care Facilities		There are no aged care facilities located within the catchment			

9.4.2 Transportation Links

There are several major roadways within the Smithfield West catchment which may be required for evacuation or emergency services access during floods. It is important to have an understanding of the impacts of flooding on these roads so that appropriate emergency planning can occur. A summary of major roadways and the level of impact is provided in **Table 30** and is also discussed below:

- **Brenan Street**: Brenan Street provides access across the catchment from the Cumberland Highway and is predicted to experience depths of up to 0.2 m in the 1% AEP flood and 0.7 m in the PMF. It is predicted to remain serviceable during the 50% AEP to the 5% AEP floods.
- **Neville Street***:* Neville Street experiences depths of over 1 metre during the PMF near the intersection of Dublin Street. During floods > 50% AEP event, the east bound lane is predicted to be cut, however, the west bound lane would remain trafficable.
- **The Horsley Drive***:* The Horsley Drive is a major east-west thoroughfare that provides access to/from the Cumberland Highway. The roadway comprises two lanes for each direction of travel. Over 1 metre of water is predicted to cover all lanes during the PMF. During the 1% AEP flood, depths in excess of 0.4m are predicted indicating that travel would not be possible. The more elevated, central travel lanes are likely to remain serviceable during events up to and including the 5% AEP flood.
- **Victoria Street**: Victoria Street is also a major east-west roadway passing through the Smithfield West catchment. It comprises two lanes in each direction of travel separated by a grassed medium strip. The median strip serves as a significant impediment to flow across Victoria Street. As a result of the 'build up' of water against the median strip, the west-bound lanes are predicted to be exposed to higher depths relative to the east-bound lanes. The west-bound lanes are predicted to be cut by floodwaters during events as frequent as the 50% AEP (maximum depth = 0.6 metres). However, one east-bound lane is likely to remain trafficable during events less severe than the 20% AEP flood.
- **Dublin Street**: Dublin Street extends north-south through the catchment, linking the major roadways described above. It is predicted to experience peak depths of up to 0.4 metres during floods as frequent as the 50% AEP event. However, this flooding is generally concentrated in the immediate vicinity of Neville Street.

Table 30 Impact of Flooding on Key Transportation Links

Roadway	Access Cut During 50% AEP Flood?	Access Cut During 20% AEP Flood?	Access Cut During 1% AEP Flood?	Access Cut During PMF?
Brenan Street			☑	☑
Neville Street		☑	☑	☑
The Horsley Drive			☑	☑
Victoria Street	☑	☑	☑	☑
Dublin Street	☑	☑	☑	☑

Although a significant number of streets are predicted to be cut during relatively frequent floods, the 'duration of inundation' information presented in **Figures 18** and **19** indicates that the majority of roadways would be cut for less than 2 hours (Victoria Street is the only roadway predicted be cut for more than 2 hours during the 1% AEP flood). **Figures 18** and **19** also show that there would generally be less than 30 minutes before most roadways would become cut after the initial onset of rainfall.

10 Public Exhibition of Draft Overland Flood Study

10.1 General

The Draft Overland Flood Study Report and associated Flood Risk Precinct Maps were exhibited from 12 October 2015 to 2 November 2015 at Fairfield City Council's Administration Centre and on Council's website under Public Exhibition with a link to http://www.SmithfieldWest.FloodStudy.com.au. The exhibition was also advertised in the Fairfield Champion on 7 and 14 October 2015.

A total of 718 owner occupiers, landlords and tenants who were identified as being within the floodplain were notified via letter to inform them of the public exhibition and to provide the opportunities to comment. A list of answers to frequently asked questions were also included in the letter. This information explained the significance of the study and pre-empted queries that the study may raise.

Property owners and tenants were invited to contact or visit Council for further information and/or to discuss the Overland Flood Study.

10.2 Comments from Public Exhibition

The flood study website was visited 80 times during the public exhibition. A total of 8 individual comments were received. All comments were collated and reviewed. The majority of respondents were concerned about the following:

1. The impact of flood notation on the property value;
2. Insurance Premium;
3. Development Control;
4. Low Flood Risk (PMF) mapping;
5. Next Stage (i.e., Floodplain Risk Management Study); and
6. Options to mitigate flood impacts to property.

Based on the review of individual responses during the Public Exhibition, no changes to the Smithfield West Overland Flood Study were deemed necessary.

11 Conclusion

This report documents the outcomes of investigations completed to quantify flood behaviour across the Smithfield West catchment. It provides information on design flood discharges, levels, depths and velocities as well as hydraulic and flood hazard categories for a range of design floods.

Flood behaviour across the catchment was defined using a two-dimensional integrated hydrologic/hydraulic computer model that was developed using the TUFLOW software.

The computer model was calibrated/verified using historic rainfall and flood marks for floods that occurred in 1990 and 2012. The model was subsequently used to simulate the 50%, 20%, 10%, 5%, 1% and 0.2% AEP events floods as well as the 1 in 10,000 year ARI flood and PMF. The following conclusions can be drawn from the results of the investigation:

- Flooding across the Smithfield West catchment generally occurs as a result of the capacity of the stormwater system being exceeded following heavy rainfall in the catchment leading to 'overland' flooding. However, flooding across the downstream sections of the catchment can also occur as a result of floodwaters overtopping the bank of Prospect Creek.
- The trunk drainage system was determined to have limited capacity (i.e., less than 50% AEP capacity in most instances). Accordingly, overland flooding is predicted to occur relatively frequently.
- Overland flooding typically occurs as result of relatively short duration, high intensity rainfall bursts. This type of storm system is most typically associated with thunder storms. The critical storm duration for those areas subject to overland flooding was determined to be 2 hours.
- Although a number of properties are predicted to be inundated during each of the simulated design floods, the depths of inundation are typically shallow. As a result, most areas are subject to a low provisional flood hazard during the 1% AEP flood (the high hazard areas are primarily restricted to roadways).
- At the peak of the 1% AEP flood, 261 properties are predicted to experience depths of inundation that exceed 0.15 metres. 22 of these properties are predicted to be flooded over floor. The areas that are most significantly impacted by floodwaters include:
  - Dublin St / Neville St intersection
  - Cartella Crescent
  - Canara Place
  - The Horsely Drive
  - Moir Street
  - Hart Street
  - Victoria St
  - Hinkler St
  - Chifley Street

- The average annual cost of flooding across the Smithfield West catchment is predicted to be $920,000.
- Major flooding within the catchment is likely to cut a number of roadways. However, the roadways would typically be cut for less than 2 hours during most floods.

The Draft Overland Flood Study Report and associated Flood Risk Precinct Maps were on Public Exhibition from 12th October 2015 to 2nd November 2015. A total of 8 individual comments were received.

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13 Glossary

acid sulphate soils — are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.

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. Eg, 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 events 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.

caravan and moveable home parks — caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the Local Governments Act.

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 OEH, as having the function to determine an application.

development	is defined in Part 4 of the Environmental Planning and Assessment Act (*EP&A Act*). <u>infill development:</u> refers to 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. <u>new development:</u> 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. <u>redevelopment:</u> 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).
ESD	Ecologically Sustainable Development (ESD) using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act, 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before 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	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 and 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 PMF event. Note that the term flood liable land covers the whole floodplain, not just that part below the FPL (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 SES.
flood planning area	the area of land below the FPL and thus subject to flood related development controls.
flood planning levels (FPLs)	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.

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	land susceptible to flooding by the PMF event. Flood prone land is synonymous with flood liable land.
flood readiness	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. <u>existing flood risk</u>**:** the risk a community is exposed to as a result of its location on the floodplain. <u>future flood risk</u>: the risk a community may be exposed to as a result of new development on the floodplain. <u>continuing flood risk</u>: 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 flow, or a significant increase in flood levels.
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 study the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in Appendix L of the *Floodplain Development Manual* (*2005*).

historical flood	a flood which has actually occurred.
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	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. 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 0.3m (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 flowpaths 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.

merit approach

the merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well-being of the State's rivers and floodplains.

The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into council plans, policy, and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local flood risk management policy and EPIs.

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.

<u>minor flooding</u>: 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.

<u>moderate flooding</u>: Low lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered.

<u>major flooding</u>: 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.

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 annual exceedance probability*).

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 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.

TUFLOW is a 1-dimensional and 2-dimensional flood simulation software. It simulates the complex movement of floodwaters across a particular area of interest using mathematical approximations to derive information on floodwater depths, velocities and levels.

velocity the speed or rate of motion (*distance per unit of time, e.g., metres per second*) in a specific direction at which the flood waters are moving.

water surface profile a graph showing the flood stage at any given location along a watercourse at a particular time.

wind fetch the horizontal distance in the direction of wind over which wind waves are generated.

APPENDIX A

HISTORIC FLOOD MARKS

Table A1: Historic Flood Marks for 1990 and 2012 Floods

Address	Floor Level	Feburary 1990 Flood Level recorded	April 2012 Flood Level recorded
85 Market St	22.03		22.53
823 The Horsley Drive	30.61	30.47	
819 The Horsley Dr	30.83		29.86
76 Dublin St	33.27	33.10	
66 Chiefley Street	25.20	25.32	
49 Rhondda Street	24.14	23.93	
39 Hinker Street	25.15	25.41	
38 Hinkler Street Front	26.78	26.12	
38 Hinkler Street Back	26.78	26.43	
35 Hinkler St	25.87		25.05
33 Hinkler St	26.07		25.16
32 Moir Street	30.47	30.05	
31 Hinkler Street	25.42	25.70	
30 Hinkler Street	27.06	27.01	
27 Moir Street	29.28	29.13	
26 Moir St	29.60		29.18
25 Moir St	29.02		28.89
21 Moir Street	28.89	28.89	
19 Moir Street	28.94	28.94	
142 Victoria Street	27.59	27.63	
14 Hart St	28.96		28.98
139B Victoria St	28.62		28.56
139B Victoria St	28.62		28.49
137C Victoria St	28.45		28.19
130 Victoria St	27.30		27.11
128 Victoria St	27.54		27.24
12 Hart St	28.89		28.81
10 Hart Street	28.75	28.98	

APPENDIX B

COMMUNITY CONSULTATION

How You Can Help

The computer models developed for the flood study will be calibrated and tested against historic flood information at various locations across the catchment. Therefore, any flood photographs, videos and descriptions of flood depths / heights that you can provide will assist with calibrating the model.

Enclosed with this brochure is a questionnaire that aims to collect as much historic flood information as possible to assist with the model calibration. You are encouraged to complete the questionnaire and return it by 14 November 2014. Alternatively, the questionnaire can be completed online via the flood study website:

www.fairfieldcity.nsw.gov.au/haveyoursay

Further Information

To obtain further information on the Smithfield West Overland Flood Study or to submit any information that you think may be valuable to the study, please contact:

Catchment Simulation Solutions

David Tetley
Catchment Simulation Solutions
Suite 2.01, 210 George Street
Sydney NSW 2000
(02) 9247 4882
dtetley@csse.com.au

Fairfield City
Celebrating diversity

Darren Ikin
Fairfield City Council
PO Box 21
Fairfield NSW 1860
(02) 9725 0265
dikin@fairfieldcity.nsw.gov.au

Alternatively, you can visit the flood study website:
www.fairfieldcity.nsw.gov.au/haveyoursay

Community Water Program
Our Water, Our Community, Our Future

Smithfield West Overland Flood Study

Information Brochure

Fairfield City Council is preparing an overland flood study for the Smithfield West catchment. This brochure provides an overview of the flood study and outlines how you can help.

Fairfield City
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Introduction

Fairfield City Council is in the initial stages of preparing an overland flood study for the Smithfield West catchment. The extent of the catchment is shown below.

During most rainfall events, runoff is carried by the stormwater system into Prospect Creek. During periods of heavy rainfall there is potential for the capacity of the stormwater system to be exceeded leading to overland flooding. Significant overland flooding has occurred in this catchment on a number of occasions in the past including 1990 and 2012.

As shown in the image on the right, overland flooding can cut roadways and inundate properties. This can result in damage to garages, sheds and even homes. It can also place lives at risk.

Council has undertaken considerable work over the last 30 years to help reduce the frequency and severity of flooding across Fairfield Local Government Area. This includes stormwater upgrades, house raising and development controls. Despite these improvements, a number of areas remain at risk of flooding. Therefore, Council is preparing an overland flood study to help identify the flooding problem areas.

The information generated as part of the flood study will allow Council to identify where additional flood mitigation measures (e.g. stormwater upgrades) may be best implemented to further reduce the impact of flooding on property owners across the Smithfield West catchment.

What is a Flood Study?

The primary objective of the flood study is to identify the nature and extent of the existing flooding problem. This will be primarily achieved through the development of a computer flood model, which will be used to quantify the capacity of the stormwater system and simulate how overland flow would move through the catchment. An example of a floodwater depth and velocity map that is produced by a computer flood model is shown below.

Council has commisioned specialist flood consultants, Catchment Simulation Solutions, to prepare the flood study.

Section 3 - Additional Flood Information

1. How fast do floodwaters typically move in your area?

- ☐ Stationary
- ☐ Walking pace
- ☐ Running pace

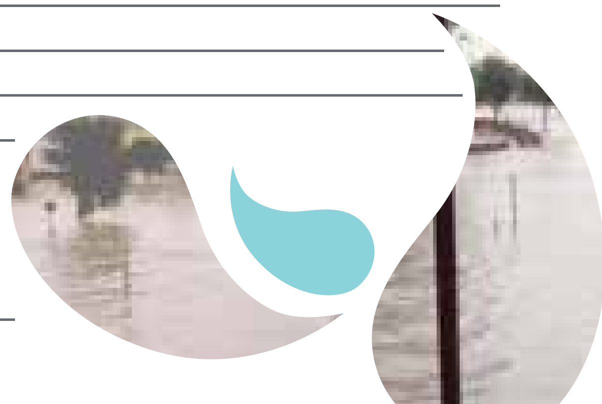
2. In your opinion, what is the main cause of flooding in the Smithfield West catchment (e.g., stormwater system blockage, stormwater capacity, obstructions to overland flow - fences, garages)?

3. Do you keep rainfall records from any past floods?

☐ Yes ☐ No

If you answered Yes, could we obtain a copy of the records to assist with the calibration of the computer model?

4. Do you have any other comments or information that you think would be useful for this investigation?



Community Water Program

Our Water, Our Community, Our Future

Smithfield West Overland Flood Study

Questionnaire

This questionnaire has been prepared to assist Fairfield City Council in better understanding the flooding "trouble spots" across the Smithfield West catchment and to assist in the calibration and testing of a computer flood model that will be developed as part of the Smithfield West Flood Study.

The following questionnaire should only take around 10 minutes to complete. Try to answer as many questions as possible and give as much detail as possible (attach additional pages if necessary).

Once complete, please return the questionnaire via email or mail (no postage stamp required) by 14 November 2014. Alternatively, if you have internet access, an online version of the questionnaire can be completed at:
www.fairfieldcity.nsw.gov.au/haveyoursay




FairfieldCity
Celebrating diversity

Section 1 - General Information

Can you please provide the following contact details in case we need to contact you for additional information? Note that answering this question is optional. If you do provide contact details, this information will remain confidential at all times and will not be published (refer to privacy statement at the bottom of this page).

Name: ____

Address: ____

Phone No. ____

Email: ____

Please tick ☑ the best answer to the following questions.

1. What type of property is this?

☐ Residential

☐ Commerical

☐ Industrial

☐ Vacant Land

☐ Other (Please specify:________________________________)

2. What is the occupier status of the property?

☐ Owner occupied

☐ Rental property

☐ Business

☐ Other (Please specify:________________________________)

3. How long have you lived / worked in the area:

(a) At this address? ______________________________

(b) In the Smithfield area? __________________________

PROTECTING YOUR PRIVACY – The personal information requested on this form will only be used for the Smithfield West Overland Flood Study. The supply of this information by you is voluntary. Council is regarded as the agency that holds the information and will endeavour to ensure that this information remains secure, accurate and up-to-date. Access to information is restricted to Council Officers and other authorised people. You may make applications for access to information held by Council. You may also request an amendment to information held by Council. Should you require further information please contact Fairfield City Council.

Section 2 - Flood History

1. As far as you know, has your property ever been affected by flooding?

☐ Yes

☐ No (If you answered No, please go to Question 3 on final page)

2. How were you impacted by flooding (you can select more than one option)?

☐ Traffic was disrupted

☐ My front / back yard was flooded

☐ My garage was flooded

☐ My house was flooded

☐ Other (Please specify: ____
____
____)

3. Can you tell us on what dates the flooding occurred and how high the flood waters reached (attach additional pages if you have information on more than 2 floods)?

Date of flood(s)		
Flood depth/ height, flow direction & location		
Are you confident of the height / depth of the flood? Yes ☐ No ☐	☐ High (within 5cm) ☐ Medium (within 20cm) ☐ Low (within 50cm)	☐ High (within 5cm) ☐ Medium (within 20cm) ☐ Low (within 50cm)
What time did you observe the flood height / depth?		

4. Do you have any photographs or videos of these (or other) floods?

☐ Yes ☐ No

If you answered Yes, can you provide a copy of these to assist with the computer model calibration (they will be returned as soon as we make a copy).

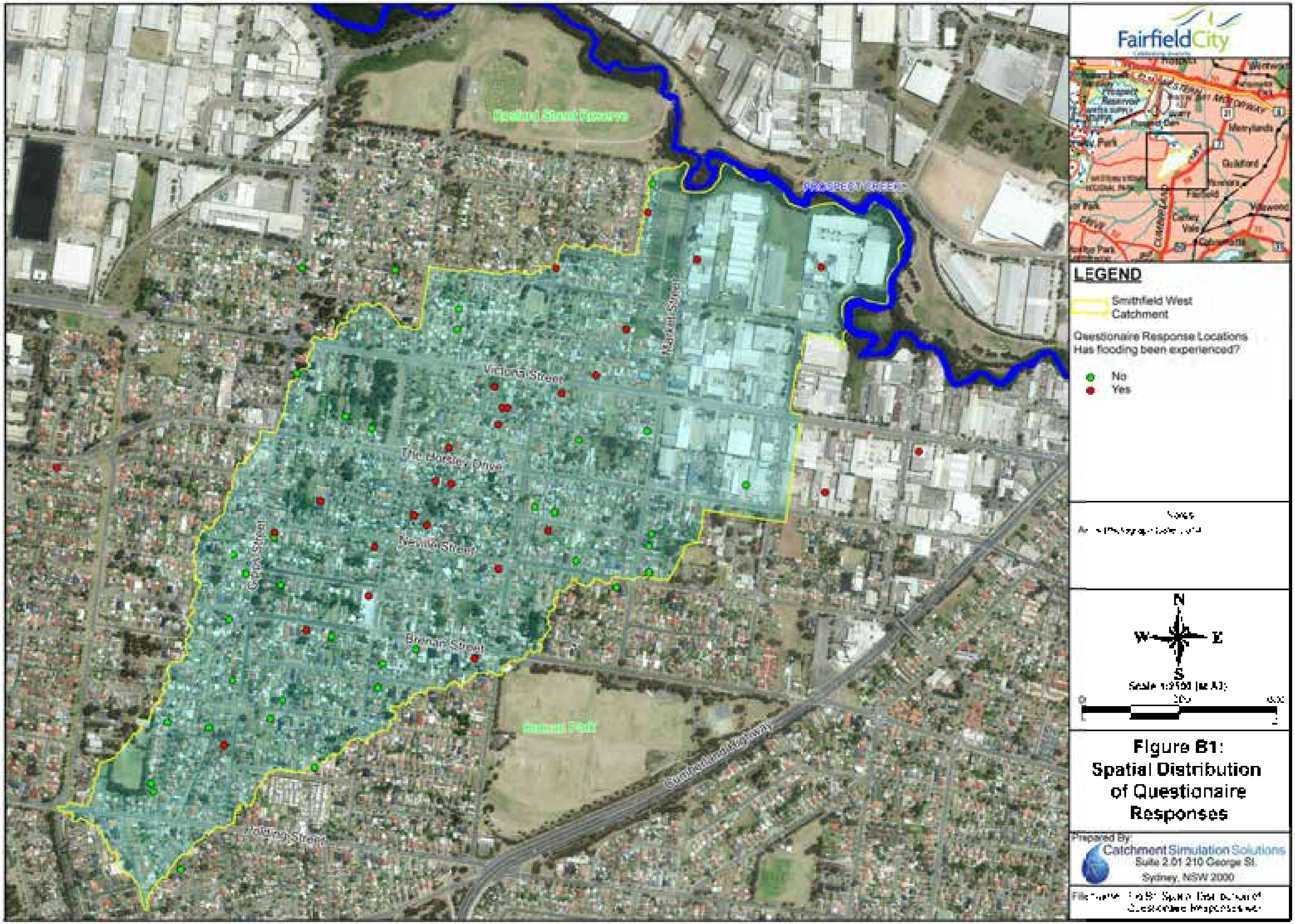


Figure B1: Spatial Distribution of Questionaire Responses

Community Questionnaire Responses - Smithfield West Overland Flood Study

#	Property Type	Occupier Status	How long have your lived in area? Current Address	In Smithfield Area	Have you been affected by flooding in the past? Traffic Disrupted	Yard Flooded	House / Business Flooded	Other	Description	Historic Flood Information Date of Floods	Flood Depth / Height & Location	Confidence Level	Time	Photo/Video	Additional Flood Information How fast do floodwaters move?	Main Cause of Flooding?	Do You Keep Rainfall Records?	Additional Comments
1	Residential	Owner Occupied	5 Years	5 Years	no	no	no	no						no				
2	Residential	Owner Occupied	49 Years	49 Years	no	no	no	no						no				
3	Residential	Owner Occupied	7 Years	7 Years	yes	yes	yes	yes	car flooded inside garage door	2012	over 1 metre flowing from Cartela Road into 10 Canara Place	High		yes	walking pace/running pace	Storm water capacity, the entire Canara place has only one drain/pit	no	Some of the stormwater drainage pipe were not connected to the front of the kerb
										early flood inbetween 1990 - 2012	over 1 metre flowing from Cartela Road into 10 Canara Place	High		yes				
4	Residential	Owner Occupied	39 Years	39 Years	yes	yes	yes	no		1990	500-800mm going Easterly	High		no		Stormwater Capacity reached, fences acting as obstruction	no	Residents forced to create barriers and damns to prevent property damage
										2012	500-800mm going Easterly	High		no				
5	Residential	Owner Occupied	14 Years	14 Years	no	yes	no	no		1990	knee deep on Victoria road going to Cumberland	High	day time	no	Stationary/walking pace	Flow obstruction and pipe capacity reached on Cumberland St	no	Maybe Widen canal on Cumberland highway, the flood arround 1986 was the worst one recorded by locals
6	Residential	Owner Occupied	28 Years	28 Years	yes	yes	yes	no						yes	Running pace		no	
7	Residential	Owner Occupied	50 Years	50 Years	no	yes	yes	no		1998 or 1999				yes		Stormwater system blockage at that time	no	There has been a couple of small floodings of roads mainly Canara Place, no. 24 and 26 were raised, drains were also not kept clean.
8	Residential	Owner Occupied	25 Years	30 Years	yes	yes	yes	yes	Back room flooded	2012	Overland Flow from Horsely Drive (going north)		Mid Afternoon	yes	walking pace	Stormwater pipes at capacity or blocked	no	Overland flows heavy obstucted by buildings on The Horsely Drive due to raised yards and driveways.
										2014	Overland Flow from Horsely Drive (going north)		Mid Afternoon	yes				
9	Residential	Owner Occupied	28 Years	50 Years	yes	yes	yes	yes	Water Damage inside property and garage		0.2m - 1.4m	High	During the day		running pace	Fenses acting as obstructions, stormwater pipes too small	yes	
10	Residential	Owner Occupied	8 Years	8 Years	no	no	no	no									no	no
11	Residential	Owner Occupied	34 Years	34 Years	no	yes	yes	no		1986	3ft in the garage	High		no	walking pace/Running pace	Stormwater Capacity reached	no	
12	Residential	Owner Occupied	7 Years	7 Years	no	yes	no	no						no	walking pace	Storm water system blockage	no	not a lot of flooding on property
13	Residential	Owner Occupied	44 Years	44 Years	no	no	no	no									no	Drains and grates potentially blocked from leaves
14	Residential	Owner Occupied	37 Years	37 Years	no	no	yes	yes	Garage flooded	2000	170mm in garage	High	Afternoon	no			no	
15	Residential	Owner Occupied	25 Years	65 Years	no	no	no	no		1950-1951				yes				Homes on Neville Street and Horsely Drive were built on a former creek
16	Residential	Owner Occupied	59 Years	59 Years	no	no	no	no									no	no
17	Residential	Owner Occupied	2 Years	10 Years	no	no	no	no						no			no	
18	Residential	Owner Occupied	54 Years	54 Years	no	no	no	no									no	
19	Industrial	Business	34 Years	42 Years	yes	no	no	no	Car park was flooded		0.5m		12pm	no	walking pace	Storm water trapped in areas	no	
20	Residential	Owner Occupied	54 Years	54 Years	no	yes	no	no		around 1989	0.3m in back yard	High	afternoon	no	walking pace		no	
21	Residential	Owner Occupied	10 Years	10 Years	no	yes	no	no						no	Stationary		no	
22	Residential	Owner Occupied	62 Years	62 Years	no	no	no	no									no	
23			Years	Years	no	no	no	no									no	
24	Residential	Owner Occupied	25 Years	25 Years	no	yes	yes	no	water almost entering home	1991/1992	flowing from right fence towards road	Low	midday	yes	walking pace	Fence acting as obstruction	no	water flowing through vacant lot across Brenan street
										2014/2013	flowing from right fence towards road	Low	midday					
25	Residential	Owner Occupied	47 Years	47 Years	yes	yes	yes			1987	1 metre	High	midday					
										1990	1 metre	Low	midday	yes	running pace	not enough drains	no	Flooding begins in Curry Place and comes throw Horsely Drive

#	Property Type	Occupier Status	How long have your lived in area? Current Address	In Smithfield Area	Have you been affected by flooding in the past? Traffic Disrupted	Yard Flooded	House / Business Flooded	Other	Description	Historic Flood Information Date of Floods	Flood Depth / Height & Location	Confidence Level	Time	Photo/Video	Additional Flood Information How fast do floodwaters move?	Main Cause of Flooding?	Do You Keep Rainfall Records?	Additional Comments
26	Residential	Owner Occupied	10 Years	10 Years	no	no	no	no									no	
27	Residential	Owner Occupied	15 Years	15 Years	no	no	no	no							Walking pace	Stormwater capacity reached and obstruction to overland flows	no	
28	Residential	Owner Occupied	57 Years	57 Years	no	no	no	no									no	
29	Residential	Rental Property	11 Years	44 Years	no	no	no	no									no	Low ground areas like Victoria St Brown St and Dubin St were primarily affect during heavy storms in 1967 with estimated flooding up to 1.3-1.5m
30	Residential	Owner Occupied	8 Years	8 Years	no	no	no	no									no	
31	Industrial	Business	41 Years	41 Years	yes	yes	yes	yes	Ware House and carpark flooded area of 1600m^2	1986	50cm	High	Daytime	yes	Running pace	Stormwater Capacity reached, in addition overland flow obstruction	no	Fix the crook at market St
										2012	20cm	High	Daytime	yes				
32	Residential	Owner Occupied	46 Years	65 Years	yes	no	no	no						no	Running pace		no	
33	Residential	Owner Occupied	48 Years	48 Years	yes	no	no	yes	Driveway Blocked	before 2006		Medium	8:30 AM	no	Walking pace	Stormwater drain on Vicotria Street near Hart Street overflowing	no	keep drains clean
34	Residential	Owner Occupied	0.5 Years	0.5 Years	no	no	no	no						no		Fence acting as obstruction	no	spent 28 years at address and never flooded
35	Residential	Owner Occupied	17 Years	17 Years	no	no	no	no									no	keep drains clean
36	Residential	Owner Occupied	39 Years	49 Years	no	no	no	no									no	Only flooding observed was near Cumberlands Highway
37	Commercial	Owner Occupied	20 Years	20 Years	yes	yes	no	no	Flooding came across roadway	2012	1	Medium		no	Walking pace	Creek overgrown	no	
38	Commercial	Owner Occupied	14 Years	14 Years	yes	no	no	no	Jane St and Brenan St closed due to flood		road submerged		daytime	no	stationary	Stormwater capacity reached	no	
39	Residential	Rental Property	2 Years	0.5 Years	no	no	no	no									no	
40	Residential	Owner Occupied	20 Years	20 Years	no	no	no	no									no	
41	Residential	Owner Occupied	50 Years	50 Years	no	no	no	no										
42	Residential	Owner Occupied	36 Years	38 Years	no	no	no	no										No flooding observed on Rowley St
43	Residential	Owner Occupied	53 Years	55 Years	no	no	no	no						no	Running pace	Stormwater drain blockage		
44	Residential	Owner Occupied	52 Years	Years	no	no	no	no						no			no	
45	Residential	Owner Occupied	35 Years	Years	no	no	no	no									no	
46	Residential	Owner Occupied	6 Years	6 Years	no	no	no	no										
47	Residential	Owner Occupied	29 Years	29 Years	no	no	no	no						no			no	
48	Residential	Owner Occupied	54 Years	54 Years	no	no	no	no							running pace		no	
49	Residential	Owner Occupied	47 Years	47 Years	no	no	no	no						no			no	
50	Residential	Owner Occupied	28 Years	28 Years	yes	no	no	no				Low		no	running pace	Stormwater system blockage and at capacity	no	
51	Industrial	Rental Property	30 Years	30 Years	no	no	no	no									no	
52	Residential	Owner Occupied	36 Years	36 Years	no	no	no	no									no	
53	Residential	Owner Occupied	54 Years	54 Years	no	no	no	no									no	
54	Residential	Owner Occupied	16 Years	16 Years	no	no	no	no									no	
55	Residential	Owner Occupied	53 Years	53 Years	no	no	no	no									no	
56	Industrial	Rental Property	30 Years	30 Years	no	no	no	no								stormwater capacity	no	
57	Residential	Owner Occupied	49 Years	49 Years	no	no	no	no							running pace		no	
58	Residential	Owner Occupied	23 Years	35 Years	no	no	no	no							walking pace	stormwater capacity and obstructions	no	
59	Residential	Rental Property	0 Years	0 Years	no	no	no	no									no	
60	Commercial	Rental Property	10 Years	30 Years	no	no	yes	no		some time in late 80s to 90s		Low		no	running pace		no	
61	Residential	Rental Property	0 Years	0 Years	no	no	yes	no									no	
62	Residential	Owner Occupied	3 Years	3 Years	no	no	yes	no						no			no	
63	Residential	Owner Occupied	20 Years	20 Years	no	no	yes	no							walking pace		no	High Floods occuring on Horsely Drive affecting traffic

#	Property Type	Occupier Status	How long have your lived in area?		Have you been affected by flooding in the past?					Historic Flood Information					Additional Flood Information			
			Current Address	In Smithfield Area	Traffic Disrupted	Yard Flooded	House / Business Flooded	Other	Description	Date of Floods	Flood Depth / Height & Location	Confidence Level	Time	Photo/Video	How fast do floodwaters move?	Main Cause of Flooding?	Do You Keep Rainfall Records?	Additional Comments
64	Residential	Rental Property	1 Years	20 Years	no	no	yes	no									no	
65	Industrial	Owner Occupied	30 Years	54 Years	no	yes	yes	yes	Car park flooded	everytime it rains	100mm	High			walking pace	Stormwater system blockage and at capacity	no	More stormwater connections would be useful
66	Residential	Owner Occupied	16 Years	16 Years	no	no	no	no							running pace		no	
67	Residential	Owner Occupied	47.75 Years	47.75 Years	no	yes	yes	no				Low	from 6:30pm to morning	no	Stationary/walking pace	Blockage of gutters from rubbish	no	Council stopped cleaning gutters regularly may be the cause behind floods
68	Residential	Rental Property	35 Years	35 Years	no	no	no	no							running pace	Stormwater blockage and capacity reached	no	

APPENDIX C

MANNING'S 'N' CALCULATIONS

Manning's 'n' Calculations

Prepared by: D. Tetley **Date:** 22/08/2014
Checked by: C. Ryan **Date:** 12/05/2015

The following provide Manning's' n roughness coefficient calculations based on the modified Cowan method documented in the USGS Paper 2339: "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains' (Arcement & Schneider). The approach is appropriate for direct rainfall modelling as it can account for the variation in 'n' with respect to flow depth.

Overview

Manning's 'n' is calculated using the modified Cowan method based on the following formula:

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

Where: n_b = a base value of n for the floodplain's natural bare soil surface
n_1 = a correction factor for the effect of surface irregularities
n_2 = a value for variations in shape and size of the floodplain cross-section (assumed to be 0.0)
n_3 = a value for obstructions
n_4 = a value for vegetation on the floodplain
m = a correction factor for sinuosity (assumed to be 1.0)

Description of Surface / Material Type

Material Type 5 - Grass
Relatively short grass. Occasional tree or fence post

n_b Calculation

n_b is extracted from the following table:

Table 1. Base Values of Manning's *n*

Bed Material	Median Size of bed material (in millimeters)	Base *n* Value: Straight Uniform Channel[1]	Base *n* Value: Smooth Channel[2]
Sand Channels			
Sand[3]	0.2	0.012	--
	.3	.017	--
	.4	.020	--
	.5	.022	--
	.6	.023	--
	.8	.025	--
	1.0	.026	--
Stable Channels and Flood Plains			
Concrete	--	0.012-0.018	0.011
Rock Cut	--	--	.025
Firm Soil	--	0.025-0.032	.020

Coarse Sand	1-2	0.026-0.035	--
Fine Gravel	--	--	.024
Gravel	2-64	0.028-0.035	--
Coarse Gravel	--	--	.026
Cobble	64-256	0.030-0.050	--
Boulder	>256	0.040-0.070	--

[Modified from Aldridge & Garret, 1973, Table 1 --No data
1Benson & Dalrymple --No data
2 For indicated material; Chow(1959)
3 Only For Upper regime flow where grain roughness is predominant

Assume "Firm Soil" for manicured grass areas

n_b = 0.025

n_1 Calculation (Degree of Irregularity)

n_1 is extracted from the following table:

Smooth	0.000	Compares to the smoothest, flattest flood-plain attainable in a given bed material.
Minor	0.001-0.005	Is a Flood Plain Slightly irregular in shape. A few rises and dips or sloughs may be more visible on the flood plain.
Moderate	0.006-0.010	Has more rises and dips. Sloughs and hummocks may occur.
Severe	0.011-0.020	Flood Plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pasture land and furrows perpendicular to the flow are also included.

Assume "moderate" to cater for undulating terrain across most of the study area

n_1 = 0.006

n_3 Calculation (Effect of Obstructions)

n_3 is extracted from the following table:

Negligible	0.000-0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	0.040-0.050	Obstructions occupy less than 15 percent of the cross-sectional area.
Appreciable	0.020-0.030	Obstructions occupy from 15 percent to 50 percent of the cross-sectional area.

Occasional tree stump or obstruction may be present:

n_3 = 0.004

n_4 Calculation (Effect of Vegetation)

n_4 is largely driven by the height of flow relative to the height of vegetation as defined in the following table:

Small	0.001-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrow-weed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1-to-2-year-old willow trees in the dormant season.
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8-to-10-years-old willow or cottonwood trees intergrow with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 0.607 m ;or mature row crops such as small vegetables, or mature field crops where depth flow is at least twice the height of the vegetation.
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is

		below branches, or mature field crops where depth of flow is less than the height of the vegetation
Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar(all vegetation in full foliage), or heavy stand of timber, few down trees, depth of reaching branches

Assume grass is equal to or less than 0.05 metres in height

n_4 = 0.065	When water depth is < 0.03m	(water depth less than height of grass)
n_4 = 0.03	When water depth is ~ 0.05m	(water depth equal in height to grass)
n_4 = 0.015	When water depth is ~ 0.07m	(water depth less than twice height of grass)
n_4 = 0.001	When water depth is > 0.1m	(water depth more than twice height of grass)

Final 'n' Value

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

n = 0.11	When water depth is < 0.03m
n = 0.075	When water depth is ~ 0.05m
n = 0.055	When water depth is ~ 0.07m
n = 0.03	When water depth is > 0.1m

Manning's 'n' Calculations

Prepared by: D. Tetley **Date:** 22/08/2014
Checked by: C. Ryan **Date:** 12/05/2015

The following provide Manning's' n roughness coefficient calculations based on the modified Cowan method documented in the USGS Paper 2339: "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains' (Arcement & Schneider). The approach is appropriate for direct rainfall modelling as it can account for the variation in 'n' with respect to flow depth.

