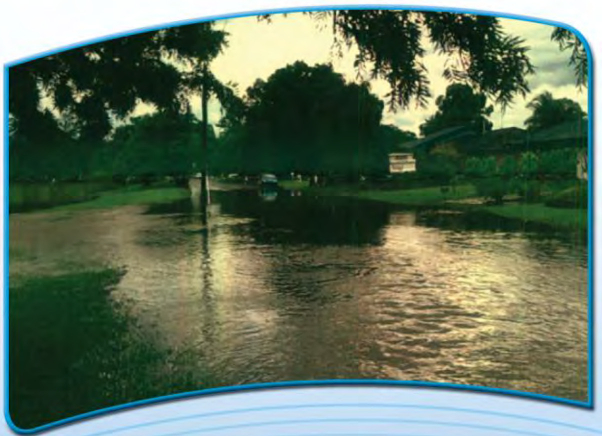
Peach Tree and Lower Surveyors Creeks Flood Study

Final Report

Volume 1 of 3: Main Report

April 2019



PENRITH CITY COUNCIL

Catchment Simulation Solutions

Peach Tree and Lower Surveyors Creeks Flood Study

Final Report

REVISION / REVIEW HISTORY

Revision #	Description	Prepared by	Reviewed by
1	Preliminary draft report	D. Tetley	C. Ryan
2	Draft report	D. Tetley	C. Ryan
3	Final report	D. Tetley	C. Ryan

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Note: The Study has been adopted by Council in its Ordinary Meeting on 29th April, 2019.

Prepared By:

Catchment Simulation Solutions

Suite 2.01
210 George Street
Sydney, NSW, 2000

(02) 8355 5500 dtetley@csse.com.au

(02) 8355 5505 www.csse.com.au

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Penrith City Council
601 High Street, Penrith NSW 2750
council@penrithcity.nsw.gov.au
(02) 4732 7777

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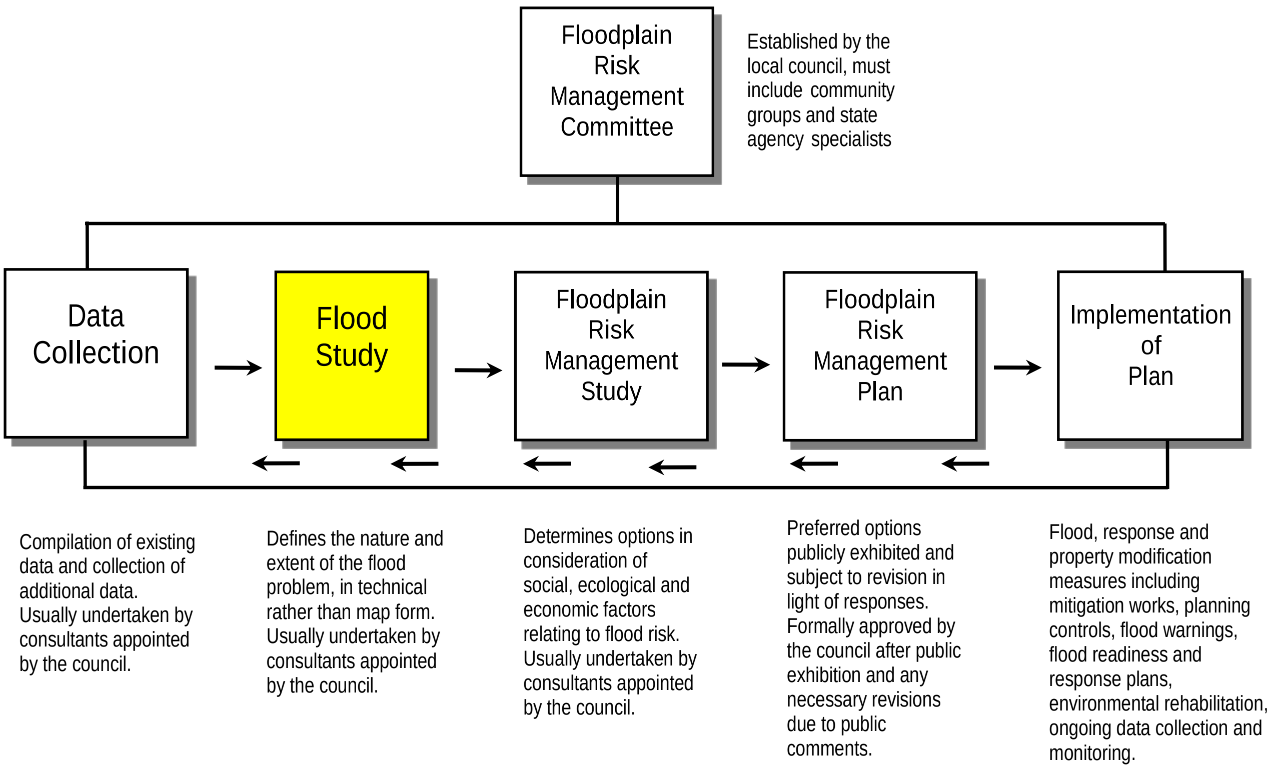
FOREWORD