Overview

Manning's 'n' is calculated using the modified Cowan method based on the following formula:

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

Where: n_b = a base value of n for the floodplain's natural bare soil surface
n_1 = a correction factor for the effect of surface irregularities
n_2 = a value for variations in shape and size of the floodplain cross-section (assumed to be 0.0)
n_3 = a value for obstructions
n_4 = a value for vegetation on the floodplain
m = a correction factor for sinuosity (assumed to be 1.0)

Description of Surface / Material Type

Material Type 3 - Trees
Trees (> 2metres in height) with medium to dense undergrowth

n_b Calculation

n_b is extracted from the following table:

Table 1. Base Values of Manning's *n*

Bed Material	Median Size of bed material (in millimeters)	Base *n* Value	
		Straight Uniform Channel[1]	Smooth Channel[2]
Sand Channels			
Sand[3]	0.2	0.012	--
	.3	.017	--
	.4	.020	--
	.5	.022	--
	.6	.023	--
	.8	.025	--
	1.0	.026	--
Stable Channels and Flood Plains			
Concrete	--	0.012-0.018	0.011
Rock Cut	--	--	.025
Firm Soil	--	0.025-0.032	.020

Coarse Sand	1-2	0.026-0.035	--
Fine Gravel	--	--	.024
Gravel	2-64	0.028-0.035	--
Coarse Gravel	--	--	.026
Cobble	64-256	0.030-0.050	--
Boulder	>256	0.040-0.070	--

[Modified from Aldridge & Garret, 1973, Table 1 --No data
1Benson & Dalrymple --No data
2 For indicated material; Chow(1959)
3 Only For Upper regime flow where grain roughness is predominant

Assume "Firm Soil"

n_b = 0.025

n_1 Calculation (Degree of Irregularity)

n_1 is extracted from the following table:

Smooth	0.000	Compares to the smoothest, flattest flood-plain attainable in a given bed material.
Minor	0.001-0.005	Is a Flood Plain Slightly irregular in shape. A few rises and dips or sloughs may be more visible on the flood plain.
Moderate	0.006-0.010	Has more rises and dips. Sloughs and hummocks may occur.
Severe	0.011-0.020	Flood Plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pasture land and furrows perpendicular to the flow are also included.

Assume "moderate" to cater for undulating terrain across most of the study area

n_1 = 0.01

n_3 Calculation (Effect of Obstructions)

n_3 is extracted from the following table:

Negligible	0.000-0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	0.040-0.050	Obstructions occupy less than 15 percent of the cross-sectional area.
Appreciable	0.020-0.030	Obstructions occupy from 15 percent to 50 percent of the cross-sectional area.

Many obstructions likely

n_3 = 0.025

n_4 Calculation (Effect of Vegetation)

n_4 is largely driven by the height of flow relative to the height of vegetation as defined in the following table:

Small	0.001-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrow-weed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1-to-2-year-old willow trees in the dormant season.
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8-to-10-years-old willow or cottonwood trees intergrow with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 0.607 m ;or mature row crops such as small vegetables, or mature field crops where depth flow is at least twice the height of the vegetation.
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is

		below branches, or mature field crops where depth of flow is less than the height of the vegetation
Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar(all vegetation in full foliage), or heavy stand of timber, few down trees, depth of reaching branches

Assume significant undergrowth up to 0.3 m in height, less dense shrubs up to 1.5m & tree branch above 2m

n_4 = 0.1	When water depth is < 0.3m	(Shrubs, trees & undergrowth in contact with flow)
n_4 = 0.05	When water depth is ~ 1.5m	(Shrubs & tree trunks in contact with flow)
n_4 = 0.02	When water depth is >2m	(Tree trunks in contact with flow)

Final 'n' Value

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

n = 0.16	When water depth is < 0.3m
n = 0.11	When water depth is ~ 1.5m
n = 0.08	When water depth is >2.0m

Manning's 'n' Calculations

Prepared by: D. Tetley **Date:** 22/08/2014
Checked by: C. Ryan **Date:** 12/05/2015

The following provide Manning's' n roughness coefficient calculations based on the modified Cowan method documented in the USGS Paper 2339: "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains' (Arcement & Schneider). The approach is appropriate for direct rainfall modelling as it can account for the variation in 'n' with respect to flow depth.

Overview

Manning's 'n' is calculated using the modified Cowan method based on the following formula:

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

Where: n_b = a base value of n for the floodplain's natural bare soil surface
n_1 = a correction factor for the effect of surface irregularities
n_2 = a value for variations in shape and size of the floodplain cross-section (assumed to be 0.0)
n_3 = a value for obstructions
n_4 = a value for vegetation on the floodplain
m = a correction factor for sinuosity (assumed to be 1.0)

Description of Surface / Material Type

Material Type 7 - Shrubs
Shrubs up to 1.5m high with some undergrowth

n_b Calculation

n_b is extracted from the following table:

Table 1. Base Values of Manning's *n*

Bed Material	Median Size of bed material (in millimeters)	Base *n* Value: Straight Uniform Channel[1]	Base *n* Value: Smooth Channel[2]
Sand Channels			
Sand[3]	0.2	0.012	--
	.3	.017	--
	.4	.020	--
	.5	.022	--
	.6	.023	--
	.8	.025	--
	1.0	.026	--
Stable Channels and Flood Plains			
Concrete	--	0.012-0.018	0.011
Rock Cut	--	--	.025
Firm Soil	--	0.025-0.032	.020

Coarse Sand	1-2	0.026-0.035	--
Fine Gravel	--	--	.024
Gravel	2-64	0.028-0.035	--
Coarse Gravel	--	--	.026
Cobble	64-256	0.030-0.050	--
Boulder	>256	0.040-0.070	--

[Modified from Aldridge & Garret, 1973, Table 1 --No data
1Benson & Dalrymple --No data
2 For indicated material; Chow(1959)
3 Only For Upper regime flow where grain roughness is predominant

Assume "Firm Soil"

n_b = 0.025

n_1 Calculation (Degree of Irregularity)

n_1 is extracted from the following table:

Smooth	0.000	Compares to the smoothest, flattest flood-plain attainable in a given bed material.
Minor	0.001-0.005	Is a Flood Plain Slightly irregular in shape. A few rises and dips or sloughs may be more visible on the flood plain.
Moderate	0.006-0.010	Has more rises and dips. Sloughs and hummocks may occur.
Severe	0.011-0.020	Flood Plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pasture land and furrows perpendicular to the flow are also included.

Assume "moderate" to cater for undulating terrain across most of the study area

n_1 = 0.008

n_3 Calculation (Effect of Obstructions)

n_3 is extracted from the following table:

Negligible	0.000-0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	0.040-0.050	Obstructions occupy less than 15 percent of the cross-sectional area.
Appreciable	0.020-0.030	Obstructions occupy from 15 percent to 50 percent of the cross-sectional area.

Minmal obstructions (soil mounds/deposits)

n_3 = 0.004

n_4 Calculation (Effect of Vegetation)

n_4 is largely driven by the height of flow relative to the height of vegetation as defined in the following table:

Small	0.001-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrow-weed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1-to-2-year-old willow trees in the dormant season.
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8-to-10-years-old willow or cottonwood trees intergrow with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 0.607 m ;or mature row crops such as small vegetables, or mature field crops where depth flow is at least twice the height of the vegetation.
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is

		below branches, or mature field crops where depth of flow is less than the height of the vegetation
Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar(all vegetation in full foliage), or heavy stand of timber, few down trees, depth of reaching branches

Assume significant undergrowth up to 0.3 m in height & shrubs up to 1.5m

n_4 = 0.1	When water depth is < 0.3m	(Shrubs & undergrowth in contact with flow)
n_4 = 0.04	When water depth is ~ 1m	(Shrubs in contact with flow)
n_4 = 0.01	When water depth is >1.5m	(Flow above shrubs)

Final 'n' Value

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

n = 0.137	When water depth is < 0.3m
n = 0.077	When water depth is ~ 1.0m
n = 0.047	When water depth is >1.5m

Manning's 'n' Calculations

Prepared by: D. Tetley **Date:** 22/08/2014
Checked by: C. Ryan **Date:** 12/05/2015

The following provide Manning's' n roughness coefficient calculations based on the modified Cowan method documented in the USGS Paper 2339: "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains' (Arcement & Schneider). The approach is appropriate for direct rainfall modelling as it can account for the variation in 'n' with respect to flow depth.

Overview

Manning's 'n' is calculated using the modified Cowan method based on the following formula:

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

Where: n_b = a base value of n for the floodplain's natural bare soil surface
n_1 = a correction factor for the effect of surface irregularities
n_2 = a value for variations in shape and size of the floodplain cross-section (assumed to be 0.0)
n_3 = a value for obstructions
n_4 = a value for vegetation on the floodplain
m = a correction factor for sinuosity (assumed to be 1.0)

Description of Surface / Material Type

Material Type 2 - Roads
Concrete kerb & gutter for containing low flows with road pavement at higher stages

n_b Calculation

n_b is extracted from the following table:

Table 1. Base Values of Manning's *n*

Bed Material	Median Size of bed material (in millimeters)	Base *n* Value: Straight Uniform Channel[1]	Base *n* Value: Smooth Channel[2]
Sand Channels			
Sand[3]	0.2	0.012	--
	.3	.017	--
	.4	.020	--
	.5	.022	--
	.6	.023	--
	.8	.025	--
	1.0	.026	--
Stable Channels and Flood Plains			
Concrete	--	0.012-0.018	0.011
Rock Cut	--	--	.025
Firm Soil	--	0.025-0.032	.020

Coarse Sand	1-2	0.026-0.035	--
Fine Gravel	--	--	.024
Gravel	2-64	0.028-0.035	--
Coarse Gravel	--	--	.026
Cobble	64-256	0.030-0.050	--
Boulder	>256	0.040-0.070	--

[Modified from Aldridge & Garret, 1973, Table 1 --No data
1Benson & Dalrymple --No data
[2] For indicated material; Chow(1959)
[3] Only For Upper regime flow where grain roughness is predominant

Assume "Concrete"

n_b = 0.012

n_1 Calculation (Degree of Irregularity)

n_1 is extracted from the following table:

Smooth	0.000	Compares to the smoothest, flattest flood-plain attainable in a given bed material.
Minor	0.001-0.005	Is a Flood Plain Slightly irregular in shape. A few rises and dips or sloughs may be more visible on the flood plain.
Moderate	0.006-0.010	Has more rises and dips. Sloughs and hummocks may occur.
Severe	0.011-0.020	Flood Plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pasture land and furrows perpendicular to the flow are also included.

Relatively minor grades along most roadways

n_1 = 0.002

n_3 Calculation (Effect of Obstructions)

n_3 is extracted from the following table:

Negligible	0.000-0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	0.040-0.050	Obstructions occupy less than 15 percent of the cross-sectional area.
Appreciable	0.020-0.030	Obstructions occupy from 15 percent to 50 percent of the cross-sectional area.

May be garbage bins etc, but assume negligible

n_3 = 0.002

n_4 Calculation (Effect of Vegetation)

n_4 is largely driven by the height of flow relative to the height of vegetation as defined in the following table:

Small	0.001-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrow-weed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1-to-2-year-old willow trees in the dormant season.
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8-to-10-years-old willow or cottonwood trees intergrow with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 0.607 m ,or mature row crops such as small vegetables, or mature field crops where depth flow is at least twice the height of the vegetation.
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is

		below branches, or mature field crops where depth of flow is less than the height of the vegetation
Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar(all vegetation in full foliage), or heavy stand of timber, few down trees, depth of reaching branches

Assume water contained in gutter initially and then spreads onto road pavement

n_4 = 0.001	When water depth is < 0.04m	(Water contained within gutter)
n_4 = 0.005	When water depth is ~ 0.1m	(Water comes into contact with pavement aggregate)
n_4 = 0.002	When water depth is > 0.15m	(Water well above aggregate/gutter height)

Final 'n' Value

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

n = 0.017	When water depth is < 0.04m
n = 0.021	When water depth is ~ 0.1m
n = 0.02	When water depth is >0.15m

Manning's 'n' Calculations

Prepared by: D. Tetley **Date:** 22/08/2014
Checked by: C. Ryan **Date:** 12/05/2015

The following provide Manning's' n roughness coefficient calculations based on the modified Cowan method documented in the USGS Paper 2339: "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains' (Arcement & Schneider). The approach is appropriate for direct rainfall modelling as it can account for the variation in 'n' with respect to flow depth.

Overview

Manning's 'n' is calculated using the modified Cowan method based on the following formula:

$n = m\,(n_b + n_1 + n_2 + n_3 + n_4)$

Where: n_b = a base value of n for the floodplain's natural bare soil surface
n_1 = a correction factor for the effect of surface irregularities
n_2 = a value for variations in shape and size of the floodplain cross-section (assumed to be 0.0)
n_3 = a value for obstructions
n_4 = a value for vegetation on the floodplain
m = a correction factor for sinuosity (assumed to be 1.0)

Description of Surface / Material Type

Material Type 7 - Concrete

n_b Calculation

n_b is extracted from the following table:

Table 1. Base Values of Manning's *n*

Bed Material	Median Size of bed material (in millimeters)	Base n Value: Straight Uniform Channel[1]	Base n Value: Smooth Channel[2]
Sand Channels			
Sand[3]	0.2	0.012	--
	.3	.017	--
	.4	.020	--
	.5	.022	--
	.6	.023	--
	.8	.025	--
	1.0	.026	--
Stable Channels and Flood Plains			
Concrete	--	0.012-0.018	0.011
Rock Cut	--	--	.025
Firm Soil	--	0.025-0.032	.020

Coarse Sand	1-2	0.026-0.035	--
Fine Gravel	--	--	.024
Gravel	2-64	0.028-0.035	--
Coarse Gravel	--	--	.026
Cobble	64-256	0.030-0.050	--
Boulder	>256	0.040-0.070	--

[Modified from Aldridge & Garret, 1973, Table 1 --No data
1Benson & Dalrymple --No data
2 For indicated material; Chow(1959)
3 Only For Upper regime flow where grain roughness is predominant

Assume "Concrete"

n_b = 0.012

n_1 Calculation (Degree of Irregularity)

n_1 is extracted from the following table:

Smooth	0.000	Compares to the smoothest, flattest flood-plain attainable in a given bed material.
Minor	0.001-0.005	Is a Flood Plain Slightly irregular in shape. A few rises and dips or sloughs may be more visible on the flood plain.
Moderate	0.006-0.010	Has more rises and dips. Sloughs and hummocks may occur.
Severe	0.011-0.020	Flood Plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pasture land and furrows perpendicular to the flow are also included.

Assume smooth

n_1 = 0

n_3 Calculation (Effect of Obstructions)

n_3 is extracted from the following table:

Negligible	0.000-0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	0.040-0.050	Obstructions occupy less than 15 percent of the cross-sectional area.
Appreciable	0.020-0.030	Obstructions occupy from 15 percent to 50 percent of the cross-sectional area.

Assume minimal obstructions

n_3 = 0.002

n_4 Calculation (Effect of Vegetation)

n_4 is largely driven by the height of flow relative to the height of vegetation as defined in the following table:

Small	0.001-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrow-weed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1-to-2-year-old willow trees in the dormant season.
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8-to-10-years-old willow or cottonwood trees intergrow with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 0.607 m ;or mature row crops such as small vegetables, or mature field crops where depth flow is at least twice the height of the vegetation.
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is

		below branches, or mature field crops where depth of flow is less than the height of the vegetation
Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar(all vegetation in full foliage), or heavy stand of timber, few down trees, depth of reaching branches.

n_4 = 0.02	When water depth is < 0.005m	(Water in contact with aggregate)
n_4 = 0.001	When water depth is > 0.005m	(Water above aggregate height)

Final 'n' Value

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

n = 0.034	When water depth is < 0.005m
n = 0.015	When water depth is > 0.005m

Manning's 'n' Calculations

Prepared by: D. Tetley **Date:** 22/08/2014
Checked by: C. Ryan **Date:** 12/05/2015

The following provide Manning's' n roughness coefficient calculations based on the modified Cowan method documented in the USGS Paper 2339: "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains' (Arcement & Schneider). The approach is appropriate for direct rainfall modelling as it can account for the variation in 'n' with respect to flow depth.

Overview

Manning's 'n' is calculated using the modified Cowan method based on the following formula:

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

Where: n_b = a base value of n for the floodplain's natural bare soil surface
n_1 = a correction factor for the effect of surface irregularities
n_2 = a value for variations in shape and size of the floodplain cross-section (assumed to be 0.0)
n_3 = a value for obstructions
n_4 = a value for vegetation on the floodplain
m = a correction factor for sinuosity (assumed to be 1.0)

Description of Surface / Material Type

Material Type 8 - Crops
Market Gardens area with small plants (<0.5m high) and soil

n_b Calculation

n_b is extracted from the following table:

Table 1. Base Values of Manning's *n*

Bed Material	Median Size of bed material (in millimeters)	Base *n* Value: Straight Uniform Channel[1]	Base *n* Value: Smooth Channel[2]
Sand Channels			
Sand[3]	0.2	0.012	--
	.3	.017	--
	.4	.020	--
	.5	.022	--
	.6	.023	--
	.8	.025	--
	1.0	.026	--
Stable Channels and Flood Plains			
Concrete	--	0.012-0.018	0.011
Rock Cut	--	--	.025
Firm Soil	--	0.025-0.032	.020

Coarse Sand	1-2	0.026-0.035	--
Fine Gravel	--	--	.024
Gravel	2-64	0.028-0.035	--
Coarse Gravel	--	--	.026
Cobble	64-256	0.030-0.050	--
Boulder	>256	0.040-0.070	--

[Modified from Aldridge & Garret, 1973, Table 1 --No data
1Benson & Dalrymple --No data
2 For indicated material; Chow(1959)
3 Only For Upper regime flow where grain roughness is predominant

Assume "Firm Soil"

n_b = 0.03

n_1 Calculation (Degree of Irregularity)

n_1 is extracted from the following table:

Smooth	0.000	Compares to the smoothest, flattest flood-plain attainable in a given bed material
Minor	0.001-0.005	Is a Flood Plain Slightly irregular in shape. A few rises and dips or sloughs may be more visible on the flood plain.
Moderate	0.006-0.010	Has more rises and dips. Sloughs and hummocks may occur.
Severe	0.011-0.020	Flood Plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pasture land and furrows perpendicular to the flow are also included.

Has rises and dips reflecting rows of crops with built up soil mounds

n_1 = 0.008

n_3 Calculation (Effect of Obstructions)

n_3 is extracted from the following table:

Negligible	0.000-0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	0.040-0.050	Obstructions occupy less than 15 percent of the cross-sectional area.
Appreciable	0.020-0.030	Obstructions occupy from 15 percent to 50 percent of the cross-sectional area.

Assume appreciable obstructions

n_3 = 0.02

n_4 Calculation (Effect of Vegetation)

n_4 is largely driven by the height of flow relative to the height of vegetation as defined in the following table:

Small	0.001-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrow-weed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1-to-2-year-old willow trees in the dormant season.
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8-to-10-years-old willow or cottonwood trees intergrow with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 0.607 m ;or mature row crops such as small vegetables, or mature field crops where depth flow is at least twice the height of the vegetation.
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is

		below branches, or mature field crops where depth of flow is less than the height of the vegetation
Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar(all vegetation in full foliage), or heavy stand of timber, few down trees, depth of reaching branches

n_4 = 0.075	When water depth is < 0.1m	(Water in contact with soil)
n_4 = 0.035	When water depth is ~ 0.5m	(Water in contact with crops)
n_4 = 0.001	When water depth is >0.5 m	(Water above crops)

Final 'n' Value

$n = m (n_b + n_1 + n_2 + n_3 + n_4)$

n = 0.133	When water depth is < 0.1m
n = 0.093	When water depth is ~ 0.5m
n = 0.059	When water depth is > 0.5m

Appendix D

Pit and Pipe Database

Table D1: Stormwater Pit Data

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
aR1/20400/10		Y	Sag	19.64	19.10	Google Street View
aR1/700/70	1.8	Y	Grade	26.81	25.26	Survey 2010
aR2/10/320	1.8	Y	Sag	47.19	46.71	Google Street View
aR2/110/10	Junction Pit			28.42	26.58	Survey 2010
aR2/120/10	Junction Pit			28.53	26.68	Survey 2010
aR2/20700/10	1.8	Y	Sag	38.86	38.50	Google Street View
aR2/21800/80	Junction Pit			31.42	29.79	Survey 2010
aR2/230/10	2.9	Y	Sag	30.53	29.46	Survey 2010
aR2/240/10	Junction Pit			31.32	29.67	Survey 2010
aR2/250/10	Junction Pit			31.39	29.74	Survey 2010
aR2/340/50	Junction Pit			37.52	36.00	Survey 2010
aR2/70/10	Junction Pit			25.07	22.70	Survey 2010
aR2/820/10	Junction Pit			22.63	20.02	Survey 2010
bR2/21800/10	2	Y	Sag	30.60	29.30	Survey 2014
R1/19700/10	2	Y	Grade	25.01	24.63	Survey 2010
R1/20100/10	2	Y	Grade	24.05	22.10	Survey 2010
R1/20200/10	2	Y	Grade	20.89	19.81	Survey 2010
R1/20300/10	2	Y	Grade	20.47	19.70	Survey 2010
R1/20400/10	1.8	Y	Grade	19.62	18.25	Survey 2010
R1/700/07		Y	Sag	19.37	16.70	Survey 2010
R1/700/09	Junction Pit			20.25	19.10	Survey 2010
R1/700/09a	1.8	Y	Grade	20.35	19.33	Google Street View
R1/700/100	1.8	Y	Grade	31.90	30.88	Google Street View
R1/700/110		Y	Grade	34.66	33.73	Google Street View
R1/700/120	1.8	Y	Grade	35.00	34.00	Google Street View
R1/700/130	1.8	Y	Sag	35.93	34.94	Google Street View
R1/700/140	1.8	Y	Grade	36.16	35.21	Google Street View
R1/700/15	2	Y	Grade	21.29	20.07	Survey 2010
R1/700/150	1.8	Y	Grade	40.45	39.45	Google Street View
R1/700/160	1.8	Y	Grade	41.51	40.57	Google Street View
R1/700/170	0.9	Y	Grade	41.85	40.85	Google Street View
R1/700/180	1.8	Y	Grade	42.08	41.08	Google Street View
R1/700/190	1.8	Y	Grade	42.55	41.62	Google Street View

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
R1/700/20		Y	Grade	21.42	20.19	Survey 2010
R1/700/50	2	Y	Grade	24.33	22.54	Survey 2010
R1/700/65	Junction Pit			25.45	25.09	Survey 2010
R1/700/70	1.8	Y	Grade	26.53	25.21	Survey 2010
R1/700/80		Y	Sag	29.50	28.23	Survey 2010
R1/700/90	1.8	Y	Grade	29.68	28.70	Google Street View
R1/710/10	1.8	Y	Grade	20.49	19.49	Google Street View
R1/720/10	1.8	Y	Grade	20.30	19.35	Google Street View
R1/720/10a	1.2	Y	Grade	20.55	19.49	Google Street View
R1/720/20	1.8	Y	Grade	21.21	20.21	Google Street View
R1/740/10	1.8	Y	Grade	24.22	23.05	Google Street View
R1/740/20	1.8	Y	Grade	24.66	23.41	Google Street View
R1/740/20a	1.2	Y	Grade	24.79	24.29	Google Street View
R1/740/30	Junction Pit			27.33	26.40	Google Street View
R1/740/40		Y	Grade	28.90	27.71	Google Street View
R1/740/40a	Junction Pit			30.57	28.57	Google Street View
R1/740/50	1.2	Y	Sag	29.70	29.18	Google Street View
R1/740/60	1.8	Y	Sag	29.64	29.19	Google Street View
R1/750/10	1.8	Y	Grade	30.37	29.88	Google Street View
R1/750/20		Y	Grade	31.93	31.45	Google Street View
R1/760/10	1.2	Y	Grade	30.47	29.97	Google Street View
R1/770/10		Y	Grade	25.48	25.20	Google Street View
R1/780/06	Junction Pit			27.06	25.69	Survey 2010
R1/780/07	Junction Pit			30.28	29.18	Survey 2010
R1/780/09	Junction Pit			32.67	31.66	Survey 2010
R1/780/10	2	Y	Sag	33.98	32.61	Survey 2010
R1/780/20	2.6	Y	Sag	33.92	32.88	Survey 2010
R1/790/10	1.8	Y	Sag	29.58	28.47	Google Street View
R1/790/20	1.8	Y	Grade	29.53	28.62	Google Street View
R1/800/10	1.8	Y	Grade	29.75	28.74	Google Street View
R1/810/10	1.8	Y	Grade	31.95	30.95	Google Street View
R1/830/10	1.8	Y	Sag	22.27	21.27	Google Street View
R1/830/20		Y	Grade	22.52	21.52	Google Street View

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
R2/10/110		Y	Sag	31.88	30.07	Survey 2014
R2/10/140	Junction Pit			33.86	32.51	Survey 2010
R2/10/15	Junction Pit			23.83	21.55	Survey 2010
R2/10/150	1.9	Y	Sag	33.83	32.59	Survey 2010
R2/10/160	2	Y	Sag	35.32	33.48	Survey 2010
R2/10/170	1.8	Y	Sag	35.14	33.55	Survey 2010
R2/10/180	1.6	Y	Sag	37.01	35.29	Survey 2010
R2/10/190	1.9	Y	Sag	37.05	35.47	Survey 2010
R2/10/20	0.9	Y	Sag	23.47	21.67	Survey 2010
R2/10/210	Junction Pit			39.07	37.17	Survey 2010
R2/10/220		Y	Sag	39.09	37.34	Survey 2010
R2/10/230	2	Y	Grade	39.24	37.37	Survey 2010
R2/10/240	2	Y	Sag	39.46	38.97	Survey 2010
R2/10/250	1.8	Y	Sag	40.59	39.13	Survey 2010
R2/10/260	1.9	Y	Sag	41.01	39.38	Survey 2010
R2/10/270	1.8	Y	Grade	41.91	40.68	Survey 2014
R2/10/280	1.8	Y	Grade	43.03	41.88	Survey 2014
R2/10/290	0.9	Y	Grade	44.41	43.24	Survey 2014
R2/10/30	1.8	Y	Grade	23.90	21.87	Survey 2014
R2/10/300	1.8	Y	Grade	46.55	45.28	Survey 2014
R2/10/310	2.4	Y	Grade	47.34	46.20	Survey 2014
R2/10/320		Y	Sag	47.34	46.70	Survey 2014
R2/10/330	2.6	Y	Sag	50.40	49.46	Survey 2014
R2/10/40	2	Y	Sag	25.58	23.41	Survey 2010
R2/10/50	3.2	Y	Sag	25.49	23.53	Survey 2010
R2/10/55	Junction Pit			26.30	23.65	Survey 2014
R2/10/60	2	Y	Grade	27.25	25.10	Survey 2010
R2/10/60a	1.2	Y	Sag	27.38	26.50	Google Street View
R2/10/70	3.7	Y	Sag	27.20	25.24	Survey 2010
R2/10/77		Y	Sag	28.78	27.24	Survey 2010
R2/10/77a	1.8	Y	Sag	28.65	27.69	Google Street View
R2/10/78	1.9	Y	Sag	30.40	28.60	Survey 2010
R2/10/90	2.1	Y	Sag	30.78	29.58	Survey 2010

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
R2/10/98		Y	Grade	31.55	29.92	Survey 2010
R2/100/10	1.2	Y	Sag	27.02	26.02	Google Street View
R2/110/10	1.8	Y	Sag	28.15	27.20	Google Street View
R2/110/20	1.8	Y	Sag	28.22	27.24	Google Street View
R2/120/10	1.8	Y	Grade	28.63	27.50	Google Street View
R2/120/20	2.6	Y	Grade	28.54	27.54	Google Street View
R2/120/30	2.6	Y	Sag	28.58	27.55	Google Street View
R2/130/10	1.8	Y	Grade	33.49	32.56	Google Street View
R2/130/20	1.8	Y	Grade	33.94	32.94	Google Street View
R2/130/30	2.4	Y	Grade	34.09	33.12	Google Street View
R2/130/40	1.8	Y	Grade	34.14	33.13	Google Street View
R2/130/50	1.8	Y	Grade	34.03	33.25	Google Street View
R2/140/10	1.8	Y	Sag	30.94	29.57	Survey 2014
R2/140/100	1.8	Y	Grade	34.71	33.70	Survey 2014
R2/140/110	1.8	Y	Grade	36.59	35.33	Survey 2014
R2/140/120	1.8	Y	Grade	36.12	35.85	Survey 2014
R2/140/20	1.8	Y	Grade	31.68	30.15	Survey 2014
R2/140/30	1.8	Y	Grade	32.04	30.52	Survey 2014
R2/140/40	1.8	Y	Grade	32.36	30.65	Survey 2014
R2/140/50	1.8	Y	Grade	32.73	30.85	Survey 2014
R2/140/60	1.8	Y	Grade	32.75	31.57	Google Street View
R2/140/60a	1.8	Y	Sag	32.81	31.70	Survey 2014
R2/140/80		Junction Pit		34.12	32.34	Survey 2014
R2/140/90	1.8	Y	Grade	34.14	32.76	Survey 2014
R2/150/10	1.8	Y	Grade	33.17	32.17	Google Street View
R2/150/20	1.8	Y	Grade	36.97	35.92	Google Street View
R2/150/20a	1.2	Y	Grade	38.42	37.42	Google Street View
R2/150/30	1.8	Y	Grade	41.89	40.81	Google Street View
R2/150/40	1.8	Y	Grade	46.15	45.24	Google Street View
R2/150/50	1.8	Y	Sag	46.91	45.91	Google Street View
R2/150/60	1.8	Y	Sag	47.04	46.03	Google Street View
R2/160/10	1.8	Y	Grade	33.33	32.40	Google Street View
R2/170/10	1.8	Y	Grade	36.97	35.95	Google Street View

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
R2/180/10	1.8	Y	Grade	42.13	41.12	Google Street View
R2/190/10	0.9	Y	Grade	32.60	31.67	Google Street View
R2/20/10	1.8	Y	Grade	24.18	23.16	Google Street View
R2/20/20	0.9	Y	Sag	24.22	23.24	Google Street View
R2/20700/10	2	Y	Sag	38.96	38.46	Survey 2010
R2/20800/10	1.8	Y	Sag	33.94	32.55	Survey 2014
R2/20900/10	2	Y	Grade	32.90	32.30	Survey 2010
R2/210/10	1.8	Y	Grade	33.96	33.07	Survey 2014
R2/21700/10	2	Y	Grade	32.19	31.66	Survey 2010
R2/21800/10	2.1	Y	Sag	30.47	28.70	Survey 2010
R2/220/10	0.9	Y	Grade	34.70	33.66	Survey 2014
R2/220/20	0.9	Y	Grade	34.82	34.10	Survey 2014
R2/230/10	2.6	Y	Sag	30.56	30.00	Google Street View
R2/240/10	1.8	Y	Grade	31.49	30.49	Google Street View
R2/240/20	1.8	Y	Grade	34.11	33.08	Google Street View
R2/240/20a	1.8	Y	Grade	34.13	33.21	Google Street View
R2/240/30	1.8	Y	Grade	37.43	36.46	Google Street View
R2/240/40		Y	Grade	38.50	37.50	Google Street View
R2/250/10	2.6	Y	Sag	31.36	30.37	Google Street View
R2/260/10	1.8	Y	Grade	32.39	31.09	Survey 2014
R2/260/20	1.8	Y	Grade	33.77	32.78	Google Street View
R2/260/30	1.8	Y	Grade	34.29	33.28	Google Street View
R2/260/40	1.8	Y	Grade	35.38	34.38	Google Street View
R2/270/10	1.8	Y	Grade	32.34	31.40	Survey 2014
R2/280/10	1.8	Y	Sag	31.83	30.97	Survey 2014
R2/290/05	1.8	Y	Sag	31.90	30.10	Survey 2014
R2/290/10	2	Y	Grade	32.06	30.97	Survey 2014
R2/290/100	1.8	Y	Grade	42.60	41.61	Google Street View
R2/290/110		Y	Grade	42.59	41.80	Google Street View
R2/290/120	1.2	N	Grade	44.11	43.15	Google Street View
R2/290/20	2	Y	Grade	32.52	31.60	Survey 2014
R2/290/30	1.8	Y	Grade	34.88	32.95	Survey 2014
R2/290/40	1.8	Y	Grade	35.25	34.25	Google Street View

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
R2/290/50	1.8	Y	Grade	35.52	34.55	Google Street View
R2/290/60	1.8	Y	Grade	38.12	37.10	Google Street View
R2/290/60a	1.8	Y	Grade	38.62	37.67	Google Street View
R2/290/70	1.8	Y	Grade	39.15	38.19	Google Street View
R2/290/80	1.8	Y	Grade	41.99	40.99	Google Street View
R2/290/90	1.8	Y	Grade	42.45	41.48	Google Street View
R2/30/10	0.9	Y	sag	23.38	22.47	Google Street View
R2/300/10	2	Y	Grade	32.06	31.15	Survey 2010
R2/300/20	1.8	Y	Grade	33.21	32.18	Google Street View
R2/300/30	1.8	Y	Grade	36.34	35.31	Google Street View
R2/300/40	1.8	Y	Grade	39.74	38.75	Google Street View
R2/300/50	1.8	Y	Grade	41.13	40.20	Google Street View
R2/310/10	1.8	Y	Grade	33.85	32.87	Google Street View
R2/320/10	1.8	Y	Grade	36.89	35.95	Google Street View
R2/330/10	1.8	Y	Grade	41.25	40.25	Google Street View
R2/340/10	4.4	Y	Sag	32.24	31.30	Survey 2010
R2/340/20	1.8	Y	Sag	35.26	33.56	Survey 2010
R2/340/30	1.8	Y	Grade	35.35	33.65	Survey 2010
R2/340/40	2.4	N	Sag	39.88	38.86	Google Street View
R2/340/50	1.8	N	Sag	40.01	39.04	Google Street View
R2/350/10	1.9	Y	Sag	41.39	39.69	Survey 2010
R2/350/20	1.8	Y	Sag	41.55	40.52	Google Street View
R2/350/30	1.8	Y	Grade	41.47	40.54	Google Street View
R2/360/10	1.8	Y	Grade	32.53	31.85	Survey 2014
R2/370/10	1.8	Y	Grade	35.65	34.58	Google Street View
R2/380/10	1.8	Y	Grade	38.84	37.83	Google Street View
R2/390/10	1.8	Y	Grade	39.21	38.20	Google Street View
R2/40/10	1.8	Y	Grade	23.90	22.90	Google Street View
R2/400/10	1.8	Y	Grade	32.26	31.85	Google Street View
R2/410/10	1.8	Y	Grade	35.77	34.77	Google Street View
R2/410/20		Y	Grade	37.30	36.27	Google Street View
R2/410/20a	1.8	Y	Grade	36.49	35.47	Google Street View
R2/420/10	1.8	Y	Grade	36.11	35.13	Google Street View

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
R2/420/20	2	Y	Grade	39.32	37.38	Survey 2014
R2/420/30	2	Y	Grade	40.13	38.87	Survey 2014
R2/420/40	1.8	Y	Grade	39.75	38.77	Google Street View
R2/430/10	1.8	Y	Grade	36.51	35.56	Google Street View
R2/440/10	1.8	Y	Grade	39.61	38.64	Google Street View
R2/450/10	1.8	Y	Grade	39.95	38.93	Google Street View
R2/450/20	1.8	Y	Grade	40.09	39.03	Google Street View
R2/460/10	1.8	Y	Grade	39.42	38.47	Google Street View
R2/470/10	1.8	Y	Grade	39.03	38.00	Google Street View
R2/470/20	1.8	Y	Grade	39.34	38.43	Google Street View
R2/480/10	1.8	Y	Grade	42.38	41.34	Google Street View
R2/480/20	1.2	N	Grade	42.91	41.91	Google Street View
R2/480/30	1.2	N	Grade	43.22	42.22	Google Street View
R2/480/40	Junction Pit			44.60	43.55	Google Street View
R2/480/50	1.2	N	Grade	44.61	43.56	Google Street View
R2/480/60	1.2	N	Grade	44.55	43.56	Google Street View
R2/480/70	1.2	N	Grade	44.38	43.61	Google Street View
R2/490/10	1.8	Y	Grade	43.87	42.92	Google Street View
R2/50/10	1.8	Y	Grade	24.96	24.11	Survey 2014
R2/50/20	1.8	Y	Grade	29.51	28.27	Survey 2014
R2/50/30	2.4	Y	Grade	29.21	28.28	Google Street View
R2/50/40	0.9	N	Grade	29.15	28.31	Google Street View
R2/500/10	1.8	Y	Grade	40.69	39.75	Google Street View
R2/510/10	1.8	Y	Grade	41.33	40.26	Google Street View
R2/520/10	1.8	Y	Grade	41.82	41.02	Survey 2010
R2/520/20		Y	Sag	44.07	42.94	Survey 2010
R2/520/30	1.9	Y	Sag	44.13	43.28	Survey 2010
R2/520/40		Y	Grade	44.49	43.48	Survey 2010
R2/520/50		Y	Grade	45.07	44.37	Google Street View
R2/520/60		Y	Grade	45.05	44.57	Google Street View
R2/530/10	1.8	Y	Sag	44.29	43.29	Google Street View
R2/540/10	1.8	Y	Grade	47.16	46.16	Google Street View
R2/560/10	1.8	Y	Grade	50.10	49.07	Google Street View

Pit ID	Lintel Length (m)	Grate?	Type	Surface Elevation (m)	Invert Elevation (m)	Source
R2/560/20	1.8	Y	Sag	50.73	49.79	Google Street View
R2/560/20a	1.8	Y	Grade	50.81	49.94	Google Street View
R2/570/10	0.6	Y	Grade	22.80	22.30	Google Street View
R2/570/10a	1.8	Y	Sag	22.38	21.88	Google Street View
R2/570/10b	1.8	Y	sag	22.33	21.83	Google Street View
R2/570/20	0.6	Y	Grade	23.00	22.50	Google Street View
R2/580/10		Y	Grade	21.47	20.20	Google Street View
R2/580/20		Y	Grade	21.75	21.25	Google Street View
R2/590/10	1.8	Y	Sag	22.45	21.95	Google Street View
R2/590/20	1.8	Y	Sag	22.52	22.02	Google Street View
R2/590/30	1.8	Y	Sag	22.97	22.47	Google Street View
R2/60/10	2.4	Y	Grade	25.21	24.18	Google Street View
R2/70/10	2.6	Y	Sag	24.63	23.42	Google Street View
R2/70/20	2.6	Y	Sag	24.56	23.66	Google Street View
R2/70/20a	1.8	Y	Grade	25.09	24.06	Google Street View
R2/70/20b	1.8	Y	Grade	25.32	24.35	Google Street View
R2/80/10	1.8	Y	Grade	25.61	24.59	Google Street View
R2/80/20	0.9	Y	Grade	26.97	26.01	Google Street View
R2/80/30	1.8	Y	Grade	27.92	26.91	Google Street View
R2/820/10	2	Y	Sag	22.24	20.05	Survey 2010
R2/820/10b	1.8	Y	Grade	22.34	21.34	Google Street View
R2/820/20	1	Y	Grade	22.38	20.25	Survey 2010
R2/90/20	1.8	Y	Grade	29.29	28.56	Survey 2014
R2/90/30	1.5	Y	Sag	29.88	28.85	Survey 2014
R2/90/40	1.8	Y	Grade	30.02	29.16	Survey 2014

Table D2: Stormwater Pipe Data

Pipe ID	Upstream Pit	Downstream Pit	Slope (%)	Length (m)	Size (m)	Diameter Interpolated?	Downstream Invert Elevation (m)	Upstream Invert Elevation (m)
P001	R2/50/30	R2/50/20	0.10	9.9	0.375 dia		28.27	28.28
P002	R2/50/20	R2/50/10	4.17	99.8	0.45 dia		24.11	28.27
P003	R2/50/10	R2/10/30	2.67	83.9	0.45 dia		21.87	24.11
P004	aR2/70/10	R2/10/30	1.17	70.5	1.5 dia		21.87	22.70
P005	R2/10/30	R2/10/20	0.89	22.2	1.5 dia		21.67	21.87
P006	R2/90/40	R2/90/30	2.34	13.2	0.375 dia		28.85	29.16
P007	R2/90/30	R2/90/20	1.72	16.9	0.375 dia		28.56	28.85
P008	R2/90/20	R2/10/55	3.59	136.8	0.375 dia		23.65	28.56
P009	R2/140/60	R2/140/50	4.98	14.5	0.675 dia		30.85	31.57
P010	R2/190/10	R2/140/60	0.90	10.6	0.375 dia		31.57	31.67
P011	R2/140/60a	R2/140/60	1.36	9.3	0.675 dia		31.57	31.70
P012	R2/140/80	R2/140/60a	0.40	161.2	0.45 dia		31.70	32.34
P013	R2/210/10	R2/140/80	6.00	12.1	0.375 dia		32.34	33.07
P014	R2/140/90	R2/140/80	5.16	8.1	0.375 dia		32.34	32.76
P015	R2/220/10	R2/140/90	4.46	20.2	0.375 dia		32.76	33.66
P016	R2/220/20	R2/220/10	2.45	18.0	0.375 dia		33.66	34.10
P017	R2/140/100	R2/140/90	1.66	56.6	0.375 dia		32.76	33.70
P018	R2/140/110	R2/140/100	3.40	48.0	0.375 dia		33.70	35.33
P019	R2/140/120	R2/140/110	2.56	20.3	0.375 dia		35.33	35.85
P020	R2/140/10	bR2/21800/10	0.45	59.6	0.675 dia		29.30	29.57
P021	R2/140/20	R2/140/10	1.97	29.5	0.675 dia		29.57	30.15
P022	R2/140/30	R2/140/20	2.06	17.9	0.675 dia		30.15	30.52
P023	R2/140/40	R2/140/30	0.99	13.1	0.675 dia		30.52	30.65
P024	R2/140/50	R2/140/40	1.25	16.0	0.675 dia		30.65	30.85
P025	R2/270/10	R2/260/10	3.47	9.1	0.375 dia		31.09	31.40
P026	R2/260/20	R2/260/10	5.42	31.3	0.45 dia		31.09	32.78
P027	R2/260/10	R2/10/110	2.31	44.0	0.75 dia		30.07	31.09
P028	R2/21700/10	R2/290/05	4.68	33.3	1.2 dia		30.10	31.66
P029	R2/290/30	R2/290/20	1.63	82.8	0.525 dia		31.60	32.95
P030	R2/360/10	R2/290/20	1.86	13.5	0.375 dia		31.60	31.85
P031	R2/290/20	R2/290/10	2.04	30.9	0.75 dia		30.97	31.60
P032	R2/290/05	R2/10/110	0.79	3.8	1.2 dia		30.07	30.10
P033	R2/280/10	R2/10/110	8.57	10.5	0.45 dia		30.07	30.97
P034	R2/290/10	R2/290/05	5.20	16.7	0.75 dia		30.10	30.97
P035	R2/420/10	R2/20800/10	2.68	96.1	0.525 dia		32.55	35.13
P036	R2/420/20	R2/420/10	2.27	99.4	0.525 dia		35.13	37.38
P037	R2/420/30	R2/420/40	0.84	12.2	0.375 dia		38.77	38.87
P038	R2/420/40	R2/440/10	1.28	10.0	0.375 dia		38.64	38.77

Pipe ID	Upstream Pit	Downstream Pit	Slope (%)	Length (m)	Size (m)	Diameter Interpolated?	Downstream Invert Elevation (m)	Upstream Invert Elevation (m)
P039	R2/440/10	R2/420/20	9.43	13.4	0.375 dia		37.38	38.64
P040	R2/10/280	R2/10/270	2.53	47.5	0.525 dia		40.68	41.88
P041	R2/10/290	R2/10/280	3.02	45.1	0.525 dia		41.88	43.24
P042	R2/10/300	R2/10/290	3.98	51.2	0.45 dia		43.24	45.28
P043	R2/540/10	R2/10/300	8.08	10.9	0.45 dia		45.28	46.16
P044	R2/10/310	R2/10/300	4.36	21.1	0.45 dia		45.28	46.20
P045	aR2/10/320	R2/10/320	0.03	31.9	0.375 dia		46.70	46.71
P046	R2/10/330	R2/10/320	4.25	64.9	0.45 dia		46.70	49.46
P047	R2/10/110	R2/10/98	0.34	42.9	1.5 w 0.75 h		29.92	30.07
P047a	R2/10/98	aR2/21800/80	0.34	39.8	1.5 w 0.75 h		29.79	29.92
P047b	aR2/21800/80	aR2/250/10	0.33	14.5	1.5 w 0.75 h		29.74	29.79
P048	R2/290/120	R2/290/110	1.61	83.5	0.375 dia		41.80	43.15
P049	R2/290/110	R2/290/100	1.61	11.7	0.375 dia		41.61	41.80
P050	R2/290/100	R2/290/90	1.39	9.4	0.375 dia		41.48	41.61
P051	R2/290/90	R2/290/80	3.94	12.6	0.375 dia		40.99	41.48
P052	R2/290/80	R2/290/70	3.53	79.3	0.375 dia		38.19	40.99
P053	R2/390/10	R2/290/70	0.10	10.6	0.375 dia		38.19	38.20
P054	R2/290/70	R2/290/60	3.53	31.0	0.375 dia		37.10	38.19
P055	R2/380/10	R2/290/60a	1.32	12.0	0.375 dia		37.67	37.83
P056	R2/290/60a	R2/290/60	5.53	10.3	0.375 dia	X	37.10	37.67
P057	R2/290/60	R2/290/50	2.99	85.3	0.375 dia		34.55	37.10
P058	R2/370/10	R2/290/50	0.26	11.0	0.375 dia		34.55	34.58
P059	R2/290/50	R2/290/40	2.58	11.5	0.375 dia		34.25	34.55
P060	R2/290/40	R2/290/30	6.88	18.9	0.375 dia		32.95	34.25
P061	R2/300/10	R2/290/10	1.48	12.5	0.45 dia		30.97	31.15
P062	R2/340/10	R2/290/10	1.47	22.5	0.675 dia		30.97	31.30
P063	R2/300/20	R2/300/10	0.93	110.4	0.375 dia		31.15	32.18
P064	R2/310/10	R2/300/20	4.61	14.9	0.375 dia		32.18	32.87
P065	R2/300/30	R2/300/20	4.35	71.9	0.375 dia		32.18	35.31
P066	R2/320/10	R2/300/30	4.23	15.2	0.375 dia		35.31	35.95
P067	R2/300/40	R2/300/30	4.10	84.1	0.375 dia		35.31	38.75
P068	R2/300/50	R2/300/40	3.98	36.4	0.375 dia		38.75	40.20
P069	R2/330/10	R2/300/40	4.51	33.2	0.375 dia		38.75	40.25
P070	R2/340/20	R2/340/10	1.16	195.8	0.675 dia		31.30	33.56
P071	R2/340/30	R2/340/20	0.49	17.1	0.6 dia		33.56	33.65
P072	aR2/340/50	R2/340/30	2.97	79.2	0.45 dia		33.65	36.00
P073	R2/340/40	aR2/340/50	4.18	68.5	0.525 dia		36.00	38.86
P074	R2/340/50	R2/340/40	1.54	11.5	0.525 dia		38.86	39.04
P075	R2/350/30	R2/350/20	0.22	8.3	0.375 dia		40.52	40.54
P076	R2/350/20	R2/350/10	6.86	12.1	0.45 dia		39.69	40.52

Pipe ID	Upstream Pit	Downstream Pit	Slope (%)	Length (m)	Size (m)	Diameter Interpolated?	Downstream Invert Elevation (m)	Upstream Invert Elevation (m)
P077	R2/350/10	aR2/340/50	3.56	103.7	0.45 dia		36.00	39.69
P078	R2/400/10	R2/21700/10	1.48	13.0	0.375 dia		31.66	31.85
P079	R2/20900/10	R2/21700/10	1.99	32.4	1.2 dia		31.66	32.30
P080	R2/410/10	R2/20900/10	3.42	72.0	0.375 dia		32.30	34.77
P081	R2/410/20	R2/410/20a	5.22	15.4	0.375 dia		35.47	36.27
P082	R2/410/20a	R2/410/10	4.15	16.9	0.375 dia	X	34.77	35.47
P083	R2/10/140	R2/20900/10	0.16	128.6	1.2 dia		32.30	32.51
P084	R2/430/10	R2/420/10	3.18	13.7	0.375 dia		35.13	35.56
P085	R2/450/20	R2/450/10	0.83	11.5	0.375 dia		38.93	39.03
P086	R2/450/10	R2/420/20	6.18	25.2	0.375 dia		37.38	38.93
P087	R2/10/150	R2/10/140	0.79	10.1	1.2 dia		32.51	32.59
P087a	R2/20800/10	R2/10/140	0.77	5.2	0.375 dia		32.51	32.55
P088	R2/10/160	R2/10/150	0.77	114.9	1.05 dia		32.59	33.48
P089	R2/10/170	R2/10/160	0.54	13.7	1.05 dia		33.48	33.55
P090	R2/10/180	R2/10/170	1.26	138.4	1.05 dia		33.55	35.29
P091	R2/10/190	R2/10/180	1.26	14.3	1.05 dia		35.29	35.47
P092	R2/10/210	R2/10/190	1.10	154.4	0.9 dia		35.47	37.17
P093	R2/470/20	R2/470/10	4.33	9.9	0.375 dia		38.00	38.43
P094	R2/470/10	R2/10/210	6.22	13.4	0.375 dia		37.17	38.00
P095	R2/460/10	R2/10/210	4.35	29.9	0.45 dia		37.17	38.47
P096	R2/10/220	R2/10/210	1.23	13.8	0.9 dia		37.17	37.34
P096A	aR2/20700/10a	R2/20700/10	0.33	11.1	0.375 dia	X	38.46	38.50
P096ab	R2/20700/10	R2/10/210	82.41	1.6	0.375 dia		37.17	38.46
P097	R2/10/230	R2/10/220	0.30	8.4	0.9 dia		37.34	37.37
P098	R2/10/240	R2/10/230	15.48	10.4	0.9 dia		37.37	38.97
P099	R2/480/10	R2/10/230	6.67	59.5	0.375 dia		37.37	41.34
P100	R2/480/20	R2/480/10	6.20	9.2	0.375 dia		41.34	41.91
P101	R2/480/30	R2/480/20	3.72	8.3	0.375 dia		41.91	42.22
P102	R2/490/10	R2/480/30	6.92	10.1	0.375 dia		42.22	42.92
P103	R2/480/70	R2/480/60	0.43	11.6	0.375 dia		43.56	43.61
P104	R2/480/60	R2/480/50	0.04	7.1	0.375 dia		43.56	43.56
P105	R2/480/50	R2/480/40	0.07	10.2	0.375 dia		43.55	43.56
P106	R2/480/40	R2/480/30	2.46	54.1	0.375 dia		42.22	43.55
P107	R2/10/250	R2/10/240	0.20	80.0	0.9 dia		38.97	39.13
P108	R2/10/260	R2/10/250	1.09	22.2	0.9 dia		39.13	39.38
P109	R2/500/10	R2/10/250	6.87	9.0	0.375 dia		39.13	39.75
P110	R2/510/10	R2/10/260	7.76	11.4	0.375 dia		39.38	40.26
P111	R2/10/270	R2/10/260	2.57	50.8	0.75 dia		39.38	40.68
P112	R2/520/10	R2/10/270	3.59	9.4	0.525 dia		40.68	41.02
P113	R2/520/20	R2/520/10	2.39	80.6	0.45 dia		41.02	42.94

Pipe ID	Upstream Pit	Downstream Pit	Slope (%)	Length (m)	Size (m)	Diameter Interpolated?	Downstream Invert Elevation (m)	Upstream Invert Elevation (m)
P114	R2/520/30	R2/520/20	3.50	9.5	0.45 dia		42.94	43.28
P115	R2/530/10	R2/520/30	0.24	6.2	0.45 dia		43.28	43.29
P116	R2/520/40	R2/520/30	2.54	8.0	0.225 dia		43.28	43.48
P116a	R2/520/50	R2/520/40	2.59	34.4	0.3 dia		43.48	44.37
P116b	R2/520/60	R2/520/50	1.68	11.9	0.3 dia		44.37	44.57
P117	R2/10/320	R2/10/310	3.55	14.1	0.45 dia		46.20	46.70
P118	R2/560/20a	R2/560/20	2.09	6.9	0.375 dia	X	49.79	49.94
P119	R2/560/20	R2/560/10	2.40	30.0	0.375 dia		49.07	49.79
P120	R2/560/10	R2/10/320	2.38	99.8	0.375 dia		46.70	49.07
P121	R2/260/30	R2/260/20	2.07	24.4	0.375 dia		32.78	33.28
P122	R2/260/40	R2/260/30	2.44	45.0	0.375 dia		33.28	34.38
P123	R2/250/10	aR2/250/10	2.90	21.6	0.375 dia		29.74	30.37
P124	R2/240/10	aR2/240/10	1.91	43.1	0.375 dia		29.67	30.49
P125	R2/240/20	R2/240/10	2.81	92.1	0.375 dia		30.49	33.08
P126	R2/240/30	R2/240/20	3.90	86.7	0.375 dia		33.08	36.46
P127	R2/240/20a	R2/240/20	1.42	8.9	0.375 dia		33.08	33.21
P128	R2/240/40	R2/240/30	4.92	21.2	0.375 dia		36.46	37.50
P129	R2/230/10	aR2/230/10	5.87	9.2	0.375 dia		29.46	30.00
P130	bR2/21800/10	R2/21800/10	3.52	17.1	0.675 dia		28.70	29.30
P131	R2/150/10	R2/140/20	4.72	42.8	0.375 dia		30.15	32.17
P132	R2/160/10	R2/150/10	1.85	12.7	0.375 dia		32.17	32.40
P133	R2/150/20	R2/150/10	4.48	83.7	0.375 dia		32.17	35.92
P134	R2/170/10	R2/150/20	0.21	12.2	0.375 dia		35.92	35.95
P135	R2/150/20a	R2/150/20	5.28	28.3	0.375 dia		35.92	37.42
P136	R2/150/30	R2/150/20a	4.70	72.3	0.375 dia		37.42	40.81
P137	R2/180/10	R2/150/30	2.34	13.1	0.375 dia		40.81	41.12
P138	R2/150/40	R2/150/30	5.05	87.6	0.375 dia		40.81	45.24
P139	R2/150/50	R2/150/40	4.85	13.9	0.375 dia		45.24	45.91
P140	R2/150/60	R2/150/50	1.23	9.1	0.375 dia		45.91	46.03
P141	R2/130/50	R2/130/40	1.03	11.7	0.45 dia		33.13	33.25
P142	R2/130/40	R2/130/30	0.11	10.1	0.45 dia		33.12	33.13
P143	R2/130/30	R2/130/20	1.39	12.8	0.45 dia		32.94	33.12
P144	R2/130/20	R2/130/10	3.49	11.1	0.45 dia		32.56	32.94
P145	R2/130/10	R2/21800/10	2.55	151.6	0.45 dia		28.70	32.56
P146	R2/21800/10	R2/10/78	0.82	11.9	1.35 dia		28.60	28.70
P147	R2/120/30	R2/120/20	0.25	7.5	0.375 dia		27.54	27.55
P148	R2/120/20	R2/120/10	0.21	17.1	0.375 dia		27.50	27.54
P149	R2/120/10	aR2/120/10	6.93	11.8	0.375 dia		26.68	27.50
P149A	R2/10/77a	aR2/120/10	4.81	21.0	0.375 dia	X	26.68	27.69
P150	R2/110/20	R2/110/10	0.41	9.0	0.375 dia		27.20	27.24

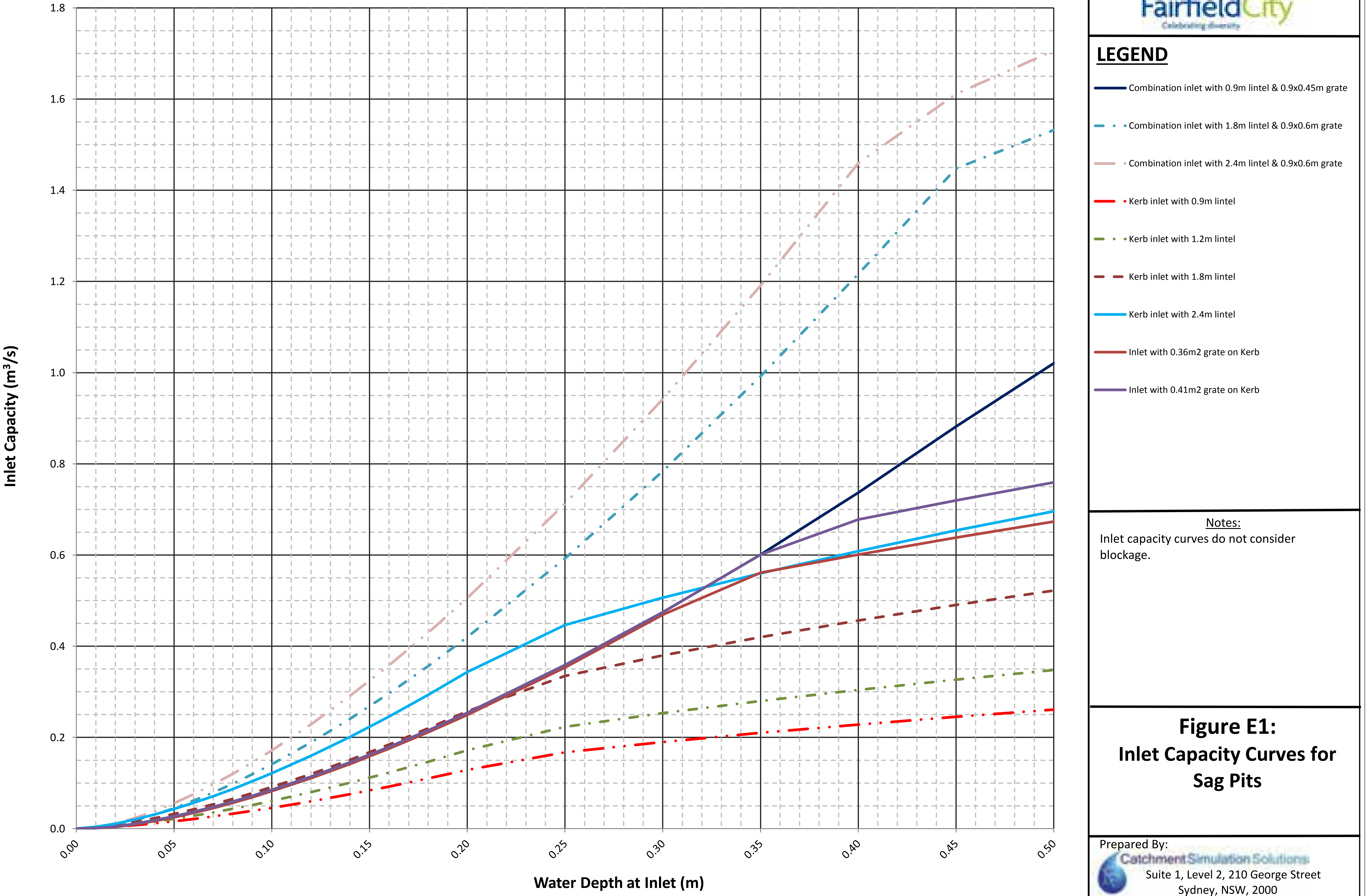
Pipe ID	Upstream Pit	Downstream Pit	Slope (%)	Length (m)	Size (m)	Diameter Interpolated?	Downstream Invert Elevation (m)	Upstream Invert Elevation (m)
P151	R2/110/10	aR2/110/10	3.45	17.8	0.375 dia		26.58	27.20
P152	R2/10/77	aR2/120/10	1.73	32.2	1.35 dia		26.68	27.24
P153	aR2/120/10	aR2/110/10	1.20	8.3	1.35 dia		26.58	26.68
P154	aR2/110/10	R2/10/70	1.08	124.0	1.35 dia		25.24	26.58
P155	R2/10/70	R2/10/60	0.62	23.1	1.8 dia		25.10	25.24
P156	R2/100/10	R2/10/60	4.18	22.1	0.3 dia		25.10	26.02
P156a	R2/10/60a	R2/10/60	3.61	38.9	0.3 dia		25.10	26.50
P157	R2/10/60	R2/10/55	1.46	99.0	1.5 dia		23.65	25.10
P158	R2/10/55	R2/10/50	0.23	52.0	1.5 dia		23.53	23.65
P159	R2/10/50	R2/10/40	1.00	11.8	1.5 dia		23.41	23.53
P160	R2/80/10	R2/10/50	4.94	21.6	0.45 dia		23.53	24.59
P161	R2/80/20	R2/80/10	2.29	62.0	0.375 dia		24.59	26.01
P162	R2/80/30	R2/80/20	3.45	26.1	0.375 dia		26.01	26.91
P163	R2/10/40	aR2/70/10	1.06	67.0	1.5 dia		22.70	23.41
P164	R2/70/20	R2/70/10	2.84	8.7	0.45 dia		23.42	23.66
P164A	R2/70/20a	R2/70/20	1.88	21.0	0.375 dia	X	23.66	24.06
P164B	R2/70/20b	R2/70/20a	3.44	8.5	0.375 dia	X	24.06	24.35
P165	R2/70/10	aR2/70/10	1.67	42.9	0.45 dia		22.70	23.42
P166	R2/50/40	R2/50/30	0.34	8.8	0.375 dia		28.28	28.31
P167	R2/60/10	R2/50/10	0.52	12.8	0.45 dia		24.11	24.18
P168	R2/40/10	R2/10/20	7.53	16.4	0.375 dia		21.67	22.90
P169	R2/30/10	R2/10/20	8.19	9.8	0.375 dia		21.67	22.47
P170	R2/10/20	R2/10/15	0.37	33.8	1.5 dia		21.55	21.67
P171	R2/20/20	R2/20/10	0.89	9.2	0.375 dia		23.16	23.24
P172	R1/720/20	R1/720/10a	0.72	99.5	0.375 dia		19.49	20.21
P173	R1/720/10a	R1/720/10	0.41	34.8	0.375 dia		19.35	19.49
P174	R1/720/10	R1/700/09	0.93	26.7	0.375 dia		19.10	19.35
P174A	R1/710/10	R1/700/09	1.54	25.1	0.375 dia		19.10	19.49
P174B	R1/700/09a	R1/700/09	0.78	29.0	0.375 dia		19.10	19.33
P175	R1/20300/10	R1/700/09	2.39	25.3	1.2 dia		19.10	19.70
P175A	R1/20200/10	R1/20300/10	0.17	63.5	1.2 dia		19.70	19.81
P175B	R1/20100/10	R1/700/20	2.48	76.8	1.05 dia		20.19	22.10
P175B1	R1/700/20	R1/700/15	0.63	19.3	1.05 dia		20.07	20.19
P175B2	R1/700/15	R1/20200/10	0.63	41.6	1.05 dia		19.81	20.07
P176	R1/700/09	aR1/20400/10	1.52	55.7	1.2 dia		18.25	19.10
P176ac	aR1/20400/10	R1/700/07	4.86	31.9	1.2 dia		16.70	18.25
P176new	R1/20400/10a	aR1/20400/10	0.81	12.4	0.45 dia		19.00	19.10
P177	R1/700/190	R1/700/180	3.52	15.2	0.375 dia		41.08	41.62
P178	R1/700/180	R1/700/170	4.66	4.9	0.375 dia		40.85	41.08
P179	R1/700/170	R1/700/160	2.72	10.3	0.375 dia		40.57	40.85

Pipe ID	Upstream Pit	Downstream Pit	Slope (%)	Length (m)	Size (m)	Diameter Interpolated?	Downstream Invert Elevation (m)	Upstream Invert Elevation (m)
P180	R1/700/160	R1/700/150	11.18	10.0	0.375 dia		39.45	40.57
P181	R1/700/150	R1/700/140	4.91	86.4	0.375 dia		35.21	39.45
P182	R1/700/140	R1/700/130	3.37	7.9	0.375 dia		34.94	35.21
P183	R1/700/130	R1/700/120	3.73	25.3	0.375 dia		34.00	34.94
P184	R1/700/120	R1/700/110	2.19	12.4	0.375 dia		33.73	34.00
P185	R1/700/110	R1/700/100	4.84	58.9	0.375 dia		30.88	33.73
P186	R1/810/10	R1/700/100	0.68	11.5	0.375 dia		30.88	30.95
P187	R1/700/100	R1/700/90	2.59	83.9	0.375 dia		28.70	30.88
P188	R1/800/10	R1/700/90	0.33	11.2	0.375 dia		28.70	28.74
P189	R1/700/90	R1/700/80	1.08	43.5	0.375 dia		28.23	28.70
P190	R1/790/20	R1/790/10	1.58	9.7	0.375 dia		28.47	28.62
P191	R1/790/10	R1/700/80	2.09	11.1	0.45 dia		28.23	28.47
P192	R1/700/80	aR1/700/70	5.11	58.2	0.375 dia		25.26	28.23
P193	aR1/700/70	R1/700/70	0.79	6.6	0.525 dia		25.21	25.26
P194	R1/700/70	R1/700/65	0.16	74.7	0.525 dia		25.09	25.21
P195	R1/770/10	R1/700/65	0.45	24.1	0.45 dia		25.09	25.20
P196	R1/700/65	R1/19700/10	0.78	59.4	1.05 dia		24.63	25.09
P197	R1/19700/10	R1/700/50	3.23	64.7	1.05 dia		22.54	24.63
P198	R1/700/50	R1/20100/10	1.51	29.0	1.05 dia		22.10	22.54
P199	R1/780/20	R1/780/10	2.19	12.5	0.375 dia		32.61	32.88
P200	R1/780/10	R1/780/09	2.77	34.4	0.525 dia		31.66	32.61
P201	R1/780/09	R1/780/07	2.94	84.2	0.525 dia		29.18	31.66
P202	R1/780/07	R1/780/06	18.59	18.8	0.525 dia		25.69	29.18
P203	R1/780/06	R1/700/65	0.59	101.2	0.75 dia		25.09	25.69
P204	R2/10/78	R2/10/77	1.22	111.7	1.35 dia		27.24	28.60
P205	R2/10/15		0.68	33.3	1.5 dia		21.32	21.55
P206	R2/20/10	R2/10/15	3.38	47.7	0.45 dia		21.55	23.16
P207		R2/820/20	7.77	3.3	1.65 dia		20.25	20.51
P208	R2/820/20	R2/820/10	0.55	36.9	1.65 dia		20.05	20.25
P209	R1/830/20	R1/830/10	2.53	10.0	0.45 dia		21.27	21.52
P210	R1/830/10	R2/820/10	13.55	9.0	0.45 dia		20.05	21.27
P211	aR2/820/10		1.50	67.8	1.65 dia		19.00	20.02
P212	R2/820/10	aR2/820/10	0.04	75.3	1.65 dia		20.02	20.05
P213	R2/820/10b	aR2/820/10	7.22	18.2	0.375 dia	X	20.02	21.34
P214	R1/740/20	R1/740/10	3.11	11.7	0.675 dia		23.05	23.41
P215	R1/740/10	R1/20100/10	4.73	20.0	0.675 dia		22.10	23.05
P216	R1/740/30	R1/740/20	3.00	99.6	0.675 dia		23.41	26.40
P216a	R1/740/20a	R1/740/20	6.86	12.8	0.375 dia		23.41	24.29
P217	R1/740/40	R1/740/30	2.29	57.4	0.675 dia		26.40	27.71
P217b	R1/740/60	R1/740/50	0.09	12.6	0.45 dia		29.18	29.19

Pipe ID	Upstream Pit	Downstream Pit	Slope (%)	Length (m)	Size (m)	Diameter Interpolated?	Downstream Invert Elevation (m)	Upstream Invert Elevation (m)
P217c	R1/740/50	R1/740/40a	1.29	47.5	0.45 dia		28.57	29.18
P217d	R1/740/40a	R1/740/40	1.78	48.0	0.45 dia		27.71	28.57
P217e	R1/750/10	R1/740/40a	8.90	14.7	0.45 dia		28.57	29.88
P217f	R1/750/20	R1/750/10	5.44	28.8	0.45 dia		29.88	31.45
P217h	R1/760/10	R1/750/10	1.01	9.0	0.45 dia		29.88	29.97
P218	aR2/250/10	aR2/240/10	0.34	21.1	1.5 w 0.75 h		29.67	29.74
P219	aR2/240/10	R2/10/90	0.34	25.7	1.5 w 0.75 h		29.58	29.67
P220	R2/10/90	aR2/230/10	0.32	38.9	1.2 w 0.9 h		29.46	29.58
P221	aR2/230/10	R2/21800/10	0.82	92.6	1.5 w 0.75 h		28.70	29.46
PipeNun1	R2/590/30	R2/590/20	0.69	64.8	0.375 dia		22.02	22.47
PipeNun2	R2/590/20	R2/590/10	0.67	10.0	0.45 dia		21.95	22.02
PipeNun3	R2/590/10	R2/580/20	7.11	9.9	0.45 dia		21.25	21.95
PipeNun4	R2/580/20	R2/580/10	1.22	85.9	1.05 dia		20.20	21.25
PipeNun5	R2/580/10		1.29	93.3	1.05 dia		19.00	20.20
PipeRON1	R2/570/20	R2/570/10	1.03	19.9	0.375 dia		22.30	22.50
PipeRON2	R2/570/10	R2/570/10a	0.53	79.3	0.375 dia		21.88	22.30
PipeRON3	R2/570/10a	R2/570/10b	0.51	9.1	0.375 dia		21.83	21.88
PipeRON4	R2/570/10b		1.39	38.3	0.375 dia		21.30	21.83

APPENDIX E

STORMWATER PIT INLET CAPACITY CURVES



Fairfield City

Celebrating diversity

Notes:

Inlet capacity curves do not consider blockage.

Figure E1: Inlet Capacity Curves for Sag Pits

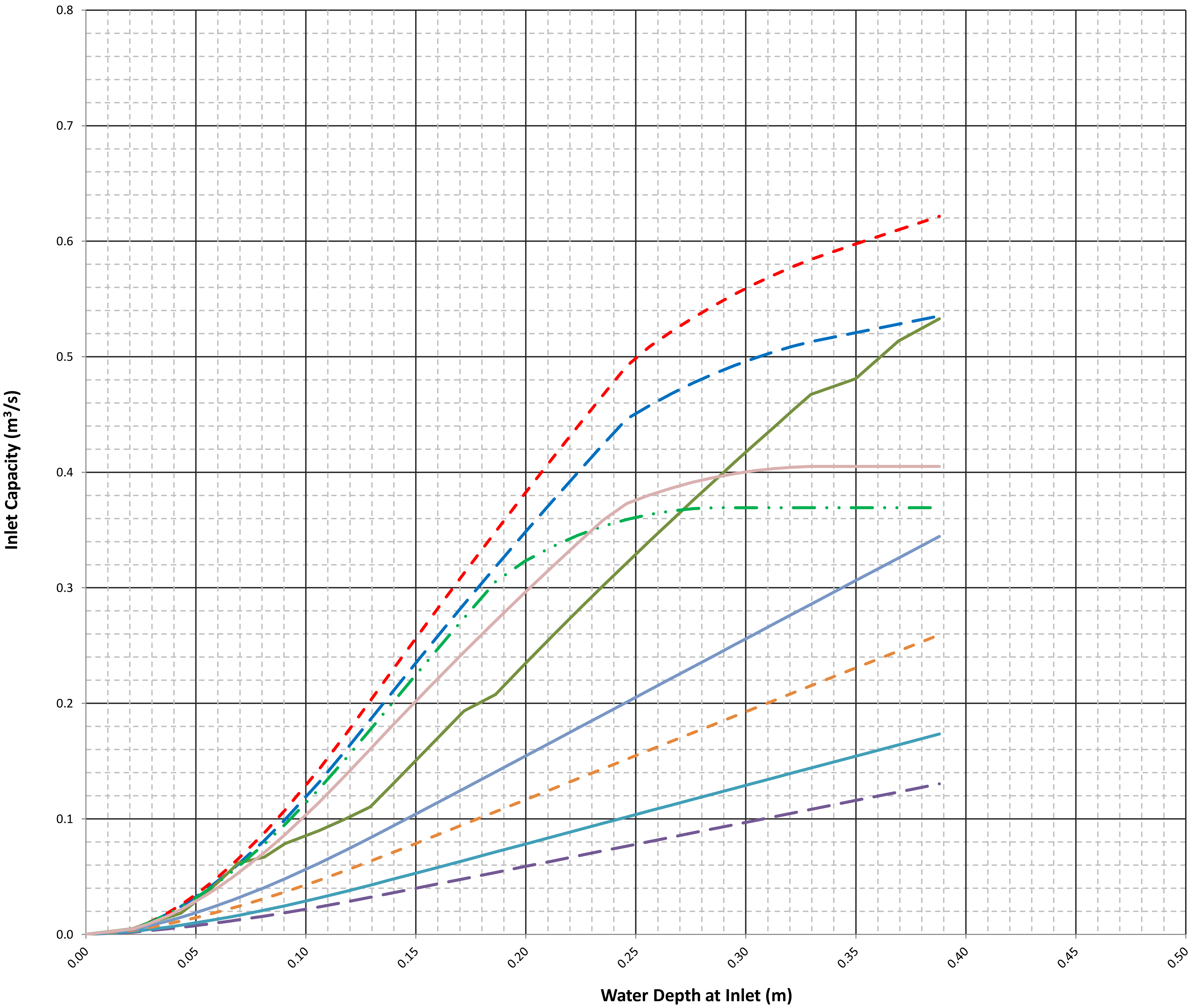
Prepared By:

Catchment Simulation Solutions

Suite 1, Level 2, 210 George Street

Sydney, NSW, 2000

File Name: Inlet Capcacity Curves.xls



Fairfield City
Celebrating diversity

LEGEND

- Combination inlet with 0.9m lintel & 0.9x0.45m grate
- Combination inlet with 1.8m lintel & 0.9x0.45m grate
- Combination inlet with 2.4m lintel & 0.9x0.45m grate
- Combination inlet with 2.6m lintel & 0.9x0.45m grate
- Kerb inlet with 0.9m lintel
- Kerb inlet with 1.2m lintel
- Kerb inlet with 1.8m lintel
- Kerb inlet with 2.4m lintel
- Inlet with 0.36m2 grate on Kerb
- Inlet with 0.41m2 grate on Kerb

Notes:
Inlet capacity curves do not consider blockage.