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 Peach Tree and Lower Surveyors Creeks Flood Study represents the second of the five stages in the 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.

TABLE OF CONTENTS

LIST OF APPENDICES (VOLUME 3)

LIST OF TABLES

LIST OF PLATES

1 Introduction

The *Peach Tree and Lower Surveyors Creeks Flood Study* covers an area of 1,250 hectares within the Penrith City Council Local Government Area (LGA). As shown in **Figure 1**, the study area extends across parts of the suburbs of Penrith, South Penrith and Jamisontown. The study area occupies the lower part of a larger 2,450 hectare catchment that originates in the Glenmore Park area and ultimately drains into the Nepean River just west of Penrith.

The study area is highly urbanised and includes a mix of residential, commercial and industrial areas with scattered areas of open space. Most of the highly urbanised portion of the study area is drained by a sub-surface stormwater system. During periods of heavy rainfall there is potential for the capacity of the stormwater system to be exceeded, leading to local overland flooding. There is also potential for "mainstream" flooding because of water overtopping the banks of the major watercourses in the study area, as well as inundation from the adjoining Nepean River.

Penrith City Council completed a broad-scale, overland flooding "Overview Study" in 2006 to identify overland flow paths and better understand the potential risk of flooding across the Penrith LGA. This Overview Study has provided Council with a basis on which to undertake a program of more detailed overland flow flood studies. The Peach Tree and Lower Surveyors Creeks Catchment has been identified as the next priority catchment requiring a detailed flood study.

Accordingly, Penrith City Council engaged Catchment Simulation Solutions to prepare the flood study for the Peach Tree and Lower Surveyors Creeks catchment. It documents flood behaviour across the catchment for a range of historic and design floods. This includes information on flood discharges, levels, depths and flow velocities. 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.

The flood study comprises two volumes:

- Volume 1 (this document): contains the report text;
- Volume 2: contains all figures and maps; and,
- Volume 3: contains all appendices referred to in this document.

It should be noted that the primary objective of the study was to define flood behaviour across the Peach Tree and Lower Surveyors Creeks study area shown in **Figure 1**. Although flooding along the Nepean River and its potential to interact with floodwaters from the local catchment was considered as part of the study, Nepean River flooding was not the focus of the study. A dedicated flood study for Nepean River was prepared and is documented in the "Nepean River Flood Study: Exhibition Draft Report" (Advisian, 2017).

2 METHODOLOGY

2.1 Objectives

Penrith City Council outlined a range of objectives for the Peach Tree and Lower Surveyors Creeks Flood Study. This included:

- to review available flood-related information and historic flood data for the catchment;
- to consult with the community to gain an understanding of flooding and drainage 'trouble spots' and gather information on past floods;
- to undertake a detailed survey of the creeks, open channels, bridges and culverts;
- to develop a computer-based hydrologic flood model to simulate the transformation of rainfall into runoff
- to develop a computer based hydraulic model to simulate the movement of runoff across the catchment;
- to calibrate and validate the computer models against observed information on past floods;
- to use the calibrated and validated computer models to estimate peak discharges, water levels, depths and velocities for the design 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods as well as the 0.5 exceedances per year (0.5EY) flood and the Probable Maximum Flood (PMF);
- to verify the design flood results against other studies as well as the experiences of residents and business owners in the catchment;
- to produce maps showing predicted floodwater depths, levels and velocities for the full range of design floods;
- to quantify the capacity of the existing stormwater drainage system;
- to produce maps showing flood hazard and flood function (i.e., hydraulic categories) for the 5%, 1% and 0.5% AEP floods and the PMF;
- to produce emergency response precinct classification mapping to assist the State Emergency Service with emergency response planning;
- to quantify the potential impact of climate change on existing design flood behaviour;
- to quantify the potential impact of future development on existing flood behaviour
- to provide information to assist with land use planning activities;
- to develop a list of preliminary flood mitigation options; and
- to map the flood planning area and preliminary flood control lots.

2.2 Adopted Approach

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

- compilation and review of available flood-related information and consultation with the community (Chapter 3);

- the development of a hydrologic model to simulate the transformation of rainfall into runoff and development of a hydraulic model to simulate the movement of floodwaters across the study area (Chapter 4);
- calibration and validation of the computer models to reproduce historic floods (Chapter 5);
- use of the computer models to estimate 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 and flood function mapping as well as flood emergency response classifications (Chapter 7);
- testing the sensitivity of the results generated by the computer model to variations in model input parameters, future development and climate change (Chapter 8);
- use of computer model outputs and sensitivity analysis results to prepare flood planning area mapping (Chapter 9); and,
- identification of flooding “hot spots” and preparation of a preliminary list of mitigation measures that could be potentially implemented to mitigate the flood risk across these areas (Chapter 10).

3 DATA COLLECTION AND REVIEW

3.1 Overview

A range of data were made available to assist with the preparation of the Peach Tree and Lower Surveyors Creeks Flood Study. This included previous reports, hydrologic and hydraulic data, plans, survey information and GIS data.

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

3.2 Previous Reports

3.2.1 Penrith Overland Flow Flood "Overview Study" (2006)

The *'Penrith Overland Flow Flood "Overview Study"'* report was prepared by Cardno Lawson Treloar Pty Ltd for Penrith City Council in 2006. The study aimed to define the nature and extent of overland flood behaviour across the Penrith City Council LGA and generate sufficient information to identify overland flow paths and define the variation in flood risk and prioritise subcatchments within the LGA for detailed overland flow studies.

Flood behaviour across the LGA was defined using two-dimensional (2D) hydraulic models that were developed using the SOBEK modelling software. The topography within the model was based on a Digital Terrain Model (DTM) developed from Airborne Laser Scanning (ALS) survey collected in 2002. The Direct Rainfall Method (DRM) was adopted to define hydrology as part of the study whereby design rainfall is applied directly to the model.

A coarse 45 metre grid model was developed to define flood behaviour across the entire LGA, and smaller, fine-scale grids were nested within the larger model to define overland flow behaviour in detail across critical areas. Two separate fine-scale grid sizes were used as part of the study:

- A 3 metre nested grid was used across urbanised areas in the central region of the LGA; and,
- a 9 metre grid was adopted for less urbanised areas in the north and south of the LGA.

The Peach Tree and Surveyors Creeks subcatchments were represented using a 3 metre grid.

Stormwater drainage infrastructure was not included in the model. As such, the modelling did not consider the conveyance of flows within the underground stormwater system. Therefore, all flows within the model were assumed to travel overland and the overland flow estimations are approximate only.

Major culverts and bridges in the study area were included as one-dimensional (1D) components within the fine grid and only a limited number of structures at critical locations were included in the coarse grid. The location of structures that were included in the overview study model are shown in **Figure A1** in **Appendix A**.

Buildings were represented in the model 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 and is also likely to result in conservative flood level estimates.

The computer model was used to simulate the design 5% and 1% AEP floods as well as the PMF and produce information on flood flows, levels and velocities across the LGA. The depth and velocity results were also used to define the provisional flood hazard.

The LGA was divided into subcatchments approximately 100 hectares in size providing a total of 249 subcatchments across the LGA. The flood risk for all subcatchments was quantified based on hazard and economic risk criteria with the objective of ranking the subcatchments and establishing priorities for undertaking detailed flood studies in the future. The hazard risk was calculated as the product of the number of properties within the provisional high hazard area and the probability of each design flood event occurring. The economic risk was estimated based upon the Annual Average Damages (AAD) estimates for each subcatchment.