Figure E2:
Inlet Capacity Curves for On Grade Pits

Prepared By:
Catchment Simulation Solutions
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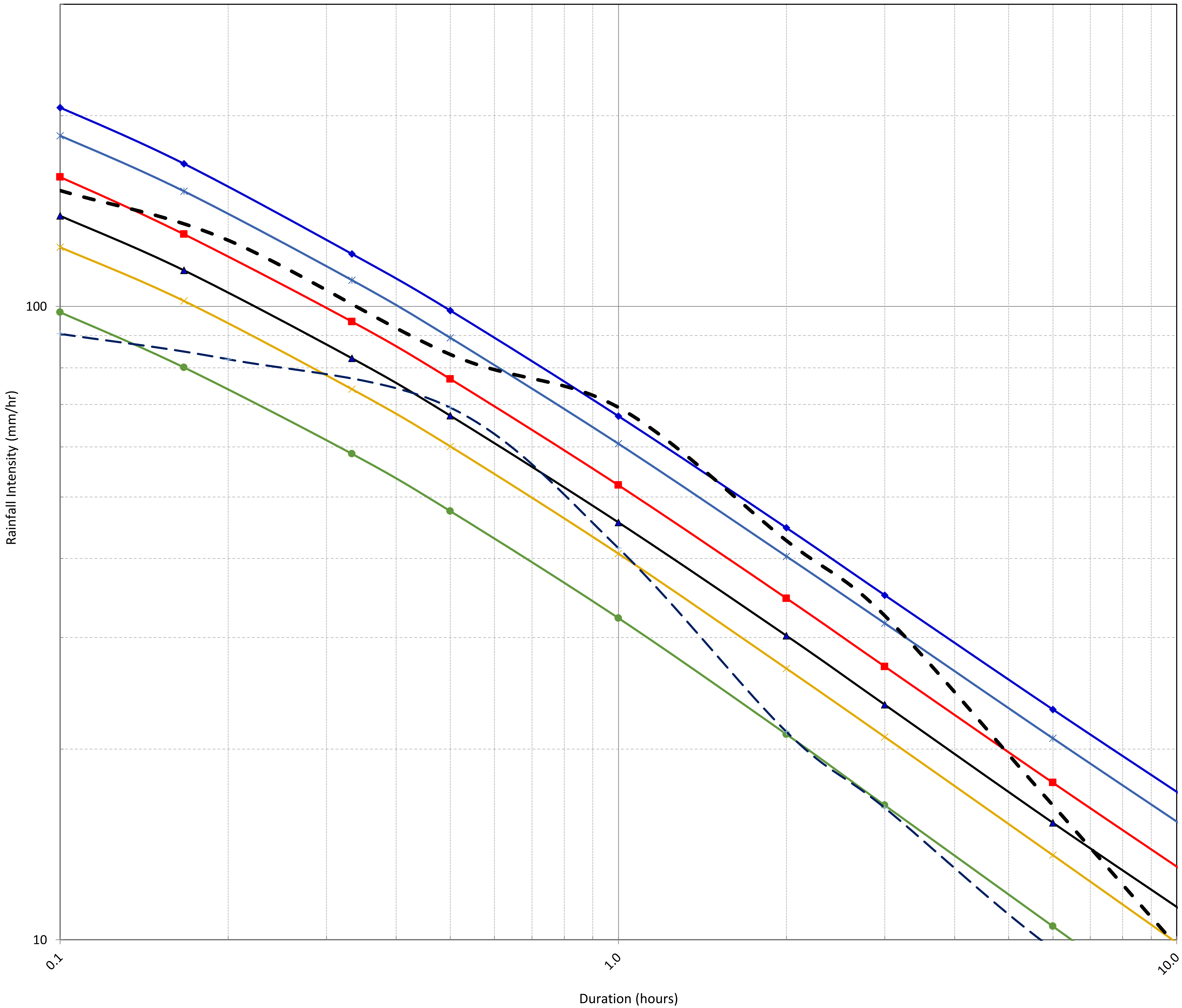
Sydney, NSW, 2000

File Name: Inlet Capcacity Curves.xls

Appendix F

Historic Rainfall Vs IFD Curves



FairfieldCity
Celebrating diversity

LEGEND:

- 1% AEP
- 2% AEP
- 5% AEP
- 10% AEP
- 20% AEP
- 50% AEP
- February 1990 Rainfall
- April 2012 Rainfall

Notes:

Figure F1:
Design Intensity - Frequency - Duration Curves Vs Historic Rainfall

Prepared By:
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Sydney, NSW, 2000

File Name: Smithfield IFD Curves.xlsx

APPENDIX G

THIRD PARTY REVIEW

REVIEW OF SMITHFIELD WEST OVERLAND FLOOD STUDY

Prepared for:	Catchment Simulation Solutions
Prepared by:	BMT WBM Pty Ltd (Member of the BMT group of companies)

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	Title:	REVIEW OF SMITHFIELD WEST OVERLAND FLOOD STUDY
	Project Manager:	Bowen Hicks
	Author:	Bowen Hicks
	Client:	Catchment Simulation Solutions (CSS)
	Client Contact:	David Tetley
	Client Reference:	S20131
Synopsis:		

REVISION/CHECKING HISTORY

Revision Number	Date	Checked by		Issued by	
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DISTRIBUTION

Destination	Revision										
	0	1	2	3	4	5	6	7	8	9	10
Catchment Simulation Solutions	1										
BMT WBM File	1										
BMT WBM Library											

Contents

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1 Introduction

BMT WBM has been requested by Catchment Simulation Solutions (CSS) to undertake a review of the Smithfield West TUFLOW flood study model prior to CSS undertaking design modelling.

The review focused on the following aspects:

- Overall model health in 1D and 2D domains (suitable time-step, mass balance reports, instabilities, etc.);
- Model Schematisation (i.e. pipe linkages, 1D-2D pipe connections, pipe surcharging, etc.);
- Representation of fences as flow constriction lines;
- Appropriateness of model parameterisation (1D and 2D roughness/blockage/form loss parameters, etc.); and
- Suitability of boundary conditions (upstream inflows and downstream creek schematisation).

When undertaking the review, model re-runs and sensitivity model runs were required. All model results presented in this review are based on the 1% AEP 2 hour event with 50% blockage scenario.

2 Hydrologic Review Aspects

2.1 Rainfall IFD

AR&R87 IFD values have been extracted for location 33.850S, 150.925E and compared to a sample of IFD values within TUFLOW bc_dbase. PMF rainfall values have also been calculated using the following parameters and compared with a sample of values within the TUFLOW bc_dbase.

(1) Area=1.75km^2

(2) Roughness=100%

(3) Elevation Adjustment Factor=1.0

(4) Moisture Adjustment Factor=0.7

A comparison between the rainfall depth values determined in the review versus the values used in the Smithfield West model is shown in Table 2-1. The results indicate that there are no differences in the rainfall values for all design rainfalls up to the 100 year ARI. There are minor differences in results for the PMF event with a maximum difference of 2%.

Table 2-1 Percent difference in rainfall storm depths of Smithfield West model versus reviewer determined values.

Duration	Difference (%) in Rainfall Storm Depths for Modelled ARI						
	2 year	5 year	10 year	20 year	50 year	100 year	PMF
30Mins	0	0	0	0	0	0	2
1Hr	0	0	0	0	0	0	2
2Hrs	0	0	0	0	0	0	2
3Hrs	0	0	0	0	0	0	2

2.2 Catchment Area based on Hydrograph Volume

A check for potential gross errors was undertaken by “calculating” the rainfall depth at a downstream location from the volume in a hydrograph and comparing it to the known rainfall depth.

Flow was recorded from the 1D and 2D domain at the downstream location shown Figure 2-1. Figure 2-2 shows the recorded hydrographs at this location. The upstream area draining to this location is known to be 1.24 km2.

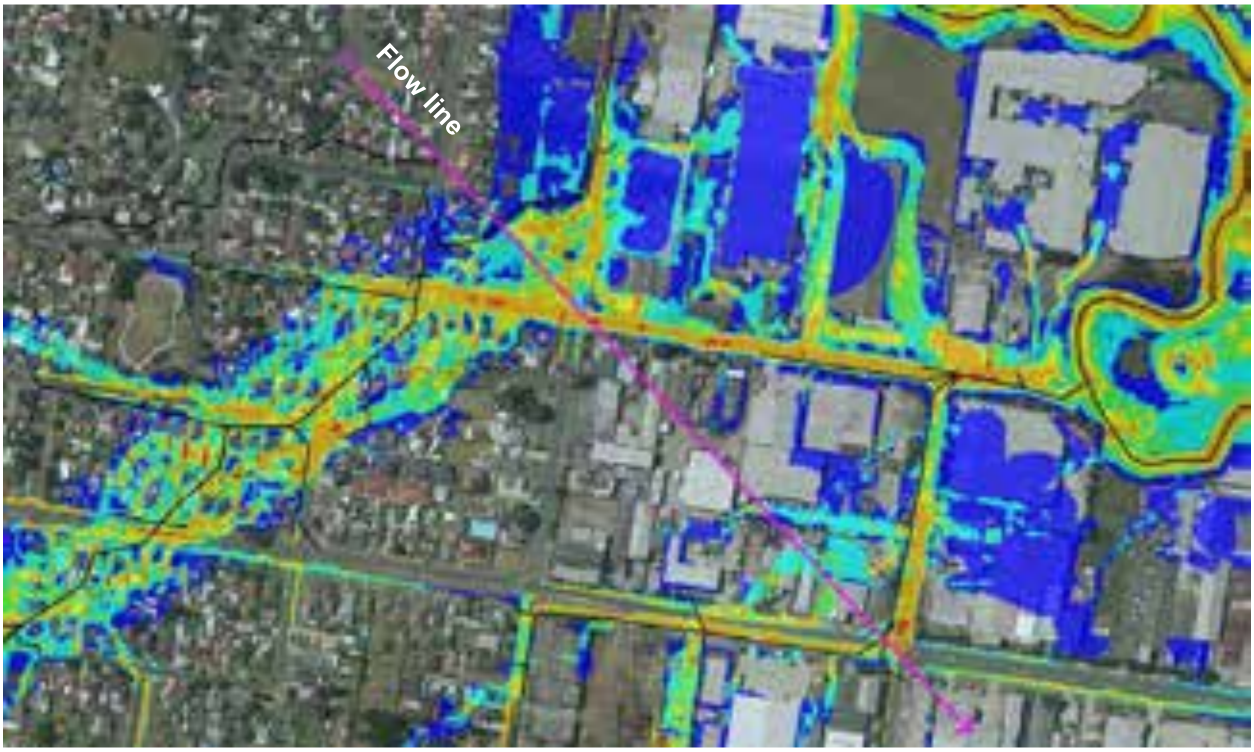


Figure 2-1 Check for anticipated flows (gross error check)

The excess rainfall depth calculated was 77.5 mm (volume in hydrograph ÷ upstream area). Noting the rainfall losses presented later in Section 3.4, this value is within expected range and confirms that the correct rainfall has been applied to the model.

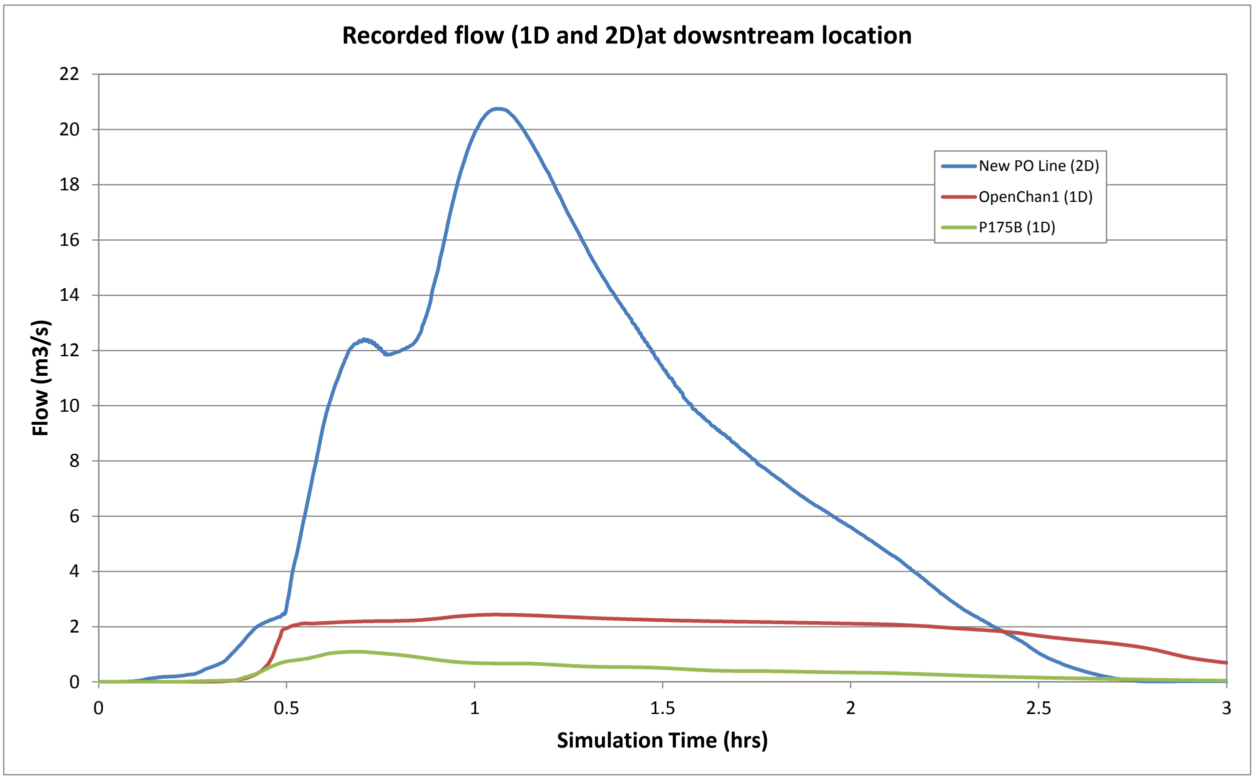


Figure 2-2 Recorded flow hydrograph at location shown in Figure 2-1

3 Hydraulic Review Aspects

3.1 Grid Size

A 2m grid size has been modelled. This is suitable for the Smithfield West Catchment which is predominantly urban.

3.2 TUFLOW build and precision

64 bit compiled TUFLOW build “2013-12-AD” has been run in double precision. The “Cell Wet/Dry Depth” has been set to the minimum value of 0.2mm. This is appropriate for the direct rainfall modelling approach adopted for this study.

3.3 Model Domain

Using the provided DEM (SmithfieldDEM.txt), the catchment boundary has been calculated using an automatic method in a GIS program called Global Mapper. The catchment boundary defined by Global Mapper was found to be consistent with the TUFLOW model boundary and direct rainfall polygon.

3.4 2D Materials Code

CSS adopted a method of classifying land use types by automatic remote sensing techniques. Figure 3-1 shows the materials code (land-use) derived from the TUFLOW “grd_check” layer. As shown in Figure 3-1, the land-use is as described in the report and provides a very good representation of the land use types. Table 3-1 presents a copy of the rainfall infiltration parameters from the CSS report. All rainfall infiltration values are within the range of acceptable values.

Table 3-1 Documented rainfall losses values (Table 3 – CSS report)

Material Description	Rainfall Losses	
	Initial Loss *(mm)*	Continuing Loss Rate *(mm/hr)*
Grass	10-20	2.5
Trees	10-20	2.5
Shrubs	10-20	2.5
Roads	1	0.0
Concrete	1	0.0
Water	0	0.0
Buildings	1	0.0
Crops	10-20	2.5

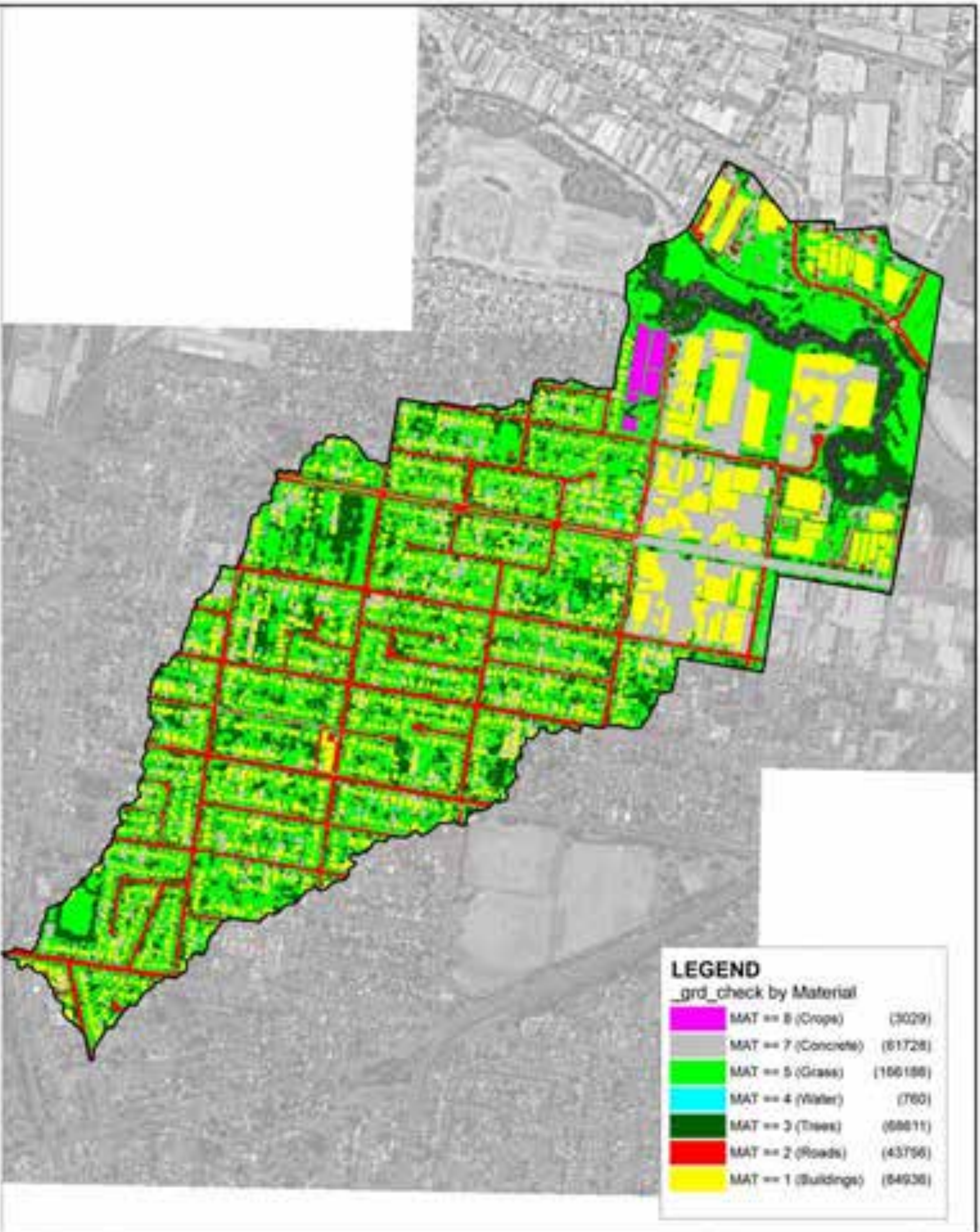
Table 3-2 presents a copy of the Manning’s “n” roughness values from the CSS report. Depth varying values have been adopted for each land use type. With the exception of the Buildings layer, all roughness values are within range of acceptable values. For depths above 0.1m, a Manning’s “n” value of 0.1 has been assumed for buildings. It is suggested that this value is too low and should be closer to 1.0 to replicate the obstruction of the building footprint to flows.

Table 3-2 Documented Manning's "*n*" roughness values (Table 4 – CSS report)

Material Description	Depth Varying Manning's 'n' Values							
	$Depth_1$ *(metres)*	'n'$_1$	$Depth_2$ *(metres)*	'n'$_2$	$Depth_3$ *(metres)*	'n'$_3$	$Depth_4$ *(metres)*	'n'$_4$
Grass	0.03	0.110	0.05	0.075	0.07	0.055	0.10	0.03
Trees	0.30	0.137	1.50	0.110	2.00	0.080		
Shrubs	0.30	0.137	1.00	0.077	1.50	0.047		
Roads	0.04	0.017	0.10	0.021	0.15	0.020		
Concrete	0.01	0.034	0.01	0.020				
Crops	0.10	0.130	0.50	0.093	>0.50	0.059		
Buildings	0.03	0.030	0.10	0.100				
Water	0.013 for all depths							

Buildings have additionally been raised to the known floor level or 0.15m above the maximum ground surface. This method is appropriate however care needs to be taken to ensure that flow paths are available between buildings. Review suggests that the buildings are spaced sufficiently far apart (in the context of grid size) to facilitate flow between properties when required.

It is noted that a higher confidence could be placed on the direct rainfall Manning's "*n*" values if flows from a sample sub catchment in the TUFLOW model were compared against flows from an established hydrologic model such as WBNM or RAFTS.



Title:
Land Use

Figure: 3-2

Rev: 1

0 250 500m
Approx. Scale

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3.5 Fences

Approximately 21,000 fence lines have been created for the study area. These fences can act to significantly alter the distribution of the flows through the catchment.

The fences have been reported to be modelled as partial blockages using a “layered flow constriction” within TUFLOW. This approach to modelling fences is suitable. Fences modelled by the approach cannot ‘fail’ and will always withstand flows.

It is noted that all fences were assumed to be 1.0m high and the bottom opening height of some fences in floodways was 0.4m to allow overland flows. Since the top of each fence line was assumed to be flat despite a sloped ground surface, the height of the fence and height of the opening was not always as reported. Figure 3-2 and Figure 3-3 presents a histogram of the modelled fence heights and fence opening heights. As presented, the reported heights are not “exact” but rather the “average” values.

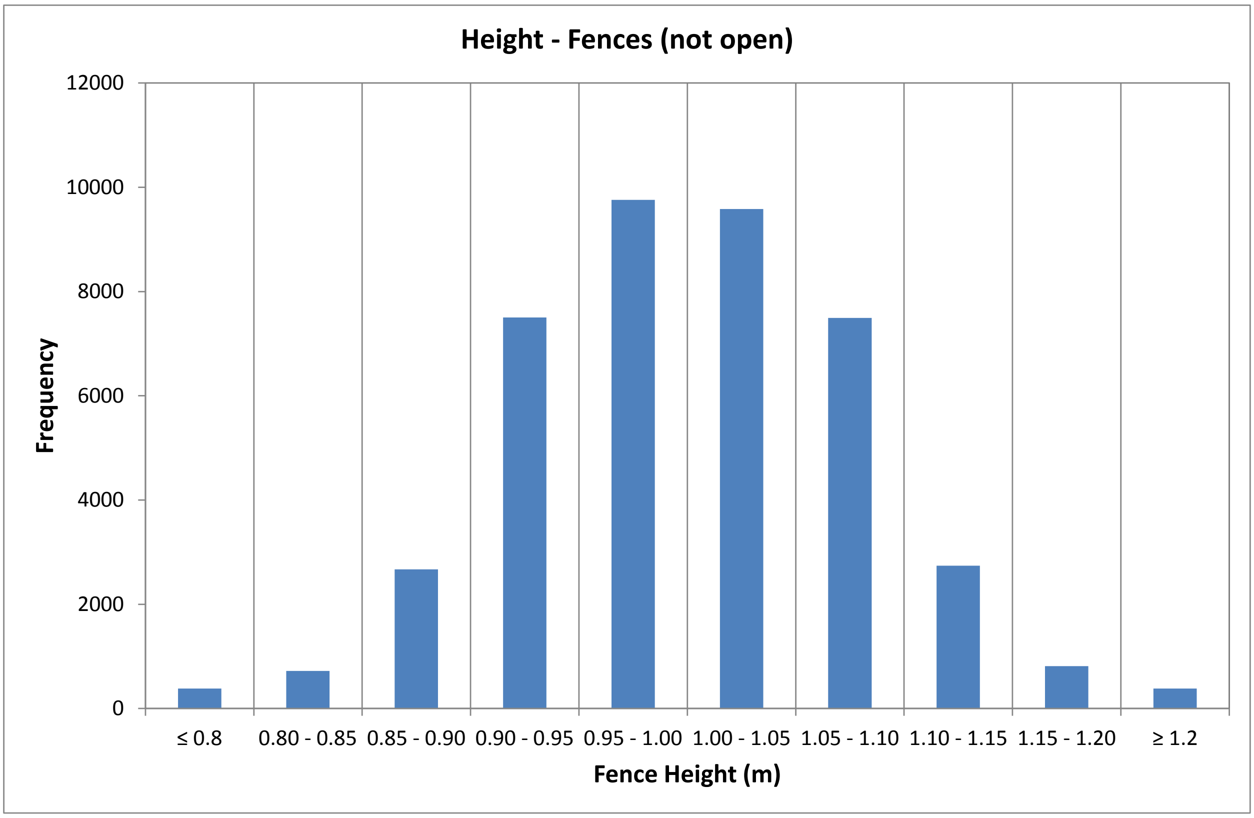


Figure 3-2 Height of fences (reported to be 1.0m)

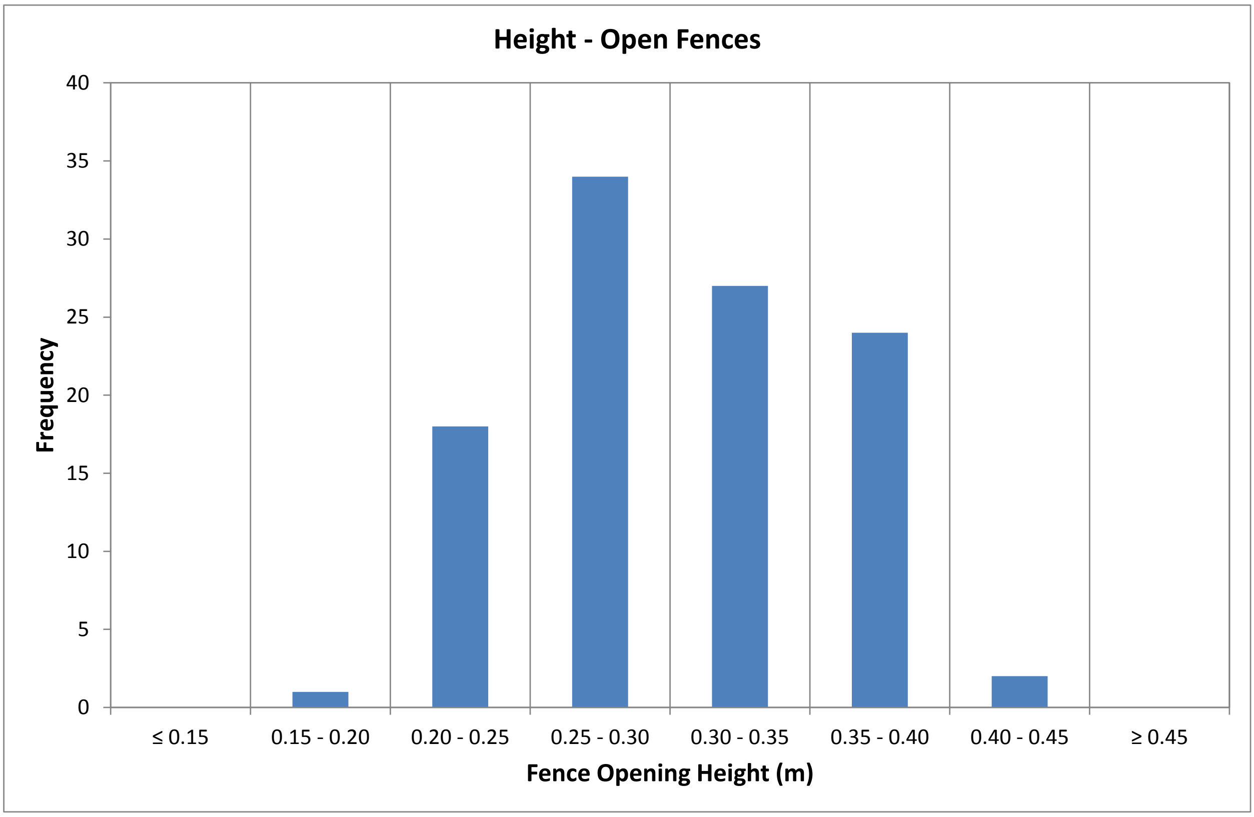


Figure 3-3 Height of fence opening (reported to be 0.4m – average value is 0.3m)

The implications for the fence height not always being 1.0m is non-significant since very few flood depths close to 1.0m interact with fences. The implications for the fence opening height being over or underestimated may be more significant since there are more flows paths in the depth range of the intended opening height (i.e. ~0.3m or 0.4m).

If the fence height data was defined in a separate point layer, sloping fence levels (hence accurate fence heights) could be schematised in the TUFLOW model as detailed in the following extract from the TUFLOW manual.

> As described in Section 4.4.11, the point objects can be placed in a separate layer for a 2d_fcsh layer. In this case, only the first two attributes are required.

In regards to widespread modelling of fences, it is suggested that this may unduly attenuate catchment flows since many fences may fail during a flood event. A sensitivity assessment was undertaken to determine if the high number of fences schematised in the model results in an impact on flood levels and flows in the catchment for the 1% AEP 2 hour design flood event.

Figure 3-4 shows the recorded flow hydrograph (at the location shown in Figure 2-1) for the base case model and the sensitivity simulation which had all fences removed. As shown the peak flow is increased by approximately 11% in the scenario were all fences are removed.

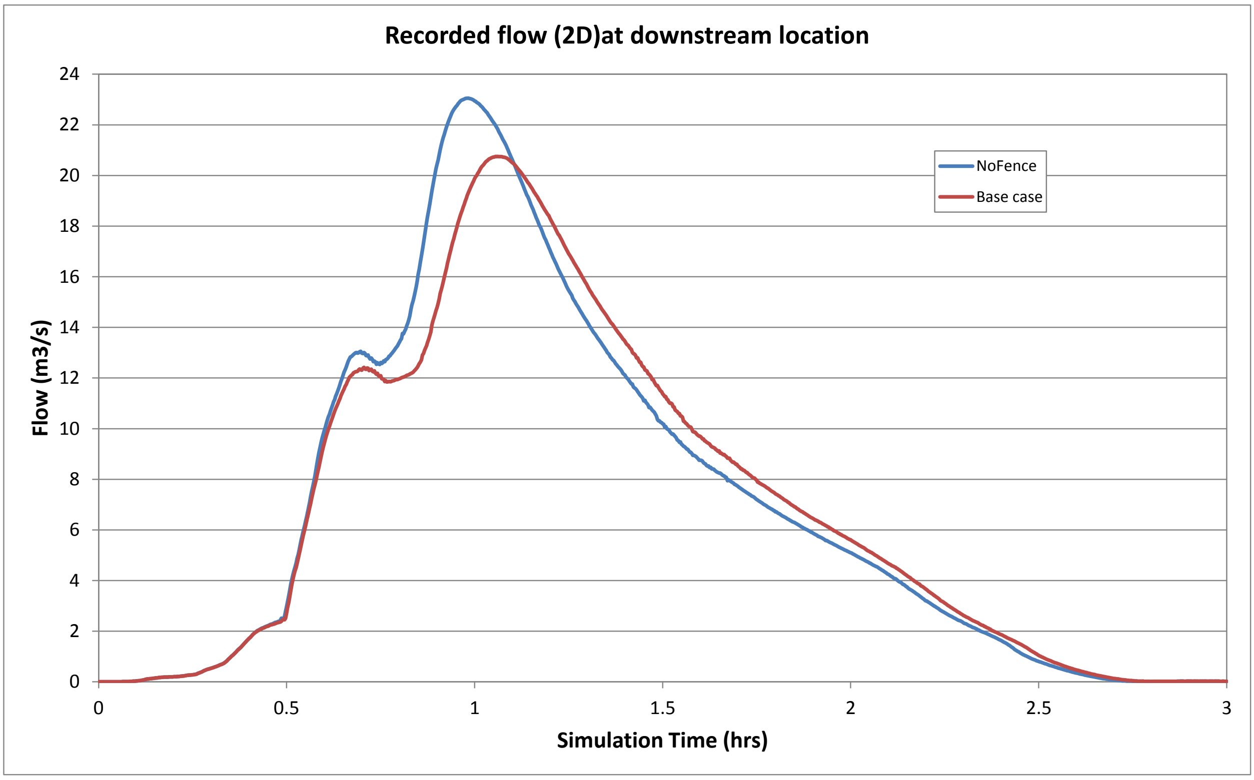
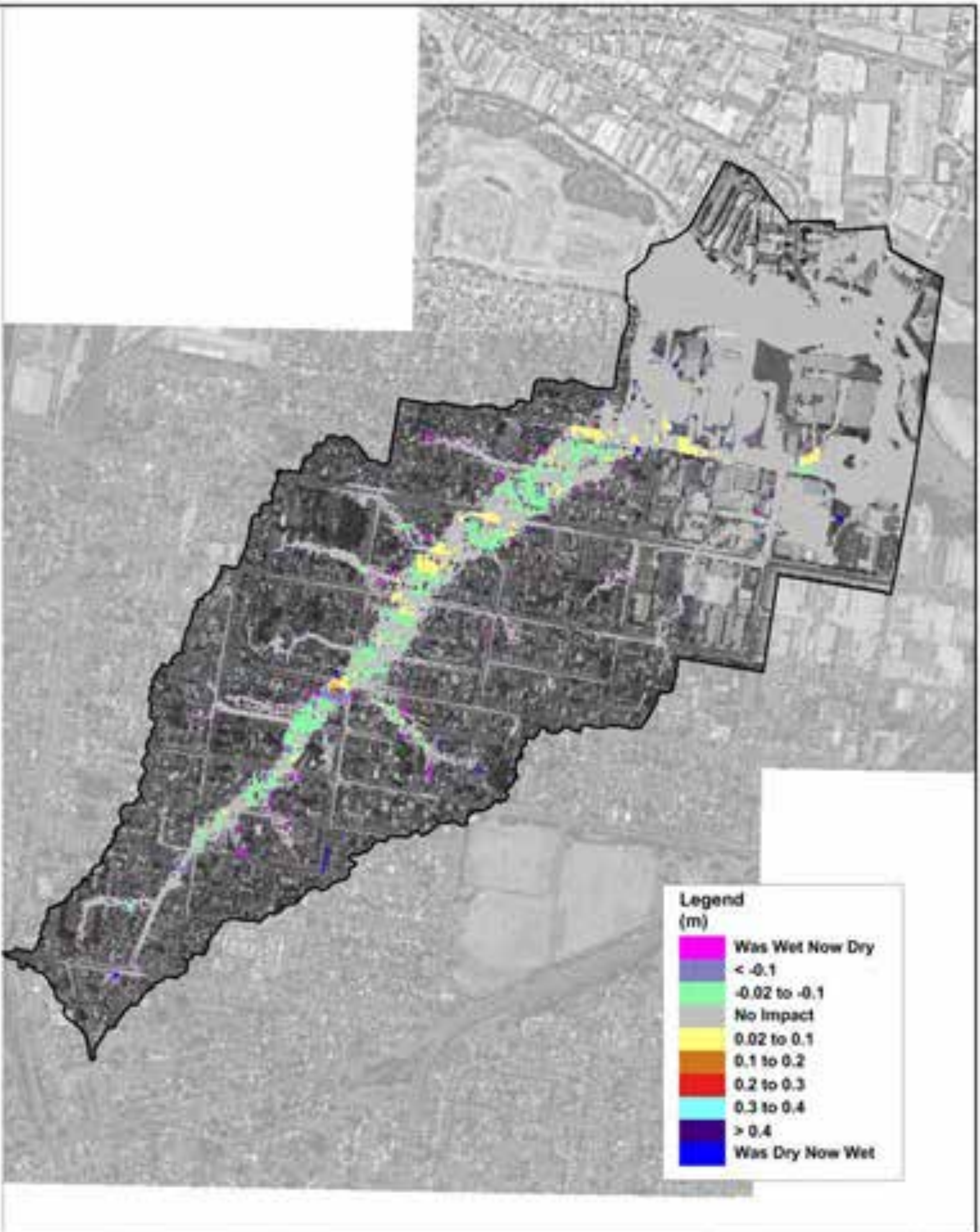


Figure 3-4 Flow hydrograph difference from fences

Figure 3-5 shows the peak flood level sensitivity analysis to the removal of all fences in the hydraulic model. When reviewing Figure 3-5, attention has focussed on areas where flood levels are increased by the removal of fences. As shown, there are some locations where flood levels have the potential to increase when the fences are removed from the model. Flood level increases are typically less 0.05m therefore may not be considered significant.

CSS and Council may want to consider creating design flood envelopes based on results “with” and “without” fences.



Title:

Fence removal sensitivity assessment

Figure: 3-5

Rev: 1

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

0 250 500m

Approx. Scale

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3.6 1D Stormwater Network Parameterisation

3.6.1 Pits and Pipes

Typical parameterisation of circular pipe elements is shown below. The adopted values are appropriate.

Manning's "*n*" == 0.011 – 0.013

Form Loss == 0.

Entry/Exit Losses* == 0.5/1.0

Width Contraction Loss == 0.9 – 1.0

Length == drawn length (auto)

** Manholes override these loss values*

Typical parameterisation of rectangular pipe elements is shown below. The adopted values are appropriate.

Manning's "*n*" == 0.011

Form Loss == 0.

Entry/Exit Losses* == 0.5/1.0

Height/Width Contraction Loss == 0.6/0.9

Length == drawn length (auto)

** Manholes override these loss values*

Typical parameterisation of nodes and pit elements (Type=Q) is shown below. The adopted values are appropriate.

Form Loss == 0.

ANA == 0.

Conn_2D (Topo_ID) == SX

It is noted that the 2013-12 TUFLOW release notes introduce the following new command which CSS may wish to take advantage of:

Pit Default Road Crossfall == <slope>

This command increases the depths at Q pits based on the height of an imaginary triangle of the road cross-section with a crossfall slope of <slope>. The larger depth will push more water into the pit.

3.6.2 Manholes

No manually created manholes have been applied in the model. Manholes have been automatically created by TUFLOW. The following lists the adopted values and the default TUFLOW values.

Manhole Default C Exit Coefficient == 0.10 (Default == 0.25)

Manhole Default R Exit Coefficient	== 0.20	(Default == 0.5)
Manhole Default Side Clearance	== 0.10	(Default == 0.3)
Manhole K Maximum Bend/Drop	== 1.0	(Default == 4.)
Manhole Minimum Dimension	== 0.5	(Default == 1.05)
Manhole Default Type	== C	(Default == CJR)

The adopted values are lower than the default values meaning that the automatically calculated manhole losses are low. Low losses result in higher potential pipe flow and lower overland flood levels.

The CSS Report states that the TUFLOW subsurface drainage has been compared to the performance of an existing ILSAX model. If CSS is confident in the ILSAX schematisation and parameterisation then this is appropriate, otherwise default values for manhole losses could be used which will result in more conservative flood levels.

3.6.3 Channels

Open channels have been defined for Prospect Creek at the downstream limits of the catchment and also for swales higher up in the catchment. Typical parameterisation of open channel elements is shown below.

Manning's "*n*"	== 0.04 – 0.1 (Prospect Creek == 0.08)
Form Loss	== 0. (Single reach in Prospect Creek is 0.2)
Length	== manually defined

It is noted that is some instances there is a difference between the drawn length and the manually defined length. The manual length should be updated or allowed to be automatically calculated by TUFLOW.

It is noted that the width of the 1D channel representation matches the width of the cross section data. This correctly represents the storage available in the model.

3.7 Pipe Integrity Assessment

A series of pipe integrity tests were undertaken to identify any pipes which were not connected properly, had negative cover or negative slopes. The following presents results of the pipe integrity assessment:

There are connection issues for the following pipe ID's:

(1) P211

(2) Holroyd_Culv

(3) Overflow_Weir

There are negative slope issues for the following pipe ID's:

(4) P078

(5) P089

(6) P129

(7) P196

(8) P198

(9) P215

3.8 Boundary Conditions

Figure 3-6 shows the TUFLOW model boundary conditions.

The intention of modelling Prospect Creek and Holroyd inflow is to provide a sloped water level boundary at the downstream limits of the model. Prospect Creek aspects are therefore discussed further in Section 3.8.1 Downstream Boundary Conditions, while direct rainfall is discussed in Section 3.8.2 Inflow Boundary Conditions.

3.8.1 Downstream Boundary Conditions

Prospect Creek has been included in the Smithfield West TUFLOW model. Prospect Creek is an important feature of the study area since local catchment runoff interacts with this boundary and some downstream water levels are primarily driven by flooding from Prospect Creek.

Upstream inflow boundaries are delivered into the 2D domain via flow-time inflow boundaries. The downstream boundary draws water from the 2D domain via an automatic stage-discharge boundary. The automatic boundary requires an assumed water slope (b=0.01) and calculates the discharge for the given depth.

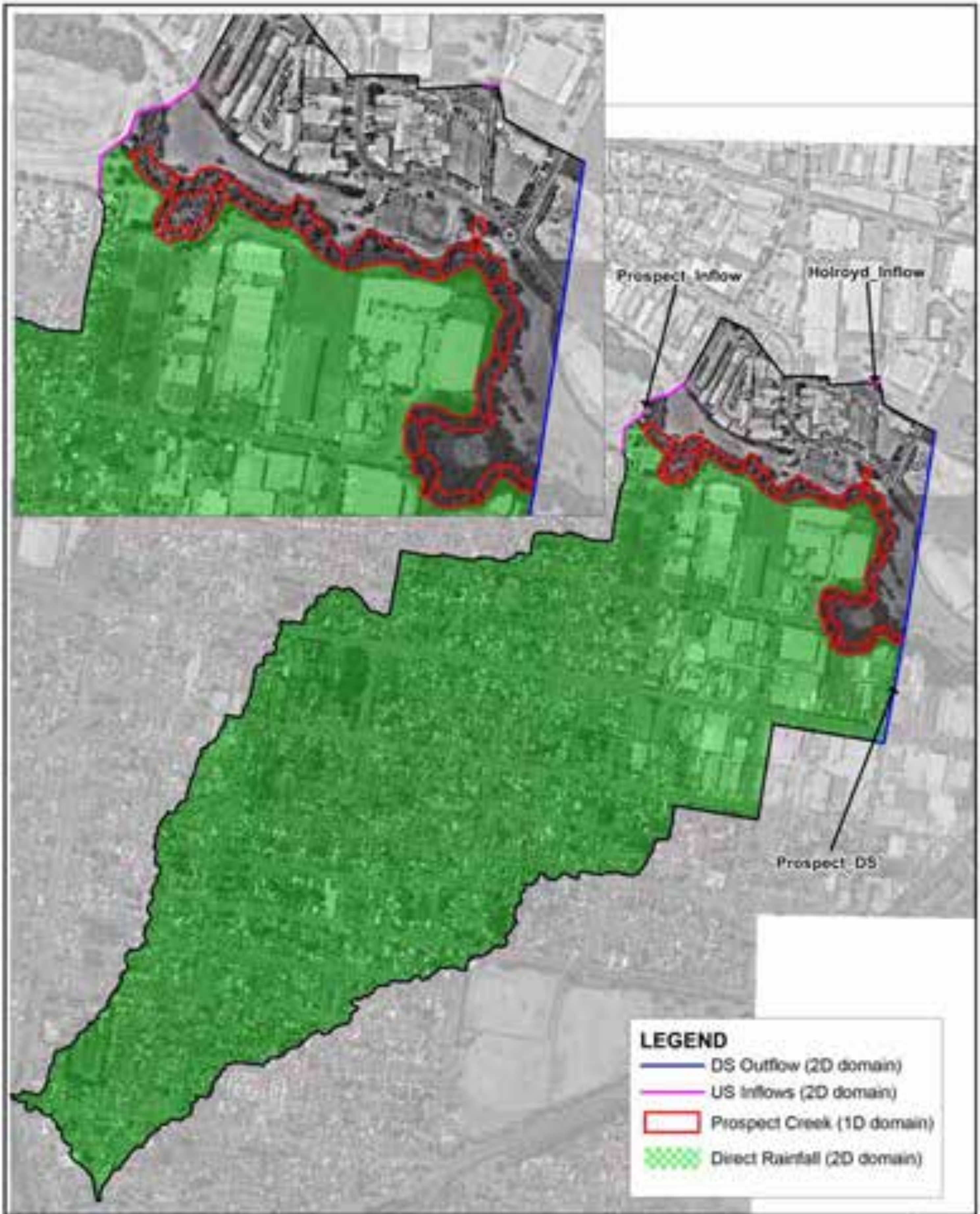
Despite using 2D domain Prospect Inflow boundaries, the conveyance of Prospect Creek is modelled as a 1D feature.

For the benefit of the modeller, a more elegant solution to modelling Prospect Creek would have been to model the upstream inflows and downstream stage-discharge as 1D boundary condition. For flows which exceed the in-bank capacity of the creek, HX lines (similar location to current 2D inflow and outflow boundaries) could be drawn.

This would have the advantage of reducing the number of deep 2D cells in the creek in-bank which may limit model 2D time step (Courant stability number) and also reduce the number of challenging 1D to 2D flow boundaries which can sometimes cause stability problems. TUFLOW Tutorial 4 presents an example of how to do this.

In principle, the adopted schematisation approach is suitable. The following qualifications are however made:

1. Sensitivity analysis needs to be presented to confirm that flood levels in the reporting area are not influence by the assumed slope (auto HQ boundary);
2. Manning's "n" of 0.08 for 1D creek is within the expected range. If possible, modelled creek water levels should be compared to previous study to confirm that Prospect Creek is generating consistent water levels;
3. The upstream Prospect Creek boundary may need to be extended further upstream, west of Rosford Street Reserve embankment (see Figure 3-7 for explanation).



Title:

Boundary Conditions

Figure: 3-6

Rev: 1

0 250 500m

Approx. Scale

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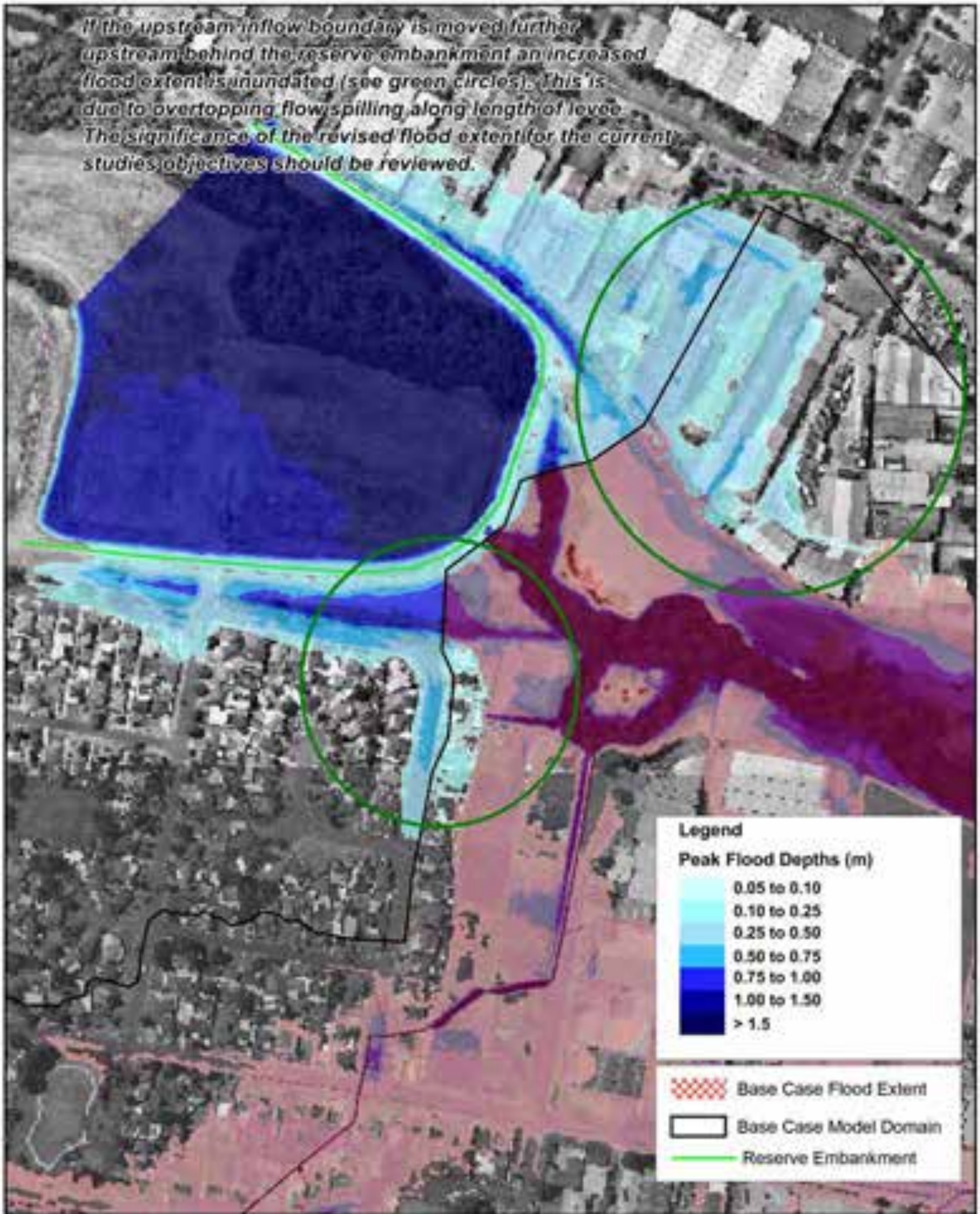
Filepath : S:\WATER\PROJECTS\S20078 - Water - Botany Bay Foreshore Beach FS\Drafting\WOR\ReportBody_Figures\



Title:

Sensitivity of moving upstream boundary further upstream

Figure: 3-7

Rev: 1

BMT WBM

www.bmtwbm.com.au

Filepath : S:\WATER\PROJECTS\S20078 - Water - Botany Bay Foreshore Beach FS\Drafting\WOR\ReportBody_Figures\

3.8.2 Inflow Boundary Conditions

A direct rainfall polygon has been defined to allocate flow to the entire local catchment. This approach is valid and indeed allows the utmost resolution in modelling minor flow paths.

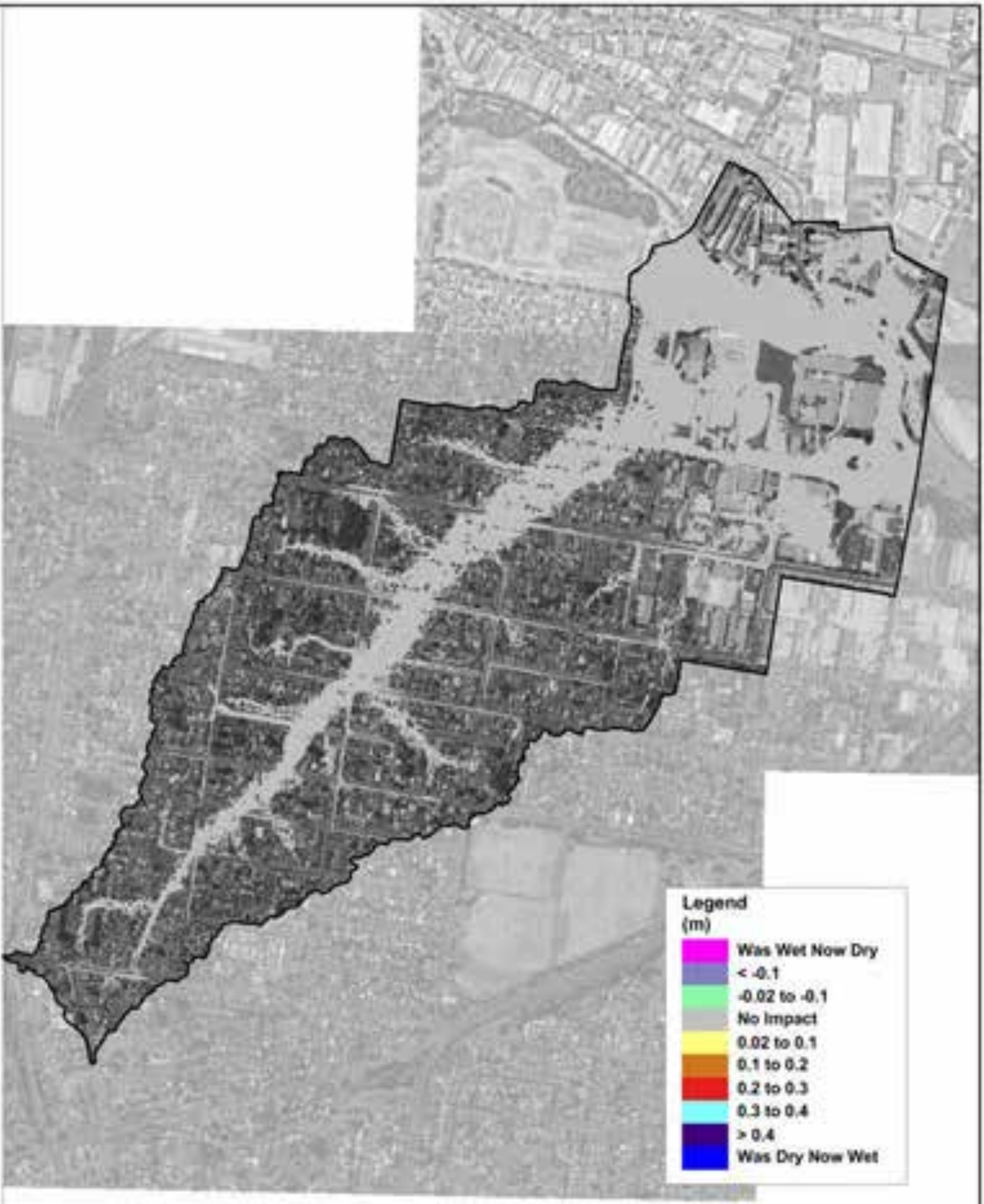
3.9 Model Health

3.9.1 Time Step

As a rule of thumb the 2D model time step should be approximately ¼ to ½ of the model grid size. It is expected a 2m model should therefore have a 2D time step between 0.5 to 1.0 seconds. The 2D time step for the Smithfield West model is 0.5 seconds which is suitable. The 1D time step is 0.25 which is suitable.

A time step higher than the standard range may generate mass error from poor convergence. A time step lower than the standard range may indicate that the model has been poorly schematised and the time step parameter has been used to manage poor schematisation.

Another useful step that should be undertaken to confirm the suitability of the time step is to assess the sensitivity of the time step. This has been undertaken as part of the review. A model simulation has been run with a 0.1 second 2D step time step and 0.05 second 1D time step and the results compared with the standard scenario. If the model results are sensitive to the reduced time step, then the original time step choice may be inappropriate or there may be underlying schematisation errors. Figure 3-8 shows the peak flood level sensitivity analysis to the reduced time step. As shown, the model results (except at highly localised locations) are insensitive to time step reduction. This suggests the model time step is appropriate and gives confidence in the model accuracy.



Title:
Time step sensitivity assessment

Figure:
3-8

Rev:
1

0 250 500m
Approx. Scale

BMT WBM
www.bmtwbm.com.au

Filepath : S:\WATER\PROJECTS\S20078 - Water - Botany Bay Foreshore Beach FS\Drafting\WOR\ReportBody_Figures

3.9.2 Mass Error

TUFLOW automatically reports mass error estimates for the 1D domain, 2D domain and total mass error combined from both domains during the simulation. Ideally the values should tend to be within ±1% (TUFLOW manual, 2010). Some models can have higher mass errors at the beginning of a simulation when rapid wetting and drying occurs which may be acceptable if it rapidly falls within acceptable limits.

Figure 3-9 shows the combined 1D and 2D domain cumulative mass error report for the 1% AEP 2 hour design storm. The initial mass error of 8% indicates poor initial model health though at the peak of the storm event (approximately 1 hour into simulation) the mass error is within acceptable limits. While improvements to model convergence could be made the mass error at the peak of the flood wave is within acceptable limits and so therefore acceptable.

The mass error can be significantly improved by initialising the model appropriately (i.e. Set IWL == 19.0). This has been confirmed by modelling undertaken as part of the review.

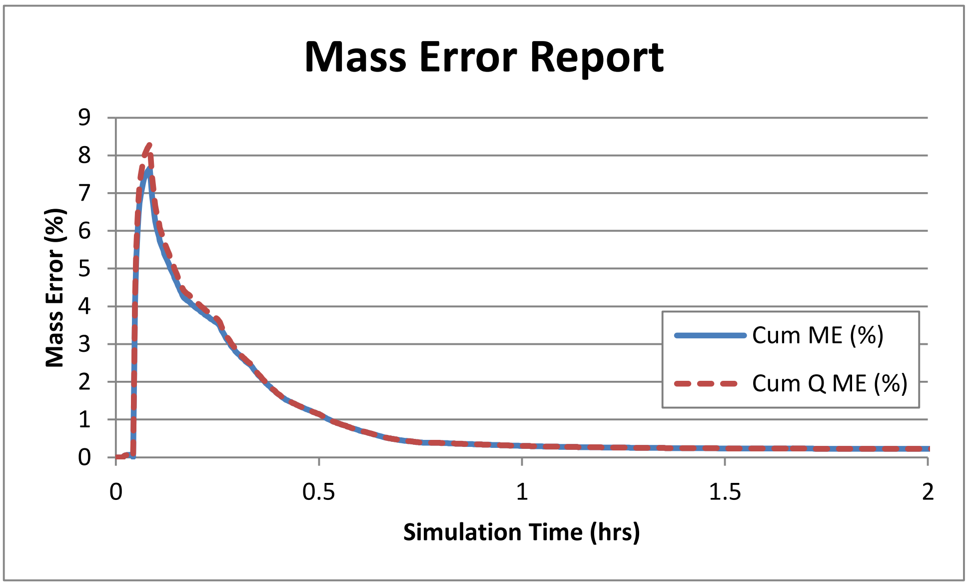


Figure 3-9 Combined 1D/2D domain ME for simulation

3.10 Projection

TUFLOW undertakes checks to confirm the projection of each GIS layer is consistent. This check is required since TUFLOW cannot reproject GIS input layers.

All GIS layers have the correctly defined projection of MGA 56. In defining the project however some layers use the Datum number of 116 and some layers use the datum number of 33 which both represent the ellipsoid GRS 80. Presumably this has resulted from different computers with different versions of MapInfo being used to build the model.

The TUFLOW projection error reporting has been suppressed to produce warnings. Prior to delivering the final flood study model it is recommended that error reporting be re-instated and Datum numbers of GIS layers be made consistent. This will reduce the likelihood of future users of the model introducing errors.

4 Conclusions and Recommendations

Generally a very well built model with innovative methods of classifying land use and properties. The following lists a summary of suggested recommendations:

(1) Revise buildings code Manning's "n" value.

(2) Define 2d_fcsh level data at points rather than on lines for modelling fences.

(3) Consider a design flood envelope of flood levels which includes both "with" and "without" fence scenarios.

(4) Resolve connectivity and negative slope issues in pipe layers.

(5) Consider extending the Prospect Creek boundary west of Rosford Street Reserve embankment.

(6) Demonstrate no sensitivity to auto HQ boundary slope assumptions.

(7) Initialise model to resolve mass balance errors early in the model simulation.

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APPENDIX H

EXTREME RAINFALL METHODOLOGY

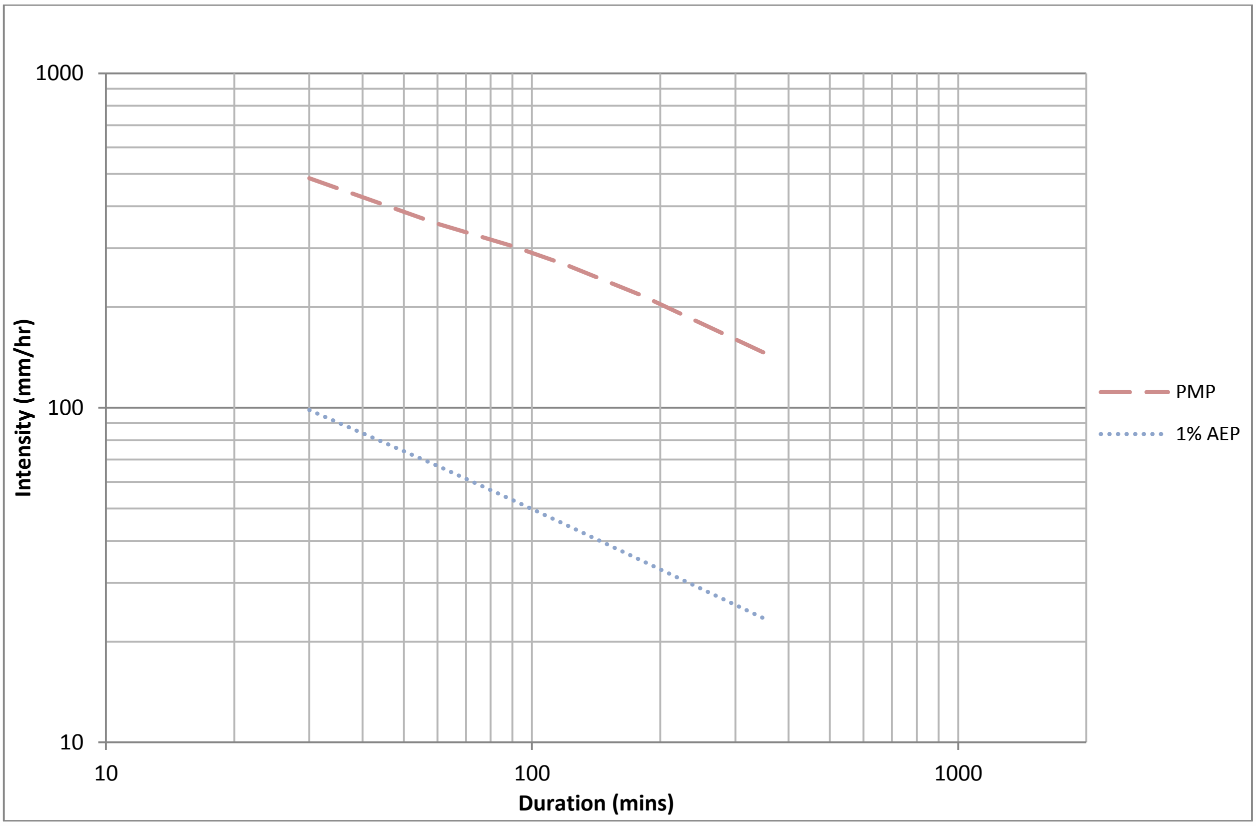
Estimation of 1 in 10,000 Year Rainfall

Overview

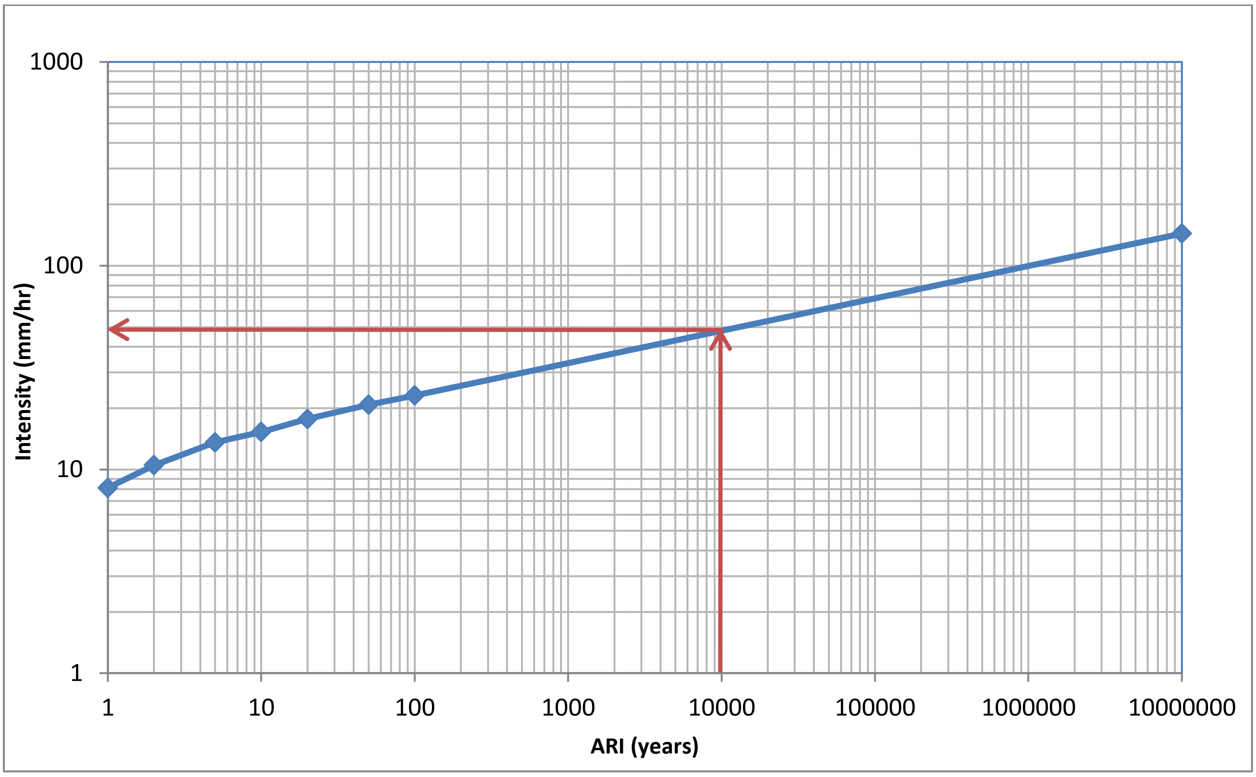
The 1 in 10,000 year rainfall was estimated as part of the Smithfield West Flood Study. The calculations were completed in accordance with procedures set out in 'Australian Rainfall & Runoff- A Guideline to Flood Estimation' (Engineers Australia, 1998) for extreme rainfall. A summary of the calculation technique is provided below.

Calculations

The 1% AEP rainfall intensities were plotted on a chart for a range of different storm durations. The Probable Maximum Precipitation intensities were also included on the chart. A nominal ARI of 10,000,000 years was adopted for the PMP in accordance with Chapter 8 of the Bureau of Meteorology's Generalised Short Duration Method (GSDM) for catchments with areas of less than 100 km^2 (Bureau of Meteorology, 2003). The resulting chart is provided below.



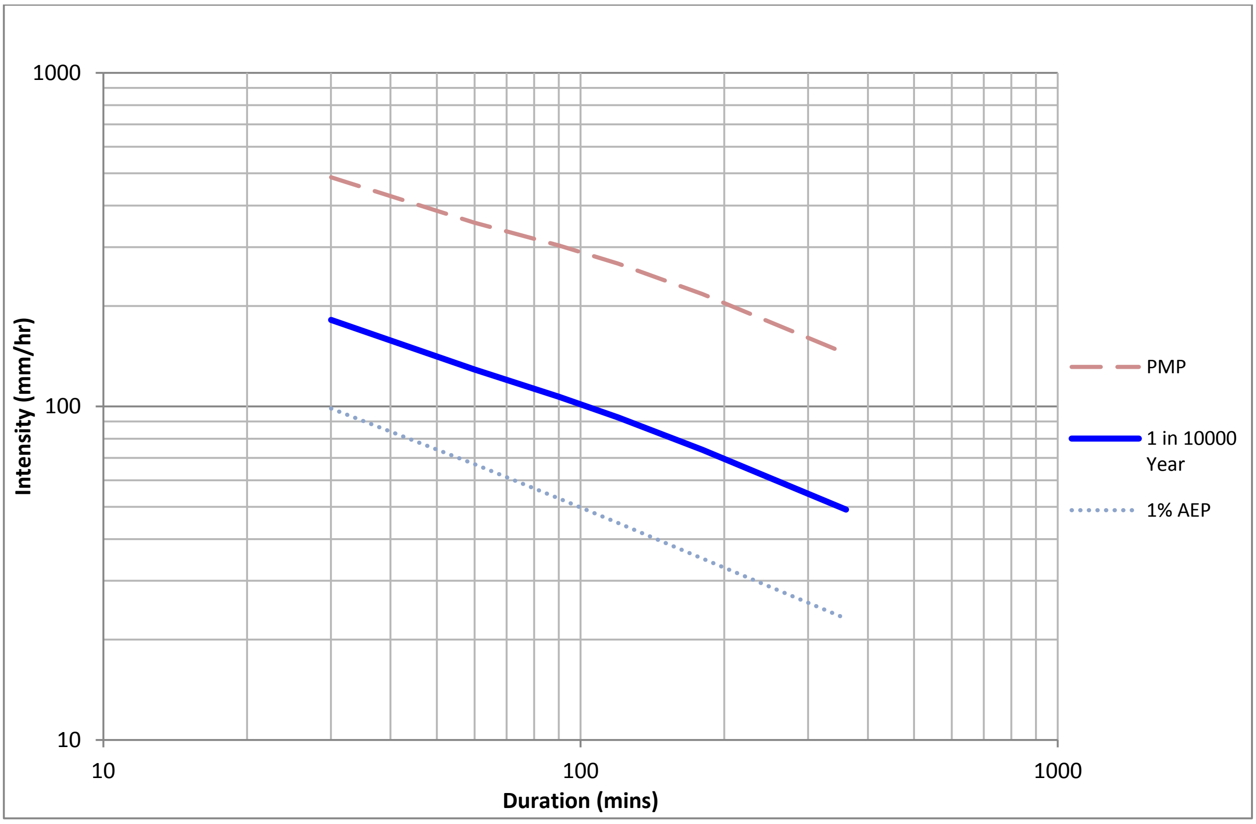
The 6 hour rainfall intensities were extracted from the above charts and were plotted against ARI. The resulting chart is presented below (note: log scales are applied to both X and Y axis).



6 hour rainfall intensities for the 1 in 10 000 year event was extracted from the above chart. This produced the following 6 hour intensity values:

- 1 in 10 000 year, 6 hour intensity = 49 mm/hr

The 1 in 10,000 year, 6 hour rainfall intensity was included on the original IFD chart and a line was drawn from this point parallel to the 1% AEP and PMF IFD lines (refer blue line in chart below). This line represents IFD curve for the 1 in 10,000 year storm.