The 249 subcatchments were subsequently split into 10 percentile bands, with the 10% band representing the highest 10% of the flood affected subcatchments. The Overview Study identified both the Peach Tree and Surveyors Creeks subcatchments within the highest 10% of flood affected subcatchments across the LGA.

The overview study is considered to provide a good initial understanding of the potential flood risk across the Peach Tree and Surveyors Creeks catchments. However, as no representation of the stormwater system is provided, it is likely to be overestimating the flood risk across the "built up" sections of each subcatchment. Furthermore, the terrain representation is based on ALS data collected in 2002, so would not reflect any changes in topography that has occurred over the past 15 years. Nevertheless, the results from this study were of value in validating the performance of the computer models developed for the current study.

3.2.2 Penrith CBD Detailed Overland Flow Flood Study (2015)

The *'Penrith CBD Detailed Overland Flow Flood Study'* report was prepared by Cardno for Penrith City Council in 2015. The study was initiated by Council after the Penrith CBD subcatchment was identified as being within the highest 10% of flood affected subcatchments within the *"Overview Study"* discussed in the previous section. Therefore, Council resolved to prepare a detailed overland flood study to better understand the nature and extent of the existing flood risk across the Penrith CBD.

The Penrith CBD study area is located immediately east of the Peach Tree and Lower Surveyors Creeks study area. Furthermore, some sections of the CBD study area drain into the Peach Tree and Lower Surveyors Creeks study area. This includes the Showground channel and Racecourse channel which drain into Peach Tree Creek in the vicinity of the Panthers World of Entertainment site. The extent of the Penrith CBD study area is shown in **Figure A2** in **Appendix A**.

A direct rainfall computer model of the CBD catchment was developed using the TUFLOW software as part of the study. The TUFLOW model incorporated the following features:

- The model uses a 1m grid size to define the variation in topography and hydraulic roughness
- Bridges and culverts were surveyed and represented as 1-dimensional structures in the model. The location of bridges and culverts included in the TUFLOW model are shown in **Figure A1**.
- The Showground channel and Racecourse channel were represented as 1-dimensional elements. The conveyance capacity of the 1-dimensional channels was defined using surveyed cross-sections. The location of the cross-sections is shown in **Figure A3**.
- All stormwater pits and pipes were included as 1-dimensional elements. The extent of the stormwater system included in the TUFLOW model is shown in **Figure A4**. The capacity of the various stormwater pits was represented using custom water depth versus pit inflow relationships.
- The model topography was defined based upon ALS data collected in 2002. Therefore, the terrain representation in the model would not reflect any topographic changes that have occurred since 2002.
- Buildings within the floodplain (i.e., within the PMF extent) were represented as complete flow impediments. Buildings outside of the floodplain were represented using a high Manning's "n" value of 0.1.
- The TUFLOW model did not include a full representation of the five upstream catchments that drain into the Penrith CBD study area. Therefore, an XP-RAFTS hydrologic model was also developed as part of the study and was used to define inflows into the TUFLOW model for those five catchments.
- A free outfall downstream boundary condition at Peach Tree Creek was adopted. It was determined that the adopted downstream boundary condition had little impact on modelling results across the majority of the study area.

An attempt to calibrate the TUFLOW model was made as part of the study. However, a lack of historic flood marks and rainfall information meant that a comprehensive calibration could not be completed. Therefore, the model was validated by comparing design flood modelling results against reports of property inundation. The outcomes of this validation showed that the inundation extents generated by the TUFLOW model results generally coincided with areas where historic inundation was reported. There were 19 locations where inundation was reported by the community that could not be reproduced by the model. The report suggests that this may be associated with localised drainage issues (e.g., blockage of stormwater pits and pipes).

The validated model was used to simulate flood behaviour for a range of design floods ranging from the 1-year ARI up to the PMF. The results from the design flood modelling were used to prepare a range of flood maps showing the level, depth and velocity of floodwaters. Flood hazard and hydraulic category mapping was also prepared

A stormwater pipe capacity assessment was also prepared which suggested that 52 pipes within the study area had a nominal capacity of less than the 1 in 5 year ARI flood.

Overall, the Penrith CBD Flood Study provides the most contemporary and detailed assessment of flood behaviour across the Penrith CBD and surrounds. It is considered that the surveyed cross-sections, bridges/culvert details and stormwater system utilised in this study could also be used to assist in the development of the hydraulic model for the current study. Furthermore, it is considered that the results of the modelling could be used to validate the results generated as part of the current study across common model areas.

3.2.3 Peach Tree Creek Flood Study (1994)

The *'Peach Tree Creek Flood Study'* report was prepared for Penrith City Council by NSW Public Works in 1994. The primary goal of the flood study was to provide a comprehensive assessment of existing flood behaviour across the Peach Tree Creek catchment that would serve as the basis for preparing a floodplain risk management plan for the catchment. The study area included the Peach Tree Creek catchment from the M4 Motorway downstream to its confluence with the Nepean River. The study area also extended west to the Nepean River and east towards Mulgoa Road. It is noted that only the very lower sections of Surveyors Creek, Racecourse channel and Showground channel (west of Mulgoa Road) were included in this flood study.

The study took advantage of an existing quasi 2-dimensional hydraulic model that was developed using the FPLAIN software. The original hydraulic model was developed in 1991 as part of the *'Surveyors Creek/Peach Tree Creek Hydraulic Study'* (Lyall and Macoun).

The report states that no information is available describing historic flood behaviour within the Peach Tree Creek floodplain. Therefore, calibration of the model was not attempted.

The study recognised that during large Nepean River floods, water would "back up" from the Nepean River into the Peach Tree Creek catchment and contribute to inundation. During the 1% AEP Nepean River flood, water from the river is predicted to spill through two sets of M4 Motorway culverts via School House Creek and enter the Peach Tree Creek system. During the 0.5% AEP flood, water is predicted to spill through the M4 culverts and overtop the eastern bank of the Nepean River.

Inflows to the hydraulic model from the Nepean River were defined by peak design water levels in the Nepean River in conjunction with weir relationships at each inflow location. Flows from the local catchment were defined using an XP-RAFTS hydrologic model that was originally developed in 1985 for the *'South Penrith Trunk Drainage Study'* (Willing and Partners).

The study notes that, with the various potential combinations of local catchment runoff and Nepean River flooding, it is difficult to assign a standard probability to flooding in the catchment. The study ultimately adopted a 'deterministic-stochastic' method whereby local catchment runoff was the deterministic component (i.e., not subject to randomness) and coincidental Nepean River flooding was the stochastic component (i.e., subject to variability). Four combinations of Nepean River and local catchment runoff were selected for the analysis:

1) 1% AEP, 72-hour storm in both catchments;
2) 1% AEP, 90-minute storm in the local catchment with 1% AEP, 72-hour storm in the Nepean River (with peaks occurring at the same time);

3) As above but the local catchment runoff is delayed; and,
4) 5% AEP, 90-minute storm in the local catchment with 1% AEP, 72-hour storm in the Nepean River (with peaks occurring at the same time).

In general, Scenario 2) produced the highest design flood levels across the catchment. However, the report notes that it is highly unlikely that a 90-minute local catchment storm will coincide exactly with the peak of a 72-hour Nepean River flood. Accordingly, the probability of this occurring was most likely rarer than the 1% AEP event. As a result, the study recommends Scenario 4) for defining the 1% AEP flood across downstream section of the catchment

The report notes the following flooding characteristics:

- Inundation of Peach Tree Creek during the early stages of the flood occurs because of water “backing up” from the Nepean River. Water levels are predicted to be sufficiently elevated during a 1% AEP Nepean River Flood to inundate large sections of the Peach Tree catchment west of Mulgoa Road.
- During the 0.5% AEP event, flooding along Peach Tree Creek is dominated by floodwater from the Nepean River.
- Many of the drainage paths in the lower catchment are relatively ill-defined and of limited capacity. Therefore, water is predicted to spill out of most watercourses during relatively frequent events.
- Inundation of the lower floodplain can occur as a result of a variety of different local catchment runoff and Nepean River floods. The critical storm duration for the Nepean River was determined to be 72 hours while the critical duration for the local catchment was determined to be 90-minutes.
- Drainage of the floodplain following a flood is controlled by the water level in the Nepean River. Water levels within the Nepean River are predicted to be maintained near the peak for a period of 12 hours and are significantly elevated for over 36 hours. Accordingly, the lower sections of the catchment can take a significant amount of time to drain.