The 1 in 10,000 year intensities were subsequently extracted from the chart for a range of durations:

Storm Duration	**1 in 10,000 Year Intensity** (mm/hr)
15 mins	255
30 mins	182
1 hour	129
2 hours	93
3 hours	74
6 hours	49

GSDM CALCULATION SHEET

LOCATION INFORMATION

Catchment Smithfield West Area 1.44 km^2

State New South Wales Duration Limit 6.0 hrs

Latitude 33.8521^{0}S Longitude 150.9275^{0}E

Portion of Area Considered:

Smooth, **S** = 0.00 (0.0 - 1.0) Rough, **R** = 1.00 (0.0 - 1.0)

ELEVATION ADJUSTMENT FACTOR (EAF)

Mean Elevation 35 m

Adjustment for Elevation (-0.05 per 300m above 1500m) 0.00

EAF = **1.00** (0.85 – 1.00)

MOISTURE ADJUSTMENT FACTOR (MAF)

MAF = **0.70** (0.40-1.00)

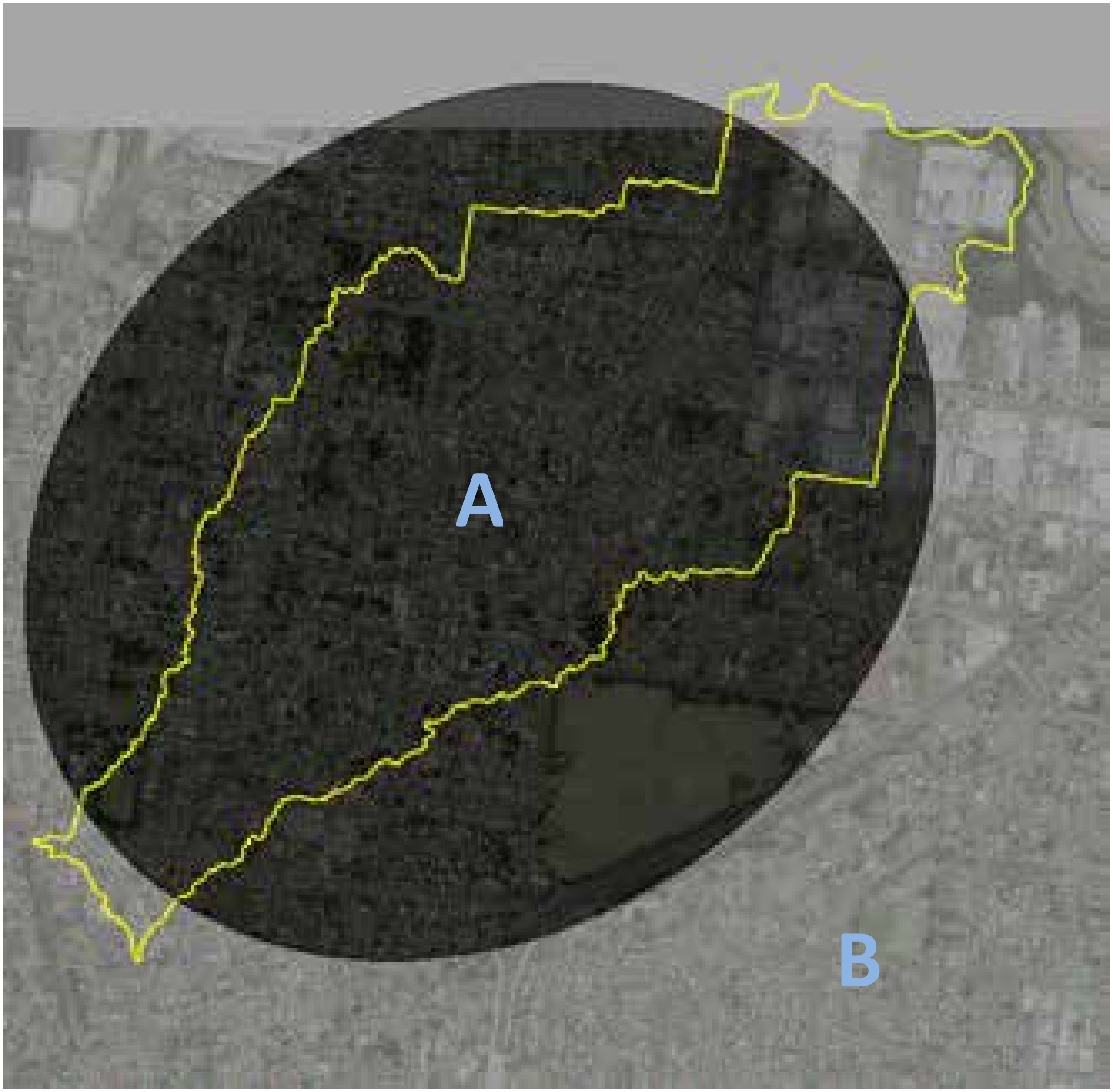
PMP VALUES (mm)

Duration (hours)	Initial Depth -Smooth (D_S)	Initial Depth -Rough (D_R)	PMP Estimate = (D_SxS + D_RxR) x MAF x EAF	Rounded PMP Estimate (nearest 10 mm)
0.25	241	241	168	170
0.50	346	346	242	240
0.75	435	435	305	300
1.00	505	505	353	350
1.50	575	650	455	450
2.00	641	761	533	530
2.50	683	842	589	590
3.00	720	926	648	650
4.00	784	1053	737	740
5.00	848	1165	815	820
6.00	894	1229	860	860

Prepared By D. Fedczyna Date 08/07/2014

Checked By D. Tetley Date 06/05/2015

GSDM SPATIAL DISTRIBUTION



GSDM SPATIAL DISTRIBUTION

DURATION = 0.25 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse (mm.km^2)	Rainfall Volume between Ellipses (mm.km^2)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	242	169	219	219	169
B	0.15	1.44	241	168	243	24	159
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

DURATION = 0.50 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse (mm.km^2)	Rainfall Volume between Ellipses (mm.km^2)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	347	243	315	315	243
B	0.15	1.44	346	242	349	35	233
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

GSDM SPATIAL DISTRIBUTION (continued)

DURATION = 0.75 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse ($mm.km^2$)	Rainfall Volume between Ellipses ($mm.km^2$)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	437	306	396	396	306
B	0.15	1.44	435	305	440	44	295
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

DURATION = 1.0 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse ($mm.km^2$)	Rainfall Volume between Ellipses ($mm.km^2$)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	506	355	459	459	355
B	0.15	1.44	505	353	510	51	342
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

GSDM SPATIAL DISTRIBUTION (continued)

DURATION = 1.5 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse ($mm.km^2$)	Rainfall Volume between Ellipses ($mm.km^2$)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	651	456	590	590	456
B	0.15	1.44	650	455	656	66	444
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

DURATION = 2.0 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse ($mm.km^2$)	Rainfall Volume between Ellipses ($mm.km^2$)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	764	535	692	692	535
B	0.15	1.44	761	533	769	77	518
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

GSDM SPATIAL DISTRIBUTION (continued)

DURATION = 2.5 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse (mm.km^2)	Rainfall Volume between Ellipses (mm.km^2)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	845	592	766	766	592
B	0.15	1.44	842	589	851	85	571
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

DURATION = 3.0 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse (mm.km^2)	Rainfall Volume between Ellipses (mm.km^2)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	930	651	843	843	651
B	0.15	1.44	926	648	936	93	624
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

GSDM SPATIAL DISTRIBUTION (continued)

DURATION = 4.0 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse ($mm.km^2$)	Rainfall Volume between Ellipses ($mm.km^2$)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	1057	740	958	958	740
B	0.15	1.44	1053	737	1064	106	716
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

DURATION = 5.0 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse ($mm.km^2$)	Rainfall Volume between Ellipses ($mm.km^2$)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	1169	818	1060	1060	818
B	0.15	1.44	1165	815	1177	117	788
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

GSDM SPATIAL DISTRIBUTION (continued)

DURATION = 6.0 Hours							
Ellipse	Catchment Area Between Ellipse (km^2)	Catchment Area Enclosed by Ellipse (km^2)	Initial Mean Rainfall Depth (mm)	Adjusted Mean Rainfall Depth (mm)	Rainfall Volume enclosed by Ellipse (mm.km^2)	Rainfall Volume between Ellipses (mm.km^2)	Mean Rainfall Depth between ellipses (mm)
A	1.29	1.29	1233	863	1118	1118	863
B	0.15	1.44	1229	860	1242	124	834
C	N/A	N/A	N/A	N/A	N/A	N/A	N/A
D	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F	N/A	N/A	N/A	N/A	N/A	N/A	N/A
G	N/A	N/A	N/A	N/A	N/A	N/A	N/A
H	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I	N/A	N/A	N/A	N/A	N/A	N/A	N/A
J	N/A	N/A	N/A	N/A	N/A	N/A	N/A

APPENDIX I

HYDROLOGIC VERIFICATION – XP-RAFTS

XP-RAFTS Validation Model

General

The XP-RAFTS software was used to develop a hydrologic computer model of the Smithfield West study area to assist with the validation of the TUFLOW computer model. XP-RAFTS is a lumped hydrologic software product that is developed by XP Software (XP Software, 2009) and is used extensively across Australia for simulating rainfall-runoff processes and producing design discharge estimates. The following sections provide a summary of the model development process and the outcomes of the model validation.

Hydrologic Model Development

Subcatchment Parameterisation

The Smithfield West catchment was subdivided into 49 subcatchments based on the alignment of major flow paths and topographic divides. The subcatchments were delineated with the assistance of the CatchmentSIM software (Catchment Simulation Solutions, 2011) using a 1 metre Digital Elevation Model (DEM). The subcatchment layout is presented in **Figure I**.

The majority of the Smithfield West catchment incorporates significant urban areas that are relatively impervious. Urbanisation effectively separates the catchment into two hydrologic systems, i.e.,:

- rapid rainfall response and low infiltration potential across impervious areas; and,
- slower rainfall response and high infiltration potential across pervious areas.

In recognition of the differing characteristics of the two hydrologic systems, each XP-RAFTS subcatchment was subdivided into two sub-areas. The first sub-area was used to represent the pervious sections of the subcatchment and the second sub-area was used to represent the impervious sections of the subcatchment. The division of each subcatchment into pervious and impervious sub-areas allows different rainfall losses and roughness coefficients to be specified, thereby providing a more realistic representation of rainfall-runoff processes from the two different hydrologic systems.

Key hydrologic properties including area and average vectored slope were calculated automatically for each subcatchment using CatchmentSIM. The adopted subcatchment slopes and areas are provided in **Table I1**.

The catchment was also subdivided into different land use types based on the remote sensing outputs that were used for assigning material types in the TUFLOW model. Percentage impervious and Manning's 'n' values were assigned to each land use and are summarised in **Table I2**. The percentage impervious and Manning's 'n' values were subsequently used to calculate weighted average percentage impervious and 'n' values for each subcatchment.
The adopted pervious and impervious areas and weighted 'n' values for each subcatchment are also provided in **Table I1**.

Table I1 - XP-RAFTS INPUT PARAMETERS

Subcatchment ID	Sub-Area	Area [ha]	Catchment Slope [%]	Percentage Impervious [%]	Mannings 'n'
1	1	0.23	3.25	0	0.044
	2	0.10	3.25	100	0.015
2	1	2.05	2.32	0	0.041
	2	0.71	2.32	100	0.015
3	1	0.36	0.89	0	0.026
	2	2.06	0.89	100	0.015
4	1	1.01	2.21	0	0.033
	2	0.82	2.21	100	0.015
5	1	3.38	1.64	0	0.036
	2	2.02	1.64	100	0.015
6	1	1.35	1.41	0	0.030
	2	1.16	1.41	100	0.015
7	1	1.27	2.41	0	0.030
	2	1.07	2.41	100	0.015
8	1	1.18	0.25	0	0.027
	2	1.27	0.25	100	0.015
9	1	0.77	1.21	0	0.025
	2	2.57	1.21	100	0.015
10	1	0.24	1.53	0	0.023
	2	1.50	1.53	100	0.015
11	1	0.78	1.29	0	0.030
	2	0.70	1.29	100	0.015
12	1	1.14	1.39	0	0.023
	2	2.27	1.39	100	0.015
13	1	1.72	2.03	0	0.031
	2	1.66	2.03	100	0.015
14	1	2.13	2.69	0	0.036
	2	1.37	2.69	100	0.015
15	1	0.67	2.11	0	0.033
	2	0.58	2.11	100	0.015
16	1	1.69	1.35	0	0.033
	2	1.35	1.35	100	0.015
17	1	3.02	1.43	0	0.036
	2	2.08	1.43	100	0.015
18	1	1.96	1.58	0	0.035
	2	1.37	1.58	100	0.015
19	1	1.04	2.13	0	0.025
	2	1.44	2.13	100	0.015
20	1	0.65	2.56	0	0.024
	2	0.69	2.56	100	0.015
21	1	3.69	1.96	0	0.043
	2	1.05	1.96	100	0.015
22	1	1.13	2.16	0	0.030
	2	1.28	2.16	100	0.015
23	1	1.77	2.47	0	0.033
	2	1.57	2.47	100	0.015

Subcatchment ID	Sub-Area	Area [ha]	Catchment Slope [%]	Percentage Impervious [%]	Mannings 'n'
24	1	0.63	2.92	0	0.033
	2	0.56	2.92	100	0.015
25	1	2.21	1.96	0	0.035
	2	1.48	1.96	100	0.015
26	1	1.19	4.12	0	0.036
	2	0.88	4.12	100	0.015
27	1	1.56	3.89	0	0.038
	2	0.99	3.89	100	0.015
28	1	2.03	3.13	0	0.039
	2	0.37	3.13	100	0.015
29	1	1.61	3.19	0	0.041
	2	1.00	3.19	100	0.015
30	1	1.41	2.82	0	0.033
	2	1.24	2.82	100	0.015
31	1	0.04	1.39	0	0.031
	2	0.07	1.39	100	0.015
32	1	0.65	2.72	0	0.029
	2	0.86	2.72	100	0.015
33	1	3.09	2.42	0	0.036
	2	2.17	2.42	100	0.015
34	1	1.61	1.97	0	0.037
	2	1.16	1.97	100	0.015
35	1	1.69	3.00	0	0.032
	2	1.40	3.00	100	0.015
36	1	0.47	3.26	0	0.030
	2	0.81	3.26	100	0.015
37	1	1.71	3.44	0	0.034
	2	1.51	3.44	100	0.015
38	1	2.11	2.88	0	0.033
	2	1.62	2.88	100	0.015
39	1	2.06	1.98	0	0.038
	2	1.35	1.98	100	0.015
40	1	1.11	4.05	0	0.042
	2	0.61	4.05	100	0.015
41	1	1.75	2.63	0	0.034
	2	1.42	2.63	100	0.015
42	1	0.99	3.57	0	0.034
	2	0.77	3.57	100	0.015
43	1	1.14	2.89	0	0.034
	2	0.89	2.89	100	0.015
44	1	2.82	2.57	0	0.033
	2	2.03	2.57	100	0.015
45	1	2.17	3.27	0	0.033
	2	1.81	3.27	100	0.015
46	1	1.64	3.40	0	0.036
	2	0.91	3.40	100	0.015
47	1	3.72	2.68	0	0.037
	2	1.98	2.68	100	0.015

Subcatchment ID	Sub-Area	Area [ha]	Catchment Slope [%]	Percentage Impervious [%]	Mannings 'n'
48	1	1.70	3.07	0	0.031
	2	1.89	3.07	100	0.015
49	1	2.39	2.27	0	0.028
	2	3.35	2.27	100	0.015
VictoriaRd	1	0.00	0.00	0	0.025
	2	0.00	0.00	0	0.025

Table I2 Adopted Impervious Percentage and Manning's 'n' Values for Hydrologic Model

Land Use Description	Manning's 'n'	Impervious (%)
Roads	0.020	100
Concrete	0.020	100
Buildings	0.100	100
Water	0.013	100
Grass	0.030	0
Trees	0.160	0
Shrubs	0.077	0
Crops	0.059	0

Stream Routing

In addition to local subcatchment runoff, most subcatchments will also carry flow from upstream catchments along the main flow path. The flow along these flowpaths in XP-RAFTS is represented using a "link" between successive subcatchment "nodes".

For this study, time delay lag routing was employed to represent the routing of runoff along the main watercourses into downstream subcatchments. The time delay value for each subcatchment was calculated using a modified version of the Bransby-Williams formula (Queensland Government, 2007).

Rainfall Loss Model

During a typical rainfall event, not all of the rain falling on a catchment is converted to runoff. Some of the rainfall may be intercepted and stored by vegetation, some may be stored in small depressions and some may infiltrate into the underlying soils.

To account for rainfall "losses" of this nature, the hydrologic model incorporates a rainfall loss model. For this study, the "Initial-Continuing" loss model was adopted, which is recommended in *"Australian Rainfall and Runoff – A Guide to Flood Estimation"* (Engineers Australia, 1987) for eastern NSW.

This loss model assumes that a specified amount of rainfall is lost during the initial saturation/wetting of the catchment (referred to as the 'Initial Loss'). Further losses are applied at a constant rate to simulate infiltration/interception once the catchment is saturated (referred to as the 'Continuing Loss Rate'). The initial and continuing losses are deducted from the total rainfall over the catchment, leaving the residual rainfall to be distributed across the catchment as runoff.

Initial and continuing losses were applied to each material type based on standard design values documented in *'Australian Rainfall and Runoff – A Guide to Flood Estimation' (Engineers Australia, 1987)* and are summarised in **Table I3**. All rainfall losses are consistent with those adopted in the TUFLOW model.

Table I3 Adopted XP-RAFTS Rainfall Loss Values

Material Description	Rainfall Losses	
	Initial Loss *(mm)*	Continuing Loss Rate *(mm/hr)*
Roads	1.0	0.0
Concrete	1.0	0.0
Buildings	1.0	0.0
Grass	10.0	2.5
Trees	10.0	2.5
Shrubs	10.0	2.5
Crops	10.0	2.5

Results

The XP-RAFTS hydrologic models were then used to simulate the 1% AEP storm for a range of design storm durations. Peak 1% AEP discharges were extracted from the model and compared to the TUFLOW hydraulic model at common locations. A summary of the flow comparison results is provided in **Table I4**, and complete results for all subcatchments in the Smithfield West Study Area is contained in **Table I5**.

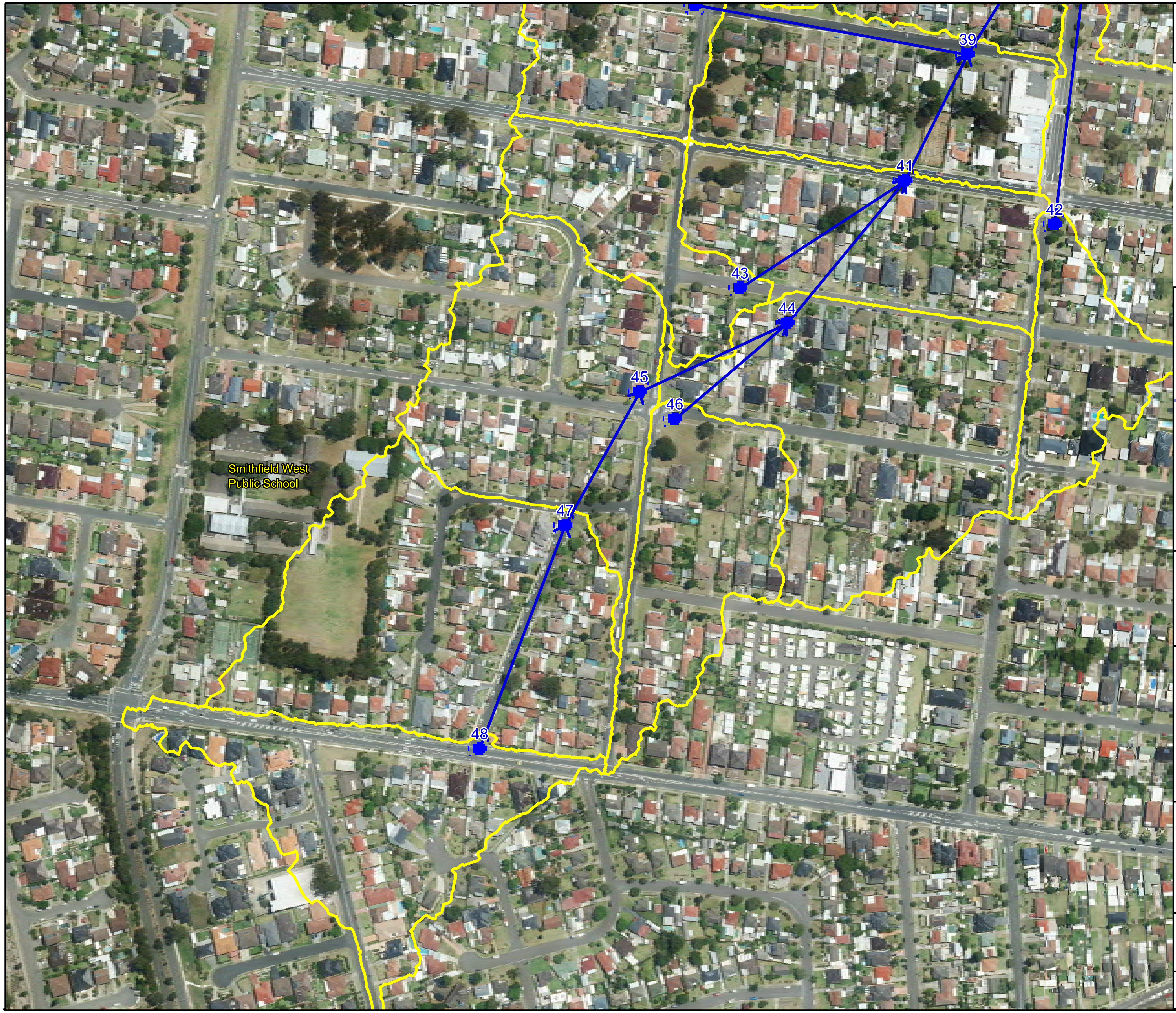
Table I4 Comparison between XP-RAFTS and TUFLOW 1%AEP peak discharges in the Smithfield West Study Area

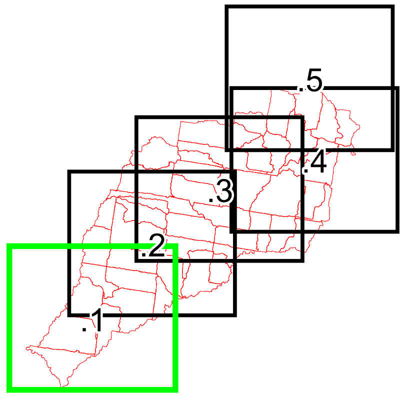
XP-RAFTS Subcatchment	Peak 1% AEP Flow (m^3/s)		
	XP-RAFTS	TUFLOW	Difference
47	3.0	3.2	0.2
41	7.0	7.2	0.2
31	12.9	12.9	0.0
22	14.4	14.8	0.4
Victoria St	18.3	18.9	0.6
4	19.6	19.2	-0.4

The comparison provided in **Tables I4** shows the TUFLOW model produces peak flows are that are typically within 3% of the XP-RAFTS model, with the biggest discrepancy being 7%. This is considered to be a reasonable level of agreement and indicates that the TUFLOW model is providing a reasonable representation of hydrologic processes across the Smithfield West Study Area.

Table I5 - PEAK DESIGN FLOOD DISCHARGES - 100 Year ARI

Subcatchment ID	Peak Discharge (m^3/s)						
	30 min	60 min	90 min	120 min	180 min	270 min	360 min
1	7.80	8.59	8.59	**8.88**	6.75	6.72	5.64
2	15.07	18.78	19.58	**19.91**	17.67	16.73	15.74
3	1.06	1.07	**1.17**	1.13	0.65	0.58	0.43
4	15.02	18.65	19.31	**19.62**	17.37	16.55	15.49
5	1.27	1.36	**1.60**	1.42	1.11	1.04	0.90
6	14.73	18.09	18.48	**18.85**	16.38	15.96	14.61
7	0.76	0.82	**0.93**	0.83	0.60	0.52	0.41
8	1.36	1.41	**1.51**	1.43	0.98	0.96	0.79
9	1.85	1.81	**2.01**	1.93	1.43	1.26	1.00
10	1.61	1.79	**2.34**	1.81	1.33	1.26	1.09
11	6.15	6.92	6.91	**7.38**	5.58	5.41	4.57
12	4.63	5.10	5.31	**5.33**	4.13	4.03	3.34
13	14.67	17.90	18.26	**18.62**	16.04	15.77	14.31
14	0.96	1.08	**1.27**	1.11	0.86	0.74	0.61
15	0.42	0.45	**0.50**	0.46	0.33	0.28	0.22
16	14.19	16.92	17.00	**17.38**	14.70	14.77	13.09
17	1.26	1.32	**1.53**	1.37	1.03	0.97	0.84
18	14.00	16.57	16.57	**16.93**	14.25	14.42	12.70
19	3.25	3.53	**3.77**	3.75	2.89	2.68	2.22
20	0.78	0.93	**1.02**	1.01	0.80	0.74	0.63
21	1.54	1.87	**2.08**	2.06	1.57	1.55	1.35
22	12.41	14.19	14.10	**14.39**	11.90	12.19	10.57
23	1.03	1.13	**1.31**	1.16	0.84	0.73	0.59
24	0.43	0.46	**0.50**	0.46	0.32	0.28	0.21
25	12.30	13.92	13.81	**14.07**	11.61	11.96	10.25
26	0.68	0.76	**0.85**	0.75	0.55	0.47	0.37
27	0.77	0.87	**1.00**	0.87	0.66	0.57	0.45
28	0.52	0.61	**0.73**	0.72	0.57	0.48	0.41
29	0.73	0.80	**0.94**	0.82	0.64	0.56	0.45
30	0.86	0.94	**1.07**	0.95	0.68	0.59	0.47
31	11.42	12.69	12.55	**12.91**	10.50	10.87	9.11
32	0.59	0.62	**0.67**	0.62	0.41	0.36	0.27
33	8.06	9.26	9.33	**9.71**	7.59	7.67	6.34
34	2.39	2.79	**3.05**	3.03	2.19	1.99	1.64
35	1.33	1.43	1.64	**1.66**	1.23	1.06	0.85
36	0.54	0.56	**0.60**	0.56	0.35	0.31	0.23
37	1.57	1.76	**1.98**	1.80	1.24	1.08	0.84
38	1.48	1.66	**1.91**	1.76	1.37	1.18	0.95
39	7.48	8.38	8.58	**8.86**	6.90	6.71	5.63
40	0.52	0.57	**0.65**	0.57	0.44	0.38	0.30
41	5.90	6.60	**7.05**	6.98	5.54	5.19	4.33
42	0.61	0.65	**0.72**	0.65	0.47	0.41	0.31
43	0.65	0.71	**0.80**	0.71	0.53	0.46	0.36
44	4.81	5.32	**5.96**	**5.96**	4.51	4.19	3.53
45	3.23	3.54	3.94	**4.02**	3.04	2.71	2.28
46	0.73	0.83	**0.97**	0.85	0.65	0.56	0.45
47	2.20	2.59	2.70	**2.99**	2.17	1.92	1.61
48	1.25	1.36	**1.52**	1.38	0.94	0.82	0.64
49	2.74	2.89	**3.25**	3.22	2.53	2.20	1.82
Victoria Rd	14.67	17.90	18.26	**18.62**	16.04	15.77	14.31





FairfieldCity
Celebrating diversity

LEGEND

26
XP - RAFTS
Subcatchment Node

XP - RAFTS
Routing Link

XP - RAFTS
Subcatchment Boundaries

Notes:

Aerial Photograph Date: 2014

N
W
E
S

Scale 1:3000 (at A3)

0 75 150

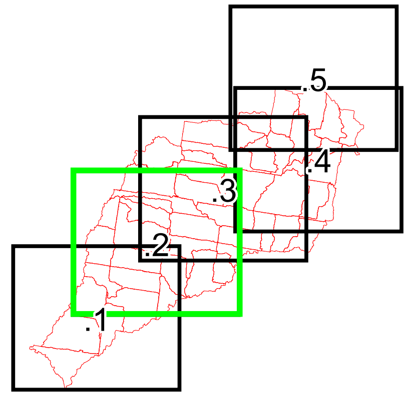
m

Figure I.1:
Smithfield West
XP - RAFTS
Subcatchment Layout

Prepared By:
Catchment Simulation Solutions
Suite 2.01 210 George St.
Sydney, NSW 2000

File Name: Figure I.1 XP - RAFTS Subcatchments.wor

FairfieldCity
Celebrating diversity



LEGEND

26 — XP - RAFTS Subcatchment Node

XP - RAFTS Routing Link

XP - RAFTS Subcatchment Boundaries

Notes:

Aerial Photograph Date: 2014

N
W E
S

Scale 1:3000 (at A3)

0 75 150 m

Figure I.2: Smithfield West XP - RAFTS Subcatchment Layout

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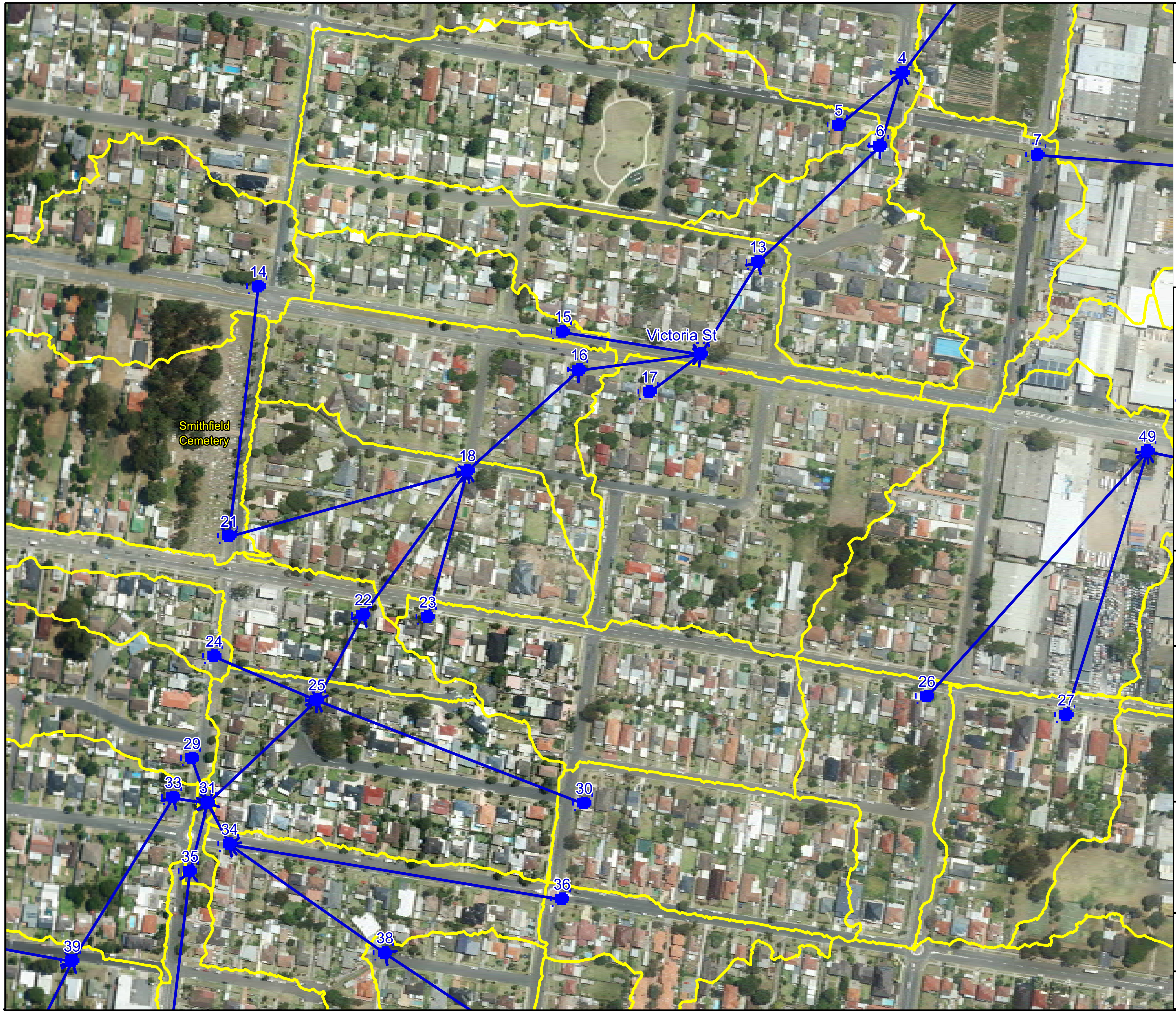
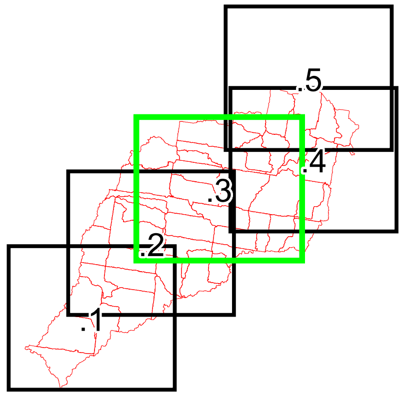
Sydney, NSW 2000

File Name: Figure I.2 XP - RAFTS Subcatchments.wor



FairfieldCity
Celebrating diversity



LEGEND

26 XP - RAFTS Subcatchment Node

XP - RAFTS Routing Link

XP - RAFTS Subcatchment Boundaries

Notes:

Aerial Photograph Date: 2014

N
W
E
S

Scale 1:3000 (at A3)

0 75 150 m

Figure I.3: Smithfield West XP - RAFTS Subcatchment Layout

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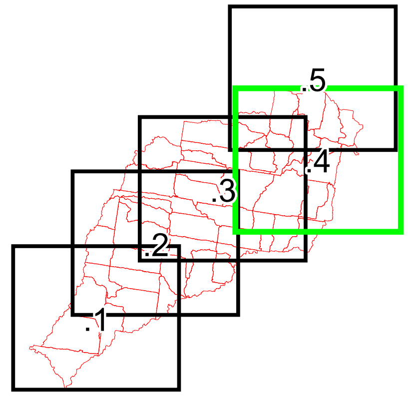
Sydney, NSW 2000

File Name: Figure I.3 XP - RAFTS Subcatchments.wor

FairfieldCity
Celebrating diversity



LEGEND

26 XP - RAFTS Subcatchment Node

XP - RAFTS Routing Link

XP - RAFTS Subcatchment Boundaries

Notes:

Aerial Photograph Date: 2014

N
W E
S

Scale 1:3000 (at A3)

0 75 150 m

Figure I.4: Smithfield West XP - RAFTS Subcatchment Layout

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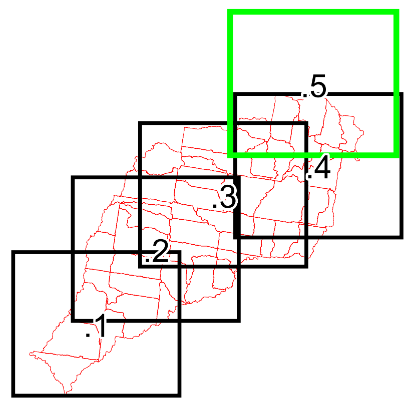
Sydney, NSW 2000

File Name: Figure I.4 XP - RAFTS Subcatchments.wor

Fairfield City
Celebrating diversity



LEGEND

26 XP - RAFTS Subcatchment Node

XP - RAFTS Routing Link

XP - RAFTS Subcatchment Boundaries

Notes:

Aerial Photograph Date: 2014

N
W E
S

Scale 1:3000 (at A3)

0 75 150 m

Figure I.5: Smithfield West XP - RAFTS Subcatchment Layout

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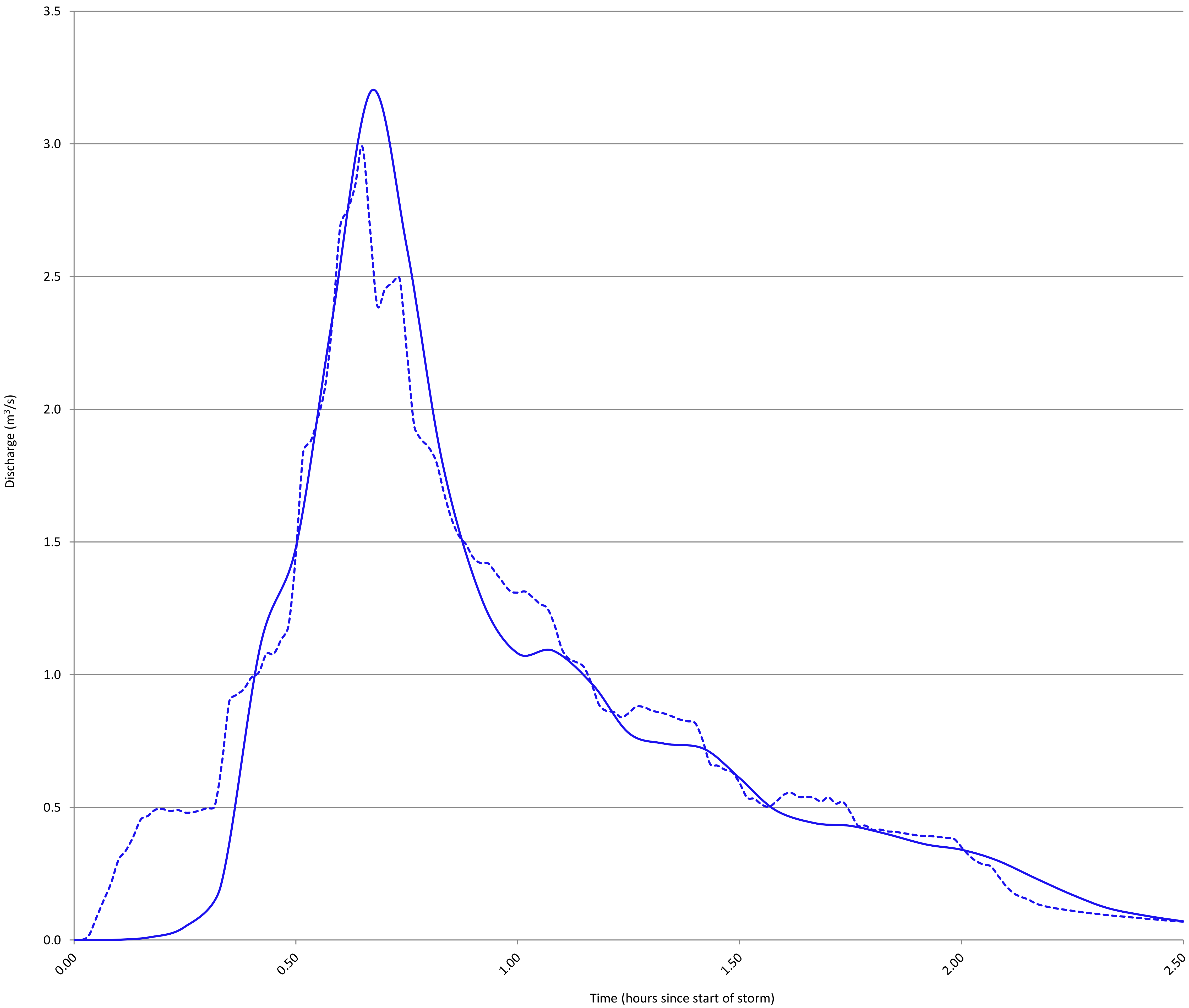
Sydney, NSW 2000

File Name: Figure I.5 XP - RAFTS Subcatchments.wor



FairfieldCity
Celebrating diversity

LEGEND:

100 Year ARI (2 hour storm) XP-RAFTS hydrograph

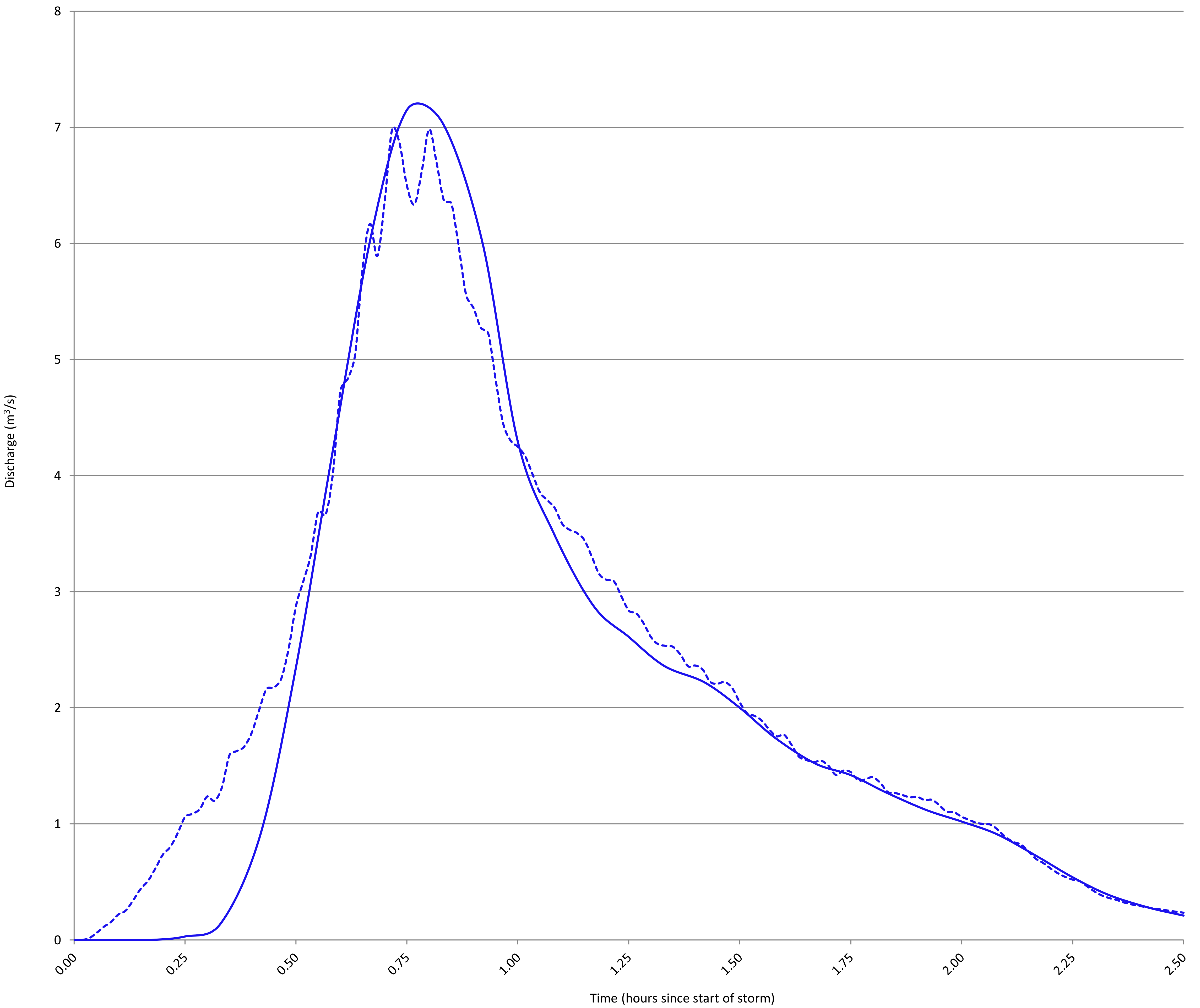
100 Year ARI (2 hour storm) TUFLOW hydrograph With Fences

Notes:
Hydrograph extracted from Smithfield West XP-RAFTS Subcatchment 47

Figure I6: Comparison Between XP-RAFTS and TUFLOW Discharge Hydrographs near Genmoore Street.

Prepared By:
Catchment Simulation Solutions
Suite 2.01, 210 George Street
Sydney, NSW, 2000

File Name: Hydrograph Comparisons.xlsx



FairfieldCity
Celebrating diversity

LEGEND:

100 Year ARI (2 hour storm) XP-RAFTS hydrograph

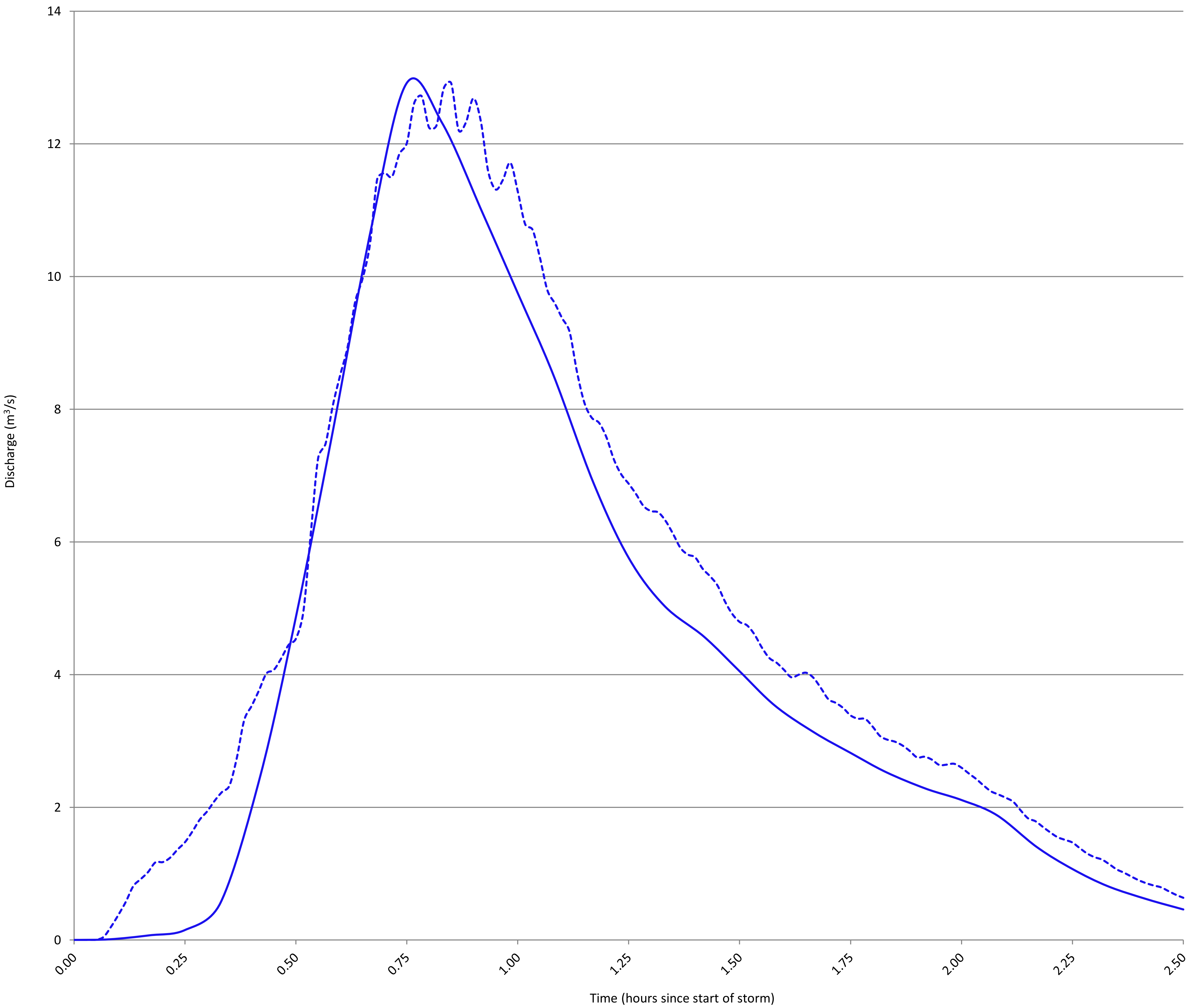
100 Year ARI (2 hour storm) TUFLOW hydrograph With Fences

Notes:
Hydrograph extracted from Smithfield West XP-RAFTS Subcatchment 41

Figure I7: Comparison Between XP-RAFTS and TUFLOW Discharge Hydrographs near Brenan Street.

Prepared By:
Catchment Simulation Solutions
Suite 2.01, 210 George Street
Sydney, NSW, 2000

File Name: Hydrograph Comparisons.xlsx



FairfieldCity
Celebrating diversity

LEGEND:

- - - - 100 Year ARI (2 hour storm) XP-RAFTS hydrograph

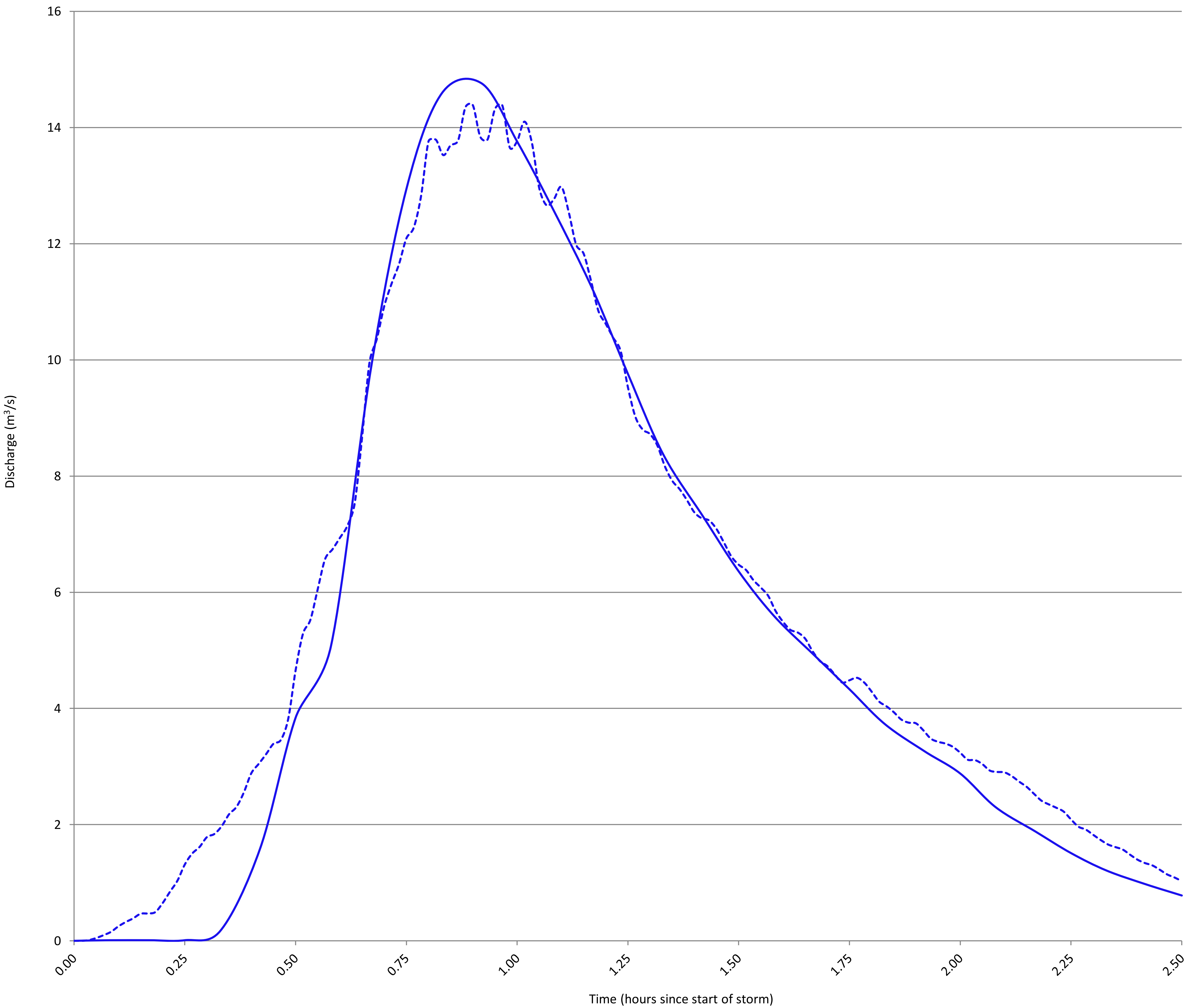
—— 100 Year ARI (2 hour storm) TUFLOW hydrograph With Fences

Notes:
Hydrograph extracted from Smithfield West XP-RAFTS Subcatchment 31

Figure I8: Comparison Between XP-RAFTS and TUFLOW Discharge Hydrographs near Cnr Dublin and Neville Street.

Prepared By:
Catchment Simulation Solutions
Suite 2.01, 210 George Street
Sydney, NSW, 2000

File Name: Hydrograph Comparisons.xlsx



FairfieldCity
Celebrating diversity

LEGEND:

100 Year ARI (2 hour storm) XP-RAFTS hydrograph

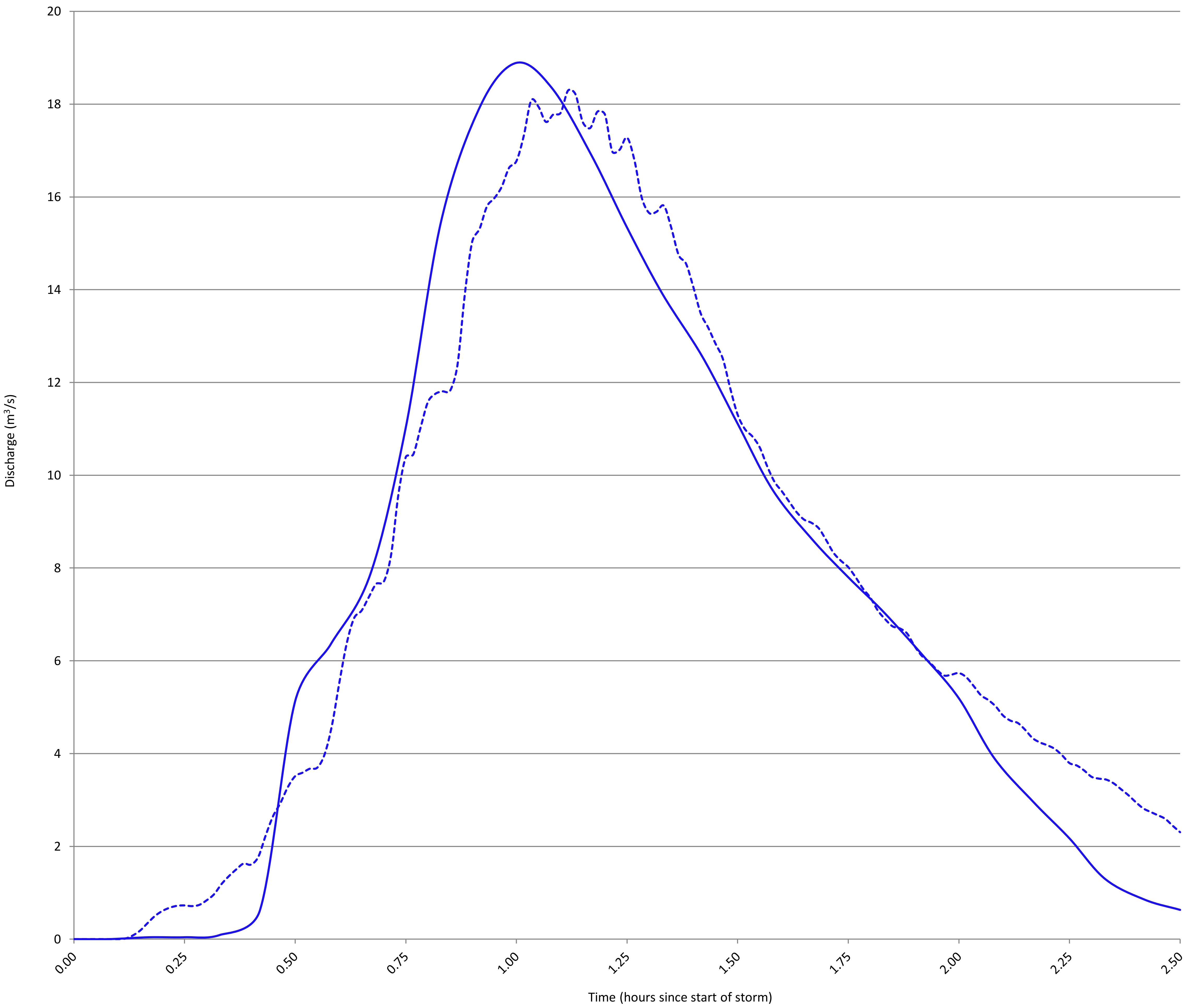
100 Year ARI (2 hour storm) TUFLOW hydrograph With Fences

Notes:
Hydrograph extracted from Smithfield West XP-RAFTS Subcatchment 22

Figure I9: Comparison Between XP-RAFTS and TUFLOW Discharge Hydrographs near The Horsley Drive.

Prepared By:
Catchment Simulation Solutions
Suite 2.01, 210 George Street
Sydney, NSW, 2000

File Name: Hydrograph Comparisons.xlsx



Fairfield City
Celebrating diversity

LEGEND:

100 Year ARI (2 hour storm) XP-RAFTS hydrograph

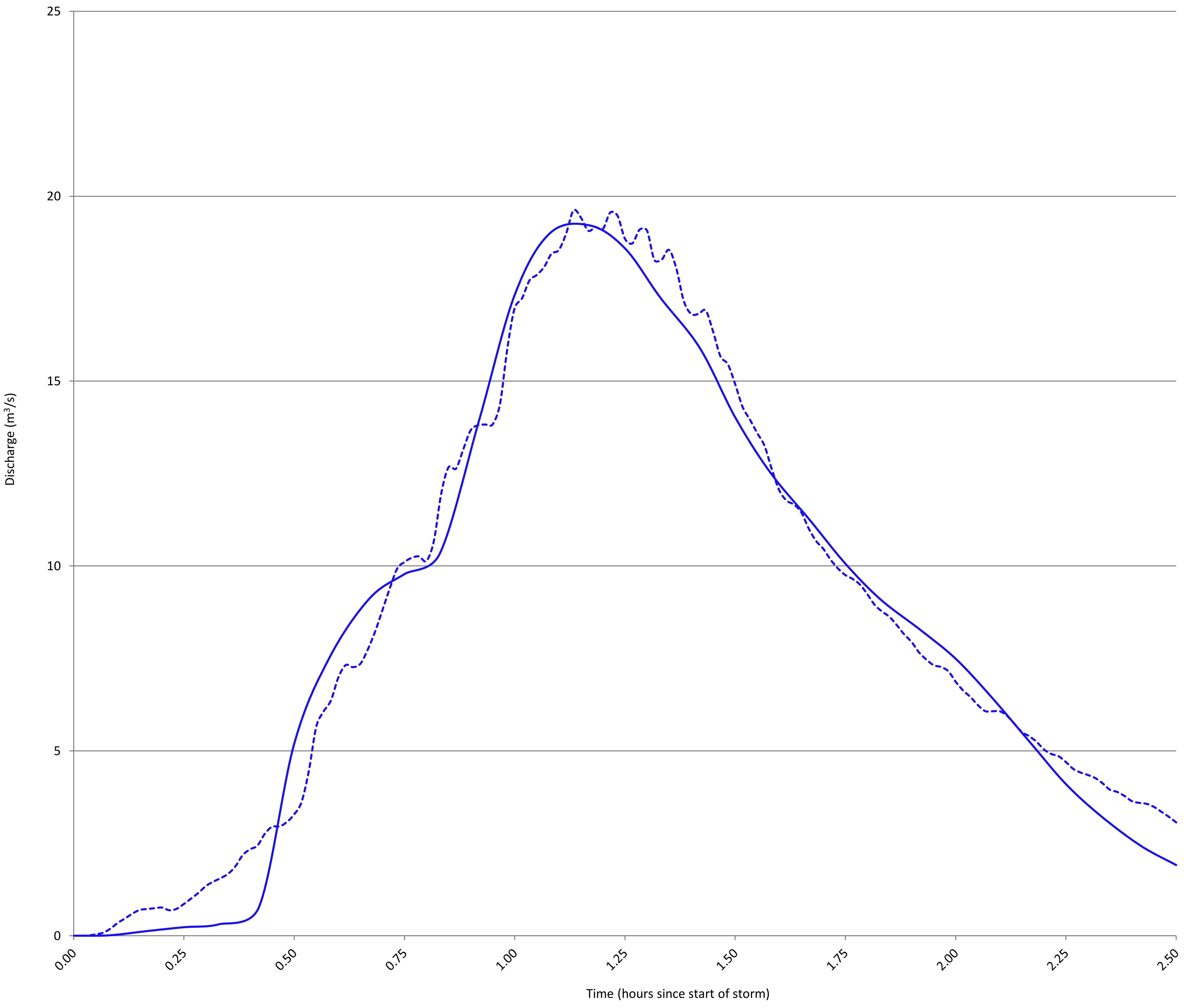
100 Year ARI (2 hour storm) TUFLOW hydrograph With Fences

Notes:
Hydrograph extracted from Smithfield West XP-RAFTS Subcatchment Victoria St

Figure I10: Comparison Between XP-RAFTS and TUFLOW Discharge Hydrographs near Victoria Street.

Prepared By:
Catchment Simulation Solutions
Suite 2.01, 210 George Street
Sydney, NSW, 2000

File Name: Hydrograph Comparisons.xlsx



LEGEND:

- 100 Year ARI (2 hour storm) XP-RAFTS hydrograph (dashed line)
- 100 Year ARI (2 hour storm) TUFLOW hydrograph With Fences (solid line)

Notes:
Hydrograph extracted from Smithfield West XP-RAFTS Subcatchment 4

Figure I11: Comparison Between XP-RAFTS and TUFLOW Discharge Hydrographs near Cnr Chifley and Rhondda St.

Prepared By:
Catchment Simulation Solutions
Suite 2.01, 210 George Street
Sydney, NSW, 2000

File Name: Hydrograph Comparisons.xlsx

APPENDIX J

FLOOD DAMAGE CALCULATIONS

ABOVE FLOOR FLOODING OUTPUTS

Table J1 - Buildings Subject to Above Floor Flooding (All Categories)								
	50% AEP	20% AEP	10% AEP	5% AEP	1% AEP	0.2% AEP	1 in 10000yr	PMF
BRAMLEY ST	0	0	0	0	0	0	0	0
POLDING ST	0	0	0	0	0	0	0	0
GEMOORE ST	0	0	0	0	0	0	0	2
DUNKLEY ST	0	0	0	0	0	0	0	0
GIPPS ST	0	0	0	0	0	1	3	5
ROSE ST	0	0	0	0	0	0	0	0
CHARLES ST	0	0	0	0	0	0	0	1
BEAUMONT ST	0	0	0	0	0	0	0	0
BROWN ST	0	0	0	0	1	3	4	10
BRENAN ST	0	0	0	0	1	1	2	6
JANE ST	0	0	0	0	1	4	6	10
LINDSAY AVE	0	0	0	0	0	0	0	2
NEVILLE ST	0	0	0	0	0	1	3	12
DUBLIN ST	0	0	0	0	0	1	3	7
GRADY GARDENS	0	0	0	0	0	0	0	0
CASANDA AVE	0	0	0	1	1	1	1	2
CARTELA CRES	0	0	0	1	1	2	3	8
CANARA PL	0	0	0	0	1	1	1	10
BOURKE ST	0	0	0	0	0	0	0	2
ROWLEY ST	0	0	0	0	0	0	0	0
THE HORSLEY DR	0	0	0	0	0	0	0	16
GALTON ST	0	0	0	0	0	0	0	0
MOIR ST	0	0	0	0	2	4	7	15
HART ST	0	0	1	1	4	5	7	8
VICTORIA ST	0	0	0	1	7	10	13	18
SHAMROCK ST	0	0	0	0	0	0	3	5
KINGSFORD ST	0	0	0	0	0	0	2	5
HINKLER ST	0	0	0	1	2	2	6	13
CHIFLEY ST	0	0	0	0	0	3	11	25
MEGAN AVE	0	0	0	0	0	0	0	1
NUNDLE ST	0	0	0	0	0	0	0	0
RHONDDA ST	0	0	0	0	0	0	0	3
MARKET ST	0	0	0	0	1	2	2	5
O'CONNELL ST	0	0	0	0	0	1	1	1
Total	**0**	**0**	**1**	**5**	**22**	**42**	**78**	**192**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)

NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J2 - Buildings Subject to Above Floor Flooding (Residential)

	50% AEP	20% AEP	10% AEP	5% AEP	1% AEP	0.2% AEP	1 in 10000yr	PMF
BRAMLEY ST	0	0	0	0	0	0	0	0
POLDING ST	0	0	0	0	0	0	0	0
GEMOORE ST	0	0	0	0	0	0	0	2
DUNKLEY ST	0	0	0	0	0	0	0	0
GIPPS ST	0	0	0	0	0	1	3	5
ROSE ST	0	0	0	0	0	0	0	0
CHARLES ST	0	0	0	0	0	0	0	1
BEAUMONT ST	0	0	0	0	0	0	0	0
BROWN ST	0	0	0	0	1	3	4	10
BRENAN ST	0	0	0	0	1	1	2	6
JANE ST	0	0	0	0	1	4	6	10
LINDSAY AVE	0	0	0	0	0	0	0	2
NEVILLE ST	0	0	0	0	0	1	3	12
DUBLIN ST	0	0	0	0	0	1	3	7
GRADY GARDENS	0	0	0	0	0	0	0	0
CASANDA AVE	0	0	0	1	1	1	1	2
CARTELA CRES	0	0	0	1	1	2	3	8
CANARA PL	0	0	0	0	1	1	1	10
BOURKE ST	0	0	0	0	0	0	0	2
ROWLEY ST	0	0	0	0	0	0	0	0
THE HORSLEY DR	0	0	0	0	0	0	0	16
GALTON ST	0	0	0	0	0	0	0	0
MOIR ST	0	0	0	0	2	4	7	15
HART ST	0	0	1	1	4	5	7	8
VICTORIA ST	0	0	0	1	7	10	13	18
SHAMROCK ST	0	0	0	0	0	0	3	5
KINGSFORD ST	0	0	0	0	0	0	2	5
HINKLER ST	0	0	0	1	2	2	6	13
CHIFLEY ST	0	0	0	0	0	3	9	18
MEGAN AVE	0	0	0	0	0	0	0	1
NUNDLE ST	0	0	0	0	0	0	0	0
RHONDDA ST	0	0	0	0	0	0	0	3
MARKET ST	0	0	0	0	0	0	0	0
O'CONNELL ST	0	0	0	0	0	0	0	0
Total	**0**	**0**	**1**	**5**	**21**	**39**	**73**	**179**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)
NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J3 - Buildings Subject to Above Floor Flooding (Industrial)								
	50% AEP	**20% AEP**	**10% AEP**	**5% AEP**	**1% AEP**	**0.2% AEP**	**1 in 10000yr**	**PMF**
BRAMLEY ST	0	0	0	0	0	0	0	0
POLDING ST	0	0	0	0	0	0	0	0
GEMOORE ST	0	0	0	0	0	0	0	0
DUNKLEY ST	0	0	0	0	0	0	0	0
GIPPS ST	0	0	0	0	0	0	0	0
ROSE ST	0	0	0	0	0	0	0	0
CHARLES ST	0	0	0	0	0	0	0	0
BEAUMONT ST	0	0	0	0	0	0	0	0
BROWN ST	0	0	0	0	0	0	0	0
BRENAN ST	0	0	0	0	0	0	0	0
JANE ST	0	0	0	0	0	0	0	0
LINDSAY AVE	0	0	0	0	0	0	0	0
NEVILLE ST	0	0	0	0	0	0	0	0
DUBLIN ST	0	0	0	0	0	0	0	0
GRADY GARDENS	0	0	0	0	0	0	0	0
CASANDA AVE	0	0	0	0	0	0	0	0
CARTELA CRES	0	0	0	0	0	0	0	0
CANARA PL	0	0	0	0	0	0	0	0
BOURKE ST	0	0	0	0	0	0	0	0
ROWLEY ST	0	0	0	0	0	0	0	0
THE HORSLEY DR	0	0	0	0	0	0	0	0
GALTON ST	0	0	0	0	0	0	0	0
MOIR ST	0	0	0	0	0	0	0	0
HART ST	0	0	0	0	0	0	0	0
VICTORIA ST	0	0	0	0	0	0	0	0
SHAMROCK ST	0	0	0	0	0	0	0	0
KINGSFORD ST	0	0	0	0	0	0	0	0
HINKLER ST	0	0	0	0	0	0	0	0
CHIFLEY ST	0	0	0	0	0	0	2	7
MEGAN AVE	0	0	0	0	0	0	0	0
NUNDLE ST	0	0	0	0	0	0	0	0
RHONDDA ST	0	0	0	0	0	0	0	0
MARKET ST	0	0	0	0	1	1	1	4
O'CONNELL ST	0	0	0	0	0	1	1	1
Total	**0**	**0**	**0**	**0**	**1**	**2**	**4**	**12**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)
NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J4 - Buildings Subject to Above Floor Flooding (Commercial)								
	50% AEP	**20% AEP**	**10% AEP**	**5% AEP**	**1% AEP**	**0.2% AEP**	**1 in 10000yr**	**PMF**
BRAMLEY ST	0	0	0	0	0	0	0	0
POLDING ST	0	0	0	0	0	0	0	0
GEMOORE ST	0	0	0	0	0	0	0	0
DUNKLEY ST	0	0	0	0	0	0	0	0
GIPPS ST	0	0	0	0	0	0	0	0
ROSE ST	0	0	0	0	0	0	0	0
CHARLES ST	0	0	0	0	0	0	0	0
BEAUMONT ST	0	0	0	0	0	0	0	0
BROWN ST	0	0	0	0	0	0	0	0
BRENAN ST	0	0	0	0	0	0	0	0
JANE ST	0	0	0	0	0	0	0	0
LINDSAY AVE	0	0	0	0	0	0	0	0
NEVILLE ST	0	0	0	0	0	0	0	0
DUBLIN ST	0	0	0	0	0	0	0	0
GRADY GARDENS	0	0	0	0	0	0	0	0
CASANDA AVE	0	0	0	0	0	0	0	0
CARTELA CRES	0	0	0	0	0	0	0	0
CANARA PL	0	0	0	0	0	0	0	0
BOURKE ST	0	0	0	0	0	0	0	0
ROWLEY ST	0	0	0	0	0	0	0	0
THE HORSLEY DR	0	0	0	0	0	0	0	0
GALTON ST	0	0	0	0	0	0	0	0
MOIR ST	0	0	0	0	0	0	0	0
HART ST	0	0	0	0	0	0	0	0
VICTORIA ST	0	0	0	0	0	0	0	0
SHAMROCK ST	0	0	0	0	0	0	0	0
KINGSFORD ST	0	0	0	0	0	0	0	0
HINKLER ST	0	0	0	0	0	0	0	0
CHIFLEY ST	0	0	0	0	0	0	0	0
MEGAN AVE	0	0	0	0	0	0	0	0
NUNDLE ST	0	0	0	0	0	0	0	0
RHONDDA ST	0	0	0	0	0	0	0	0
MARKET ST	0	0	0	0	0	1	1	1
O'CONNELL ST	0	0	0	0	0	0	0	0
Total	**0**	**0**	**0**	**0**	**0**	**1**	**1**	**1**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)
NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J5 - Properties Subject to Flooding (All Categories)								
	50% AEP	20% AEP	10% AEP	5% AEP	1% AEP	0.2% AEP	1 in 10000yr	PMF
BRAMLEY ST	0	0	0	0	0	0	0	0
POLDING ST	0	0	0	0	0	0	0	1
GEMOORE ST	0	0	0	0	3	3	11	16
DUNKLEY ST	0	0	0	0	1	1	2	14
GIPPS ST	0	0	0	3	8	8	9	14
ROSE ST	0	0	0	0	0	0	0	0
CHARLES ST	0	0	0	2	3	3	3	8
BEAUMONT ST	0	0	0	0	0	0	0	0
BROWN ST	0	6	6	9	10	11	12	22
BRENAN ST	0	9	10	10	11	12	16	20
JANE ST	6	9	9	10	13	13	16	25
LINDSAY AVE	0	0	0	0	2	3	9	10
NEVILLE ST	4	5	6	7	14	16	18	23
DUBLIN ST	7	8	8	8	9	9	12	24
GRADY GARDENS	0	0	0	0	0	0	0	0
CASANDA AVE	2	2	2	2	2	3	4	9
CARTELA CRES	6	6	7	7	8	8	10	18
CANARA PL	5	7	7	10	10	12	12	14
BOURKE ST	1	2	2	2	2	2	2	7
ROWLEY ST	0	0	0	0	0	0	0	0
THE HORSLEY DR	8	14	14	17	18	18	21	33
GALTON ST	0	0	0	0	0	0	0	0
MOIR ST	8	10	11	12	12	13	15	23
HART ST	7	8	8	8	8	8	8	10
VICTORIA ST	10	17	18	18	26	26	28	64
SHAMROCK ST	0	3	3	4	6	6	7	8
KINGSFORD ST	3	4	7	8	8	8	9	12
HINKLER ST	10	14	16	16	16	16	17	29
CHIFLEY ST	6	18	19	24	31	35	37	46
MEGAN AVE	0	0	1	1	2	2	2	5
NUNDLE ST	0	0	0	2	3	3	3	5
RHONDDA ST	5	6	7	9	11	11	14	19
MARKET ST	3	8	8	8	12	14	16	19
O'CONNELL ST	0	0	0	0	2	3	3	3
Total	**91**	**156**	**169**	**197**	**251**	**267**	**316**	**501**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to No

NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J6 - Buildings Subject to Flooding (All Categories)																
	50% AEP		20% AEP		10% AEP		5% AEP		1% AEP		0.2% AEP		1 in 10000yr		PMF	
	Above Floor	<0.3m Freeboard*	Above Floor	<0.3m Freeboard*	Above Floor	<0.3m Freeboard*	Above Floor	<0.3m Freeboard*	Above Floor	<0.3m Freeboard*	Above Floor	<0.3m Freeboard*	Above Floor	<0.3m Freeboard*	Above Floor	<0.3m Freeboard*
BRAMLEY ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
POLDING ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
GEMOORE ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0
DUNKLEY ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
GIPPS ST	0	0	0	0	0	0	0	2	0	2	1	1	3	1	5	1
ROSE ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
CHARLES ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
BEAUMONT ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
BROWN ST	0	0	0	1	0	1	0	2	1	1	3	1	4	0	10	2
BRENAN ST	0	0	0	0	0	0	0	2	1	1	1	1	2	0	6	0
JANE ST	0	0	0	1	0	1	0	2	1	4	4	1	6	0	10	3
LINDSAY AVE	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	1
NEVILLE ST	0	0	0	1	0	1	0	2	0	3	1	4	3	3	12	2
DUBLIN ST	0	0	0	1	0	1	0	2	0	3	1	3	3	2	7	1
GRADY GARDENS	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
CASANDA AVE	0	0	0	0	0	1	1	0	1	0	1	0	1	1	2	0
CARTELA CRES	0	0	0	0	0	0	1	0	1	1	2	2	3	2	8	0
CANARA PL	0	0	0	0	0	0	0	1	1	0	1	0	1	6	10	2
BOURKE ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0
ROWLEY ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
THE HORSLEY DR	0	0	0	0	0	0	0	0	0	2	0	3	0	6	16	3
GALTON ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
MOIR ST	0	0	0	0	0	2	0	4	2	3	4	2	7	1	15	4
HART ST	0	2	0	3	1	3	1	4	4	1	5	1	7	0	8	0
VICTORIA ST	0	1	0	1	0	5	1	11	7	7	10	5	13	3	18	0
SHAMROCK ST	0	0	0	0	0	0	0	0	0	0	0	0	3	1	5	0
KINGSFORD ST	0	0	0	0	0	0	0	1	0	1	0	2	2	2	5	0
HINKLER ST	0	0	0	0	0	0	1	1	2	4	2	6	6	3	13	1
CHIFLEY ST	0	0	0	4	0	7	0	11	0	17	3	16	11	10	25	3
MEGAN AVE	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
NUNDLE ST	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
RHONDDA ST	0	0	0	0	0	0	0	0	0	0	0	1	0	2	3	4
MARKET ST	0	0	0	3	0	3	0	3	1	2	2	1	2	2	5	1
O'CONNELL ST	0	0	0	0	0	0	0	0	0	0	1	0	1	0	1	1
Total	**0**	**3**	**0**	**15**	**1**	**25**	**5**	**48**	**22**	**52**	**42**	**50**	**78**	**45**	**192**	**32**

* The number of properties quoted as having <0.3m freeboard does not include those already quoted as having above floor flooding

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)

NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J7 - Buildings Subject to Above Floor Flooding (All Categories)				
	1%AEP	**10% Increase in 1%AEP Rainfall Intensity**	**20% Increase in 1%AEP Rainfall Intensity**	**30% Increase in 1%AEP Rainfall Intensity**
BRAMLEY ST	0	0	0	0
POLDING ST	0	0	0	0
GEMOORE ST	0	0	0	0
DUNKLEY ST	0	0	0	0
GIPPS ST	0	0	0	1
ROSE ST	0	0	0	0
CHARLES ST	0	0	0	0
BEAUMONT ST	0	0	0	0
BROWN ST	1	1	1	1
BRENAN ST	1	1	1	1
JANE ST	1	3	4	4
LINDSAY AVE	0	0	0	0
NEVILLE ST	0	0	0	2
DUBLIN ST	0	0	1	1
GRADY GARDENS	0	0	0	0
CASANDA AVE	1	1	1	1
CARTELA CRES	1	2	2	2
CANARA PL	1	1	1	1
BOURKE ST	0	0	0	0
ROWLEY ST	0	0	0	0
THE HORSLEY DR	0	0	0	0
GALTON ST	0	0	0	0
MOIR ST	2	3	3	4
HART ST	4	5	5	5
VICTORIA ST	7	8	10	10
SHAMROCK ST	0	0	0	0
KINGSFORD ST	0	0	0	0
HINKLER ST	2	2	2	2
CHIFLEY ST	0	1	3	4
MEGAN AVE	0	0	0	0
NUNDLE ST	0	0	0	0
RHONDDA ST	0	0	0	0
MARKET ST	1	2	2	2
O'CONNELL ST	0	0	0	1
Total	**22**	**30**	**36**	**42**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)
NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J8 - Buildings Subject to Above Floor Flooding (Residential)				
	1% AEP	**10% Increase in 1%AEP Rainfall Intensity**	**20% Increase in 1%AEP Rainfall Intensity**	**30% Increase in 1%AEP Rainfall Intensity**
BRAMLEY ST	0	0	0	0
POLDING ST	0	0	0	0
GEMOORE ST	0	0	0	0
DUNKLEY ST	0	0	0	0
GIPPS ST	0	0	0	1
ROSE ST	0	0	0	0
CHARLES ST	0	0	0	0
BEAUMONT ST	0	0	0	0
BROWN ST	1	1	1	1
BRENAN ST	1	1	1	1
JANE ST	1	3	4	4
LINDSAY AVE	0	0	0	0
NEVILLE ST	0	0	0	2
DUBLIN ST	0	0	1	1
GRADY GARDENS	0	0	0	0
CASANDA AVE	1	1	1	1
CARTELA CRES	1	2	2	2
CANARA PL	1	1	1	1
BOURKE ST	0	0	0	0
ROWLEY ST	0	0	0	0
THE HORSLEY DR	0	0	0	0
GALTON ST	0	0	0	0
MOIR ST	2	3	3	4
HART ST	4	5	5	5
VICTORIA ST	7	8	10	10
SHAMROCK ST	0	0	0	0
KINGSFORD ST	0	0	0	0
HINKLER ST	2	2	2	2
CHIFLEY ST	0	1	3	4
MEGAN AVE	0	0	0	0
NUNDLE ST	0	0	0	0
RHONDDA ST	0	0	0	0
MARKET ST	0	0	0	0
O'CONNELL ST	0	0	0	0
Total	**21**	**28**	**34**	**39**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)
NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J9 - Buildings Subject to Above Floor Flooding (Industrial)				
	1% AEP	10% Increase in 1%AEP Rainfall Intensity	20% Increase in 1%AEP Rainfall Intensity	30% Increase in 1%AEP Rainfall Intensity
BRAMLEY ST	0	0	0	0
POLDING ST	0	0	0	0
GEMOORE ST	0	0	0	0
DUNKLEY ST	0	0	0	0
GIPPS ST	0	0	0	0
ROSE ST	0	0	0	0
CHARLES ST	0	0	0	0
BEAUMONT ST	0	0	0	0
BROWN ST	0	0	0	0
BRENAN ST	0	0	0	0
JANE ST	0	0	0	0
LINDSAY AVE	0	0	0	0
NEVILLE ST	0	0	0	0
DUBLIN ST	0	0	0	0
GRADY GARDENS	0	0	0	0
CASANDA AVE	0	0	0	0
CARTELA CRES	0	0	0	0
CANARA PL	0	0	0	0
BOURKE ST	0	0	0	0
ROWLEY ST	0	0	0	0
THE HORSLEY DR	0	0	0	0
GALTON ST	0	0	0	0
MOIR ST	0	0	0	0
HART ST	0	0	0	0
VICTORIA ST	0	0	0	0
SHAMROCK ST	0	0	0	0
KINGSFORD ST	0	0	0	0
HINKLER ST	0	0	0	0
CHIFLEY ST	0	0	0	0
MEGAN AVE	0	0	0	0
NUNDLE ST	0	0	0	0
RHONDDA ST	0	0	0	0
MARKET ST	1	1	1	1
O'CONNELL ST	0	0	0	1
Total	**1**	**1**	**1**	**2**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)

NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

Table J10 - Buildings Subject to Above Floor Flooding (Commercial)				
	1% AEP	10% Increase in 1%AEP Rainfall Intensity	20% Increase in 1%AEP Rainfall Intensity	30% Increase in 1%AEP Rainfall Intensity
BRAMLEY ST	0	0	0	0
POLDING ST	0	0	0	0
GEMOORE ST	0	0	0	0
DUNKLEY ST	0	0	0	0
GIPPS ST	0	0	0	0
ROSE ST	0	0	0	0
CHARLES ST	0	0	0	0
BEAUMONT ST	0	0	0	0
BROWN ST	0	0	0	0
BRENAN ST	0	0	0	0
JANE ST	0	0	0	0
LINDSAY AVE	0	0	0	0
NEVILLE ST	0	0	0	0
DUBLIN ST	0	0	0	0
GRADY GARDENS	0	0	0	0
CASANDA AVE	0	0	0	0
CARTELA CRES	0	0	0	0
CANARA PL	0	0	0	0
BOURKE ST	0	0	0	0
ROWLEY ST	0	0	0	0
THE HORSLEY DR	0	0	0	0
GALTON ST	0	0	0	0
MOIR ST	0	0	0	0
HART ST	0	0	0	0
VICTORIA ST	0	0	0	0
SHAMROCK ST	0	0	0	0
KINGSFORD ST	0	0	0	0
HINKLER ST	0	0	0	0
CHIFLEY ST	0	0	0	0
MEGAN AVE	0	0	0	0
NUNDLE ST	0	0	0	0
RHONDDA ST	0	0	0	0
MARKET ST	0	1	1	1
O'CONNELL ST	0	0	0	0
Total	**0**	**1**	**1**	**1**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)

NOTE: The number of buildings subject to above floor flooding is indicative only. The potential for above floor flooding should be confirmed by comparing flood levels with surveyed floor level data.

DEPTH-DAMAGE INPUTS AND CURVES

Table J11 - Residential Flood Damage Input Paramaters

DIRECT COST INPUTS

Flood Damage Parameter	Recommended Range	Adopted Value	Source
Regional Cost Variation Factor		1	From Rawlinsons
Post late 2001 adjustments	AWE as factor compared to late 2001	2.28	From ABS (http://www.abs.gov.au/ausstats/abs@.nsf/mf/6302.0)AWE in November 2014 is $1539.40
Post Flood Inflation Factor	1.0 to 1.5	1	From OEH Residential Damage Curve Spreadsheet v 3.00, Metro City
Typical Duration of Immersion		0.5 hours	From Emergency Response Classification Results
Building Damage Repair Limitation Factor	0.85 (short duration) to 1.00 (long duration)	0.85	From OEH Residential Damage Curve Spreadsheet v 3.00
Typical House Size		120 & 190m^2	From GIS analysis of housing polygons
Average Contents Relevant To Site		$29,548	2009-10 contents value for Smithfield from ABS (http://www.abs.gov.au/AUSSTATS/abs@.nsf/Lookup/4102.0Main+Features10Dec+2010) = $61,000. Adjusted to 2015 dollars = $67,370 and then adjusted to 2001 dollars = $29,548 for input into OEH spreadsheet
Contents Damage Repair Limitation Factor	0.75 (short duration) to 0.90 (long duration)	0.75	From OEH Residential Damage Curve Spreadsheet v 3.00
Level of Flood Awareness	Low default unless otherwise justifiable	Low	From OEH Residential Damage Curve Spreadsheet v 3.00
Effective Warning Time		0	As quick onset and short duration floods typical
Typical Table/Bench Height (TTBH)	0.9m is typical height. If typical is 2 storey, use 2.6m	0.9	From OEH Residential Damage Curve Spreadsheet v 3.00
External Damage	$6,700 recommended	$6,700	From OEH Residential Damage Curve Spreadsheet v 3.00
Up to Second Floor Level, less than		2.6m	From OEH Residential Damage Curve Spreadsheet v 3.00
From Second Storey up, greater than		2.6m	From OEH Residential Damage Curve Spreadsheet v 3.00
Up to Second Floor Level, less than (% single storey slab on ground)		70%	From OEH Residential Damage Curve Spreadsheet v 3.00
From Second Storey up, greater than (% single storey slab on ground)		110%	From OEH Residential Damage Curve Spreadsheet v 3.00

INDIRECT COST INPUTS

Flood Damage Parameter	Recommended Range	Adopted Value	Source
Clean Up Costs	$4,000 recommended	$5,600	From OEH Residential Damage Curve Spreadsheet v 3.00
Likely Time in Alternate Accommodation		2 weeks (above floor flooding only)	Assuming it takes 2 weeks to clean up and re-establish habitability of house
Additional accommodation costs/Loss of Rent	$220 recommended without justification	$310/week	ABS (Smithfield locality average rental payments)
Loss of wage during clean up	$1,040 per week	$1,040 (below floor flooding) $2,080 (above floor flooding)	Assuming it takes 1 week to clean up yard / external areas and an additional week to clean inside buildings. ABS (Smithfield locality average weekly household income)

Table J12 - Depth-Damage relationships for Residential Properties (2014 $)						
Depth	Single Storey High 120m^2 Floor Area	Single Storey Low 120m^2 Floor Area	Two Storey 120m^2 Floor Area	Single Storey High 190m^2 Floor Area	Single Storey Low 190m^2 Floor Area	Two Storey 190m^2 Floor Area
-5	$0	$0	$0	$0	$0	$0
-1.5	$0	$0	$0	$0	$0	$0
-1.4	$0	$0	$0	$0	$0	$0
-1.3	$0	$0	$0	$0	$0	$0
-1.2	$15,276	$0	$0	$15,276	$0	$0
-1.1	$21,236	$0	$0	$24,713	$0	$0
-1	$21,958	$0	$0	$25,856	$0	$0
-0.9	$22,681	$0	$0	$27,000	$0	$0
-0.8	$23,403	$0	$0	$28,144	$0	$0
-0.7	$24,125	$0	$0	$29,287	$0	$0
-0.6	$24,847	$0	$0	$30,431	$0	$0
-0.5	$25,570	$15,276	$15,276	$31,574	$15,276	$15,276
-0.4	$26,292	$15,276	$15,276	$32,718	$15,276	$15,276
-0.3	$27,014	$15,276	$15,276	$33,861	$15,276	$15,276
-0.2	$27,736	$15,276	$15,276	$35,005	$15,276	$15,276
-0.1	$28,459	$15,276	$15,276	$36,148	$15,276	$15,276
0	$46,023	$28,032	$24,205	$54,134	$35,472	$29,413
0.1	$48,430	$47,030	$37,504	$56,962	$54,746	$42,905
0.2	$50,836	$49,186	$39,013	$59,790	$57,178	$44,607
0.3	$53,243	$51,343	$40,523	$62,618	$59,609	$46,309
0.4	$55,649	$53,499	$42,032	$65,446	$62,041	$48,011
0.5	$58,056	$55,655	$43,541	$68,273	$64,472	$49,714
0.6	$60,462	$57,811	$45,051	$71,101	$66,904	$51,416
0.7	$62,869	$59,968	$46,560	$73,929	$69,336	$53,118
0.8	$65,275	$62,124	$48,069	$76,757	$71,767	$54,820
0.9	$67,682	$64,280	$49,579	$79,585	$74,199	$56,522
1	$70,088	$66,436	$51,088	$82,412	$76,630	$58,224
1.1	$72,495	$68,592	$52,597	$85,240	$79,062	$59,926
1.2	$74,901	$70,749	$54,107	$88,068	$81,493	$61,628
1.3	$77,308	$72,905	$55,616	$90,896	$83,925	$63,330
1.4	$79,714	$75,061	$57,126	$93,724	$86,356	$65,032
1.5	$82,121	$77,217	$58,635	$96,551	$88,788	$66,734
1.6	$84,527	$79,373	$60,144	$99,379	$91,219	$68,436
1.7	$86,934	$81,530	$61,654	$102,207	$93,651	$70,138
1.8	$89,340	$83,686	$63,163	$105,035	$96,082	$71,841
1.9	$91,747	$85,842	$64,672	$107,863	$98,514	$73,543
2	$94,153	$87,998	$66,182	$110,691	$100,946	$75,245
2.1	$94,875	$88,470	$66,512	$111,834	$101,693	$75,768
2.2	$95,598	$88,942	$66,842	$112,978	$102,440	$76,291
2.3	$96,320	$89,414	$67,173	$114,121	$103,187	$76,814
2.4	$97,042	$89,886	$67,503	$115,265	$103,935	$77,337
2.5	$97,764	$90,358	$67,834	$116,408	$104,682	$77,860
2.6	$98,487	$90,830	$68,164	$117,552	$105,429	$78,383
2.7	$99,209	$91,302	$98,905	$118,695	$106,177	$115,267
2.8	$99,931	$91,774	$99,424	$119,839	$106,924	$116,089
2.9	$100,653	$92,246	$99,943	$120,983	$107,671	$116,911
3	$101,376	$92,718	$100,462	$122,126	$108,419	$117,733
3.5	$104,987	$95,078	$103,058	$127,844	$112,155	$121,843
4	$108,598	$97,438	$105,654	$133,562	$115,892	$125,953
4.5	$112,209	$99,798	$108,250	$139,280	$119,628	$130,063
5	$115,821	$102,158	$110,846	$144,998	$123,365	$134,173

Table J13 - Depth-Damage relationships for Commercial Properties (2014 $)			
Depth above main work areas (m)	CL	CM	CH
	Low Value	Medium Value	High Value
0.0-0.2	$6,900	$13,800	$28,980
0.21-0.25	$9,660	$17,940	$35,880
0.26-0.3	$11,040	$20,700	$40,020
0.31-0.5	$15,180	$31,740	$63,480
0.51-0.6	$17,940	$37,260	$71,760
0.61-0.75	$23,460	$48,300	$88,320
0.76-0.9	$24,840	$51,060	$103,500
0.91-1.0	$27,600	$56,580	$111,780
1.01-1.2	$31,740	$63,480	$128,340
1.21-1.25	$33,120	$67,620	$136,620
1.26-1.5	$34,500	$71,760	$143,520
1.51-1.75	$37,260	$75,900	$151,800
1.76-2.0	$40,020	$80,040	$160,080
>2.0	$41,400	$82,800	$165,600

Table J14 - Depth-Damage relationships for Industrial Properties (2014 $)			
Depth above main work areas (m)	IL	IM	IH
	Low Value	Medium Value	High Value
0.0-0.2	$23,460	$48,300	$96,600
0.21-0.25	$28,980	$56,580	$111,780
0.26-0.3	$31,740	$63,480	$128,340
0.31-0.5	$48,300	$96,600	$191,820
0.51-0.6	$56,580	$111,780	$223,560
0.61-0.75	$71,760	$136,620	$271,860
0.76-0.9	$80,040	$160,080	$320,160
0.91-1.00	$88,320	$176,640	$343,620
1.01-1.2	$96,600	$200,100	$400,200
1.21-1.25	$103,500	$208,380	$416,760
1.26-1.5	$111,780	$223,560	$440,220
1.51-1.75	$120,060	$231,840	$463,680
1.76-2.0	$128,340	$240,120	$480,240
>2.0	$131,100	$241,500	$483,000

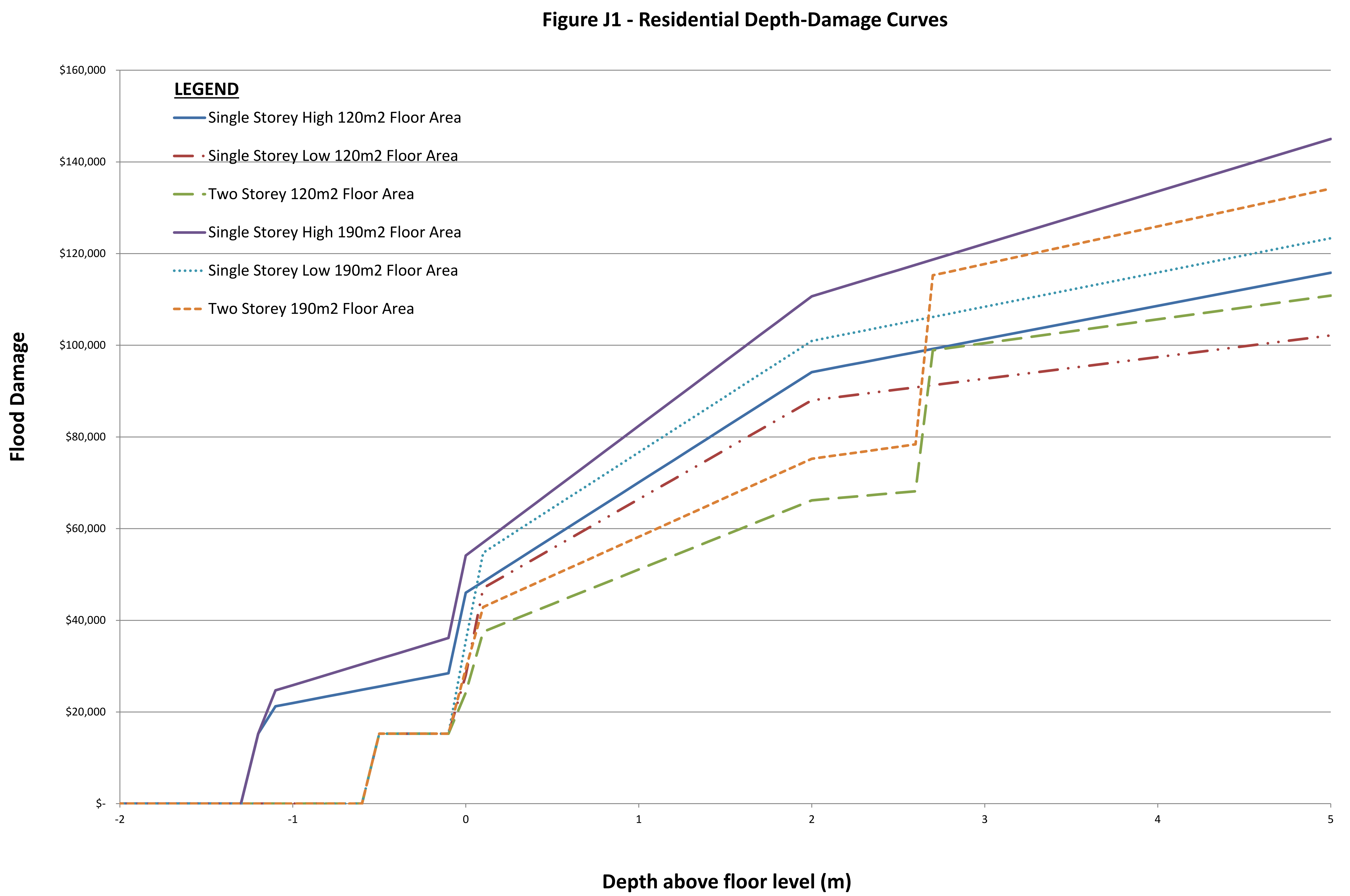


Figure J1 - Residential Depth-Damage Curves
LEGEND
Single Storey High 120m2 Floor Area
Single Storey Low 120m2 Floor Area
Two Storey 120m2 Floor Area
Single Storey High 190m2 Floor Area
Single Storey Low 190m2 Floor Area
Two Storey 190m2 Floor Area
$160,000
$140,000
$120,000
$100,000
$80,000
$60,000
$40,000
$20,000
$-
-2
-1
0
1
2
3
4
5
Flood Damage
Depth above floor level (m)

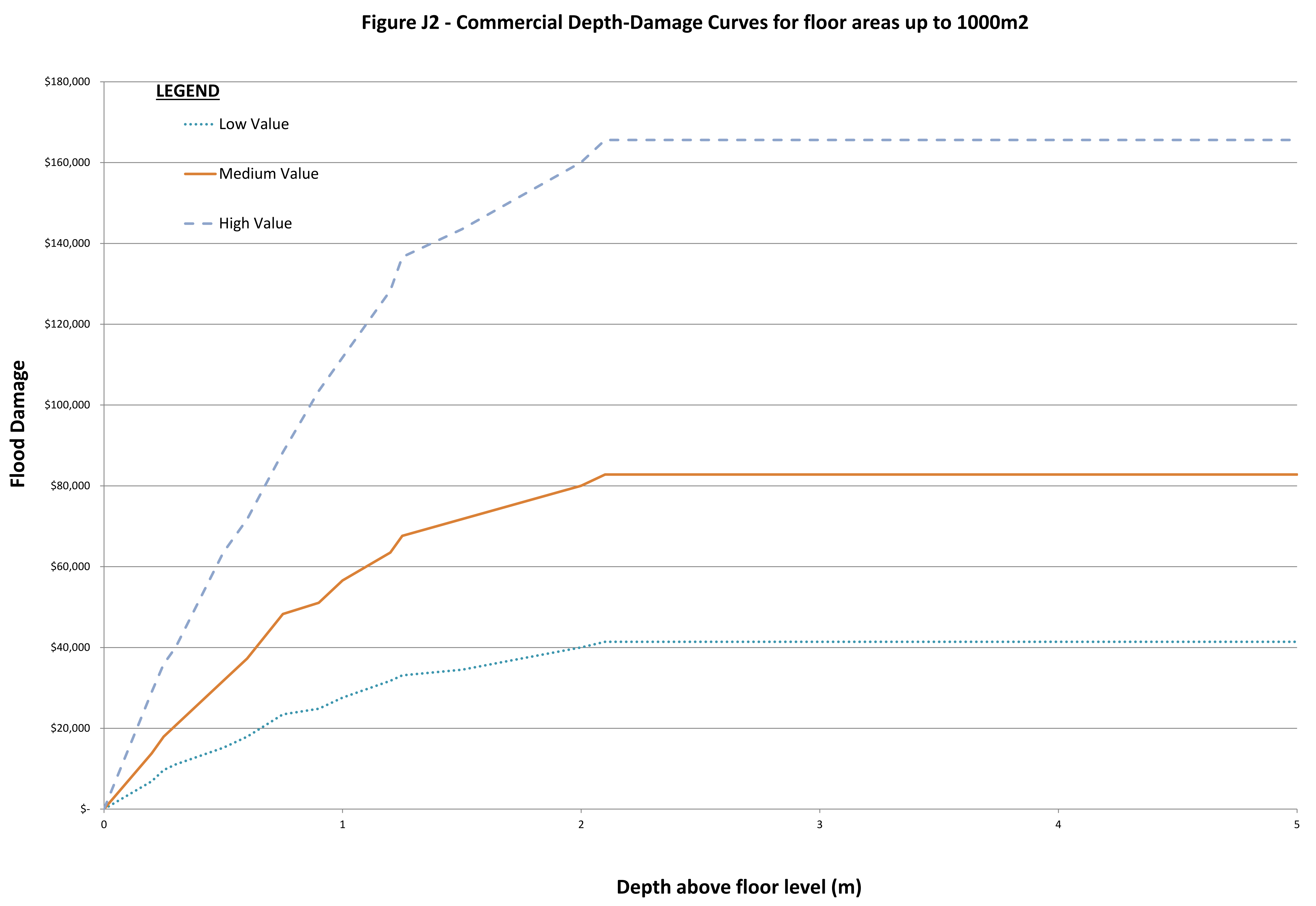


Figure J2 - Commercial Depth-Damage Curves for floor areas up to 1000m2
LEGEND
Low Value
Medium Value
High Value
$180,000
$160,000
$140,000
$120,000
$100,000
$80,000
$60,000
$40,000
$20,000
$-
0
1
2
3
4
5
Flood Damage
Depth above floor level (m)

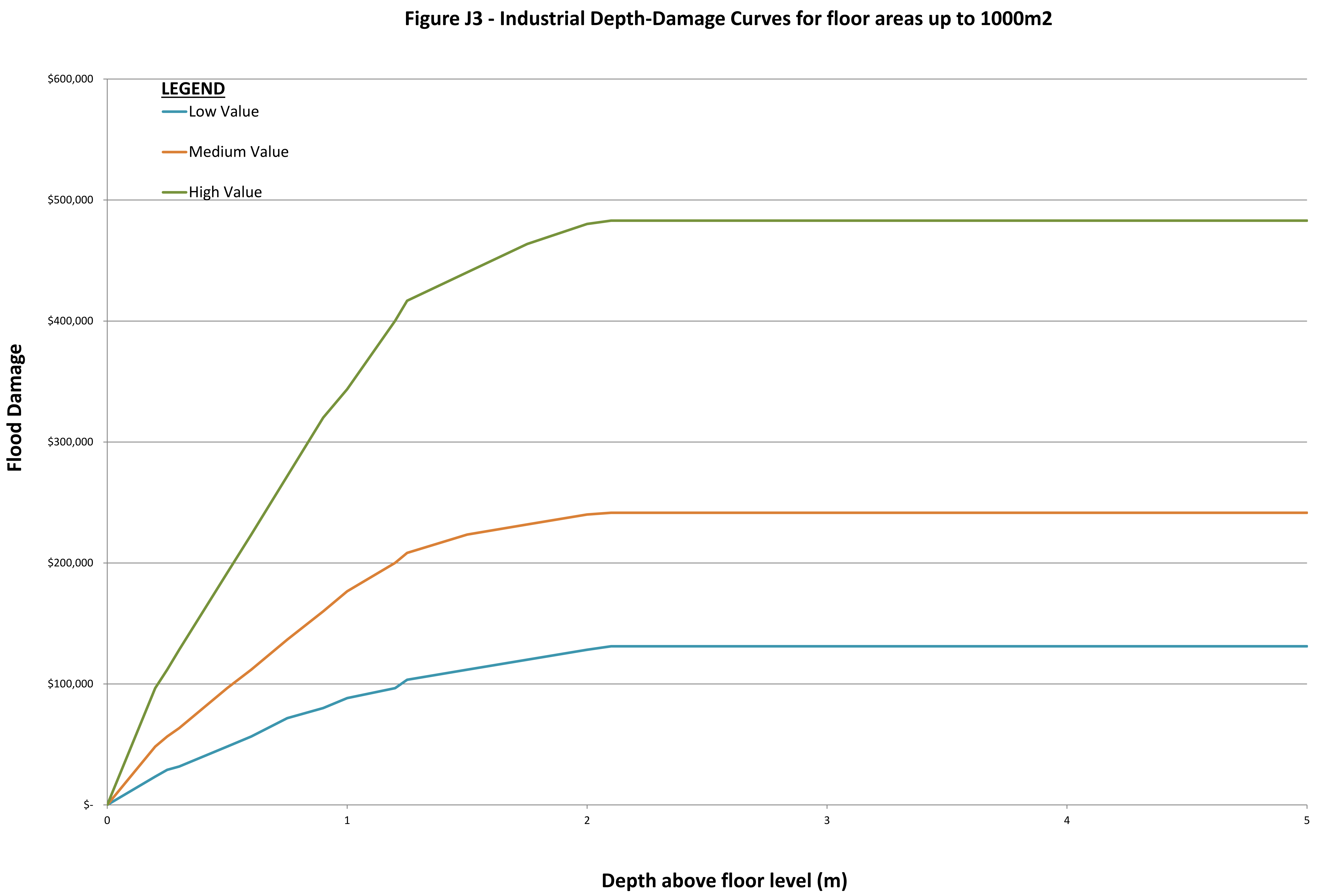


Figure J3 - Industrial Depth-Damage Curves for floor areas up to 1000m2
LEGEND
Low Value
Medium Value
High Value
Flood Damage
$600,000
$500,000
$400,000
$300,000
$200,000
$100,000
$-
0
1
2
3
4
5
Depth above floor level (m)

FLOOD DAMAGE ASSESSMENT

Table J15 - Direct Flood Damages per Street ($1000s)								
	50% AEP	20% AEP	10% AEP	5% AEP	1% AEP	0.2% AEP	1 in 10000yr	PMF
BRAMLEY ST	$0	$0	$0	$0	$0	$0	$0	$0
POLDING ST	$0	$0	$0	$0	$0	$0	$0	$0
GEMOORE ST	$0	$0	$0	$0	$0	$0	$26	$95
DUNKLEY ST	$0	$0	$0	$0	$0	$0	$25	$27
GIPPS ST	$0	$0	$0	$50	$53	$67	$205	$303
ROSE ST	$0	$0	$0	$0	$0	$0	$0	$0
CHARLES ST	$0	$0	$0	$0	$0	$0	$0	$48
BEAUMONT ST	$0	$0	$0	$0	$0	$0	$0	$0
BROWN ST	$0	$30	$35	$79	$84	$195	$220	$691
BRENAN ST	$0	$0	$0	$68	$75	$79	$102	$393
JANE ST	$0	$54	$54	$84	$215	$237	$334	$779
LINDSAY AVE	$0	$0	$0	$0	$0	$26	$27	$154
NEVILLE ST	$0	$54	$54	$55	$84	$199	$241	$749
DUBLIN ST	$25	$61	$62	$91	$101	$151	$248	$506
GRADY GARDENS	$0	$0	$0	$0	$0	$0	$0	$0
CASANDA AVE	$0	$0	$69	$73	$76	$79	$83	$126
CARTELA CRES	$26	$26	$50	$97	$100	$194	$257	$519
CANARA PL	$46	$72	$73	$104	$122	$127	$272	$681
BOURKE ST	$0	$0	$0	$0	$0	$0	$0	$116
ROWLEY ST	$0	$0	$0	$0	$0	$0	$0	$0
THE HORSLEY DR	$50	$103	$135	$137	$170	$173	$275	$1,078
GALTON ST	$0	$0	$0	$0	$0	$0	$0	$0
MOIR ST	$0	$27	$55	$127	$191	$275	$453	$1,080
HART ST	$88	$135	$144	$204	$236	$304	$393	$579
VICTORIA ST	$28	$55	$202	$507	$629	$682	$788	$1,265
SHAMROCK ST	$0	$0	$0	$0	$0	$0	$181	$307
KINGSFORD ST	$25	$25	$25	$54	$57	$133	$196	$307
HINKLER ST	$0	$25	$25	$144	$303	$338	$443	$860
CHIFLEY ST	$0	$108	$139	$163	$330	$458	$771	$2,378
MEGAN AVE	$0	$0	$0	$0	$0	$0	$0	$93
NUNDLE ST	$0	$0	$0	$0	$0	$0	$0	$15
RHONDDA ST	$0	$0	$0	$0	$0	$54	$55	$492
MARKET ST	$0	$0	$0	$0	$1	$18	$53	$276
O'CONNELL ST	$0	$0	$0	$0	$0	$2	$19	$81
Total	**$288**	**$775**	**$1,123**	**$2,036**	**$2,827**	**$3,792**	**$5,668**	**$13,997**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)

Table J16 - Direct Flood Damages per Street ($1000s)				
	1% AEP	10% Increase in 1%AEP Rainfall Intensity	20% Increase in 1%AEP Rainfall Intensity	30% Increase in 1%AEP Rainfall Intensity
BRAMLEY ST	$0	$0	$0	$0
POLDING ST	$0	$0	$0	$0
GEMOORE ST	$0	$0	$0	$25
DUNKLEY ST	$0	$0	$0	$0
GIPPS ST	$53	$60	$64	$71
ROSE ST	$0	$0	$0	$0
CHARLES ST	$0	$0	$0	$0
BEAUMONT ST	$0	$0	$0	$0
BROWN ST	$84	$85	$88	$96
BRENAN ST	$75	$76	$78	$82
JANE ST	$215	$230	$235	$241
LINDSAY AVE	$0	$26	$26	$26
NEVILLE ST	$84	$90	$103	$182
DUBLIN ST	$101	$140	$147	$158
GRADY GARDENS	$0	$0	$0	$0
CASANDA AVE	$76	$77	$78	$79
CARTELA CRES	$100	$150	$192	$198
CANARA PL	$122	$124	$127	$157
BOURKE ST	$0	$0	$0	$0
ROWLEY ST	$0	$0	$0	$0
THE HORSLEY DR	$170	$171	$173	$174
GALTON ST	$0	$0	$0	$0
MOIR ST	$191	$203	$213	$222
HART ST	$236	$296	$332	$310
VICTORIA ST	$629	$653	$675	$693
SHAMROCK ST	$0	$0	$0	$0
KINGSFORD ST	$57	$61	$66	$138
HINKLER ST	$303	$317	$331	$347
CHIFLEY ST	$330	$351	$450	$473
MEGAN AVE	$0	$0	$0	$0
NUNDLE ST	$0	$0	$0	$0
RHONDDA ST	$0	$0	$0	$0
MARKET ST	$1	$4	$13	$21
O'CONNELL ST	$0	$0	$0	$5
Total	**$2,827**	**$3,115**	**$3,391**	**$3,698**

NOTE: Streets are arranged from upstream to downstream through the Smithfield West Study Area (ie: South to North)

Table J17 - Total Flood Damages ($1000s)				
	1% AEP	**10% Increase in 1%AEP Rainfall Intensity**	**20% Increase in 1%AEP Rainfall Intensity**	**30% Increase in 1%AEP Rainfall Intensity**
Direct Residential	$2,826	$3,111	$3,379	$3,673
Indirect Residential	$978	$1,030	$1,081	$1,126
Direct Industrial	$1	$3	$6	$13
Indirect Industrial	$0	$1	$1	$3
Direct Commercial	$0	$1	$7	$13
Indirect Commercial	$0	$0	$1	$3
Infrastructure	$424	$467	$509	$555
Total	**$4,229**	**$4,613**	**$4,984**	**$5,384**