The study only investigated the 1% AEP and 0.5% AEP floods. Accordingly, the impact of smaller floods was not investigated. Furthermore, the study concentrated on mainstream flooding west of Mulgoa Road. That is, the study did not consider mainstream flooding east of Mulgoa Road or the potential for overland flooding.

The study also utilises flood modelling technology that is considered outdated by modern standards. Furthermore, significant changes have occurred across the upstream catchment areas that would have modified flood behaviour from the local catchment relative to when this study was prepared. Revised design flood information is also available for the Nepean River (refer Section 3.2.4), which may alter inundation characteristics, particularly during large Nepean River floods.

Therefore, the results documented in this report are likely to be outdated. Nevertheless, this study still provides valuable information describing the characterises of flooding across the lower catchment and the interactions between Nepean River and local catchment flooding.

Therefore, it is considered to be a valuable reference document and the results can be used to assist in the validation of the computer model developed for the current study.

3.2.4 Nepean River Flood Study: Exhibition Draft Report (2017)

The 'Nepean River Flood Study: Exhibition Draft Report' was prepared by Advisian for Penrith City Council. The study was prepared to define existing design flood behaviour along the Nepean River. As shown in **Figure 1**, Peach Tree Creek drains into the Nepean River and the Nepean River also forms the western boundary of the Peach Tree and Lower Surveyors Creeks catchment. As a result, flooding within the Nepean River can have a significant impact on flood behaviour across the western sections of the catchment.

The 'Nepean River Flood Study: Exhibition Draft Report' was concerned with mainstream flooding along the Nepean River only. As a result, it does not explicitly include an assessment of flooding from the local Peach Tree and Surveyors Creek catchment. Nevertheless, it provides a considerable amount of information about flooding along the Nepean River.

The outcomes of the flood study showed that during the 5% AEP flood, floodwaters were typically contained within the main channel in the vicinity of the study area. However, water was shown to "back up" along Peach Tree Creek to Jamison Road. The flood study also showed that during the 1% AEP flood, floodwaters overtopped the Nepean River banks south of the M4 Motorway and discharged into the Peach Tree Creek catchment via three of the motorway culverts. During the 0.5% AEP flood (as well as larger events), water enters the lower catchment via the motorway culverts as well as from the Nepean River overtopping its banks between the motorway and Great Western Highway. This outcome confirms that Nepean River flooding can have a significant impact on the western sections of the catchment, particularly during events equal to and greater than the 1% AEP flood.

At the time this current study was prepared the 'Nepean River Flood Study: Exhibition Draft Report' was at exhibition draft stage. Therefore, a final set of flood maps and modelling results were not available. Nevertheless, Penrith City Council considered the modelling results unlikely to change and issued the modelling files in waterRIDE format for use as part of the study. The waterRIDE files allow the full times series of flood information to be reviewed and extracted at any location for the design 5%, 2%, 1%, 0.5%, 0.2% AEP events. waterRIDE files were also provided for the Probable Maximum Flood.

The waterRIDE information was ultimately used to assist in setting Nepean River tailwater.

3.2.5 Panthers Precinct Master Plan – Flood Assessment Report (2016)

The *'Panthers Precinct Master Plan – Flood Assessment Report'* was prepared by J. Wyndam Prince for Panthers Group in 2016. The report was prepared to inform and support the proposed Master Plan for the Panthers Precinct and to support future development applications for the Panthers site. The Panthers Precinct extends across a 51-hectare area that is contained within the Peach Tree and Lower Surveyors Creek study area.

A TUFLOW hydraulic model was developed as part of the study to assist in defining flood behaviour across the Panthers precinct. The TUFLOW model developed for the study includes the following features:

- The model covers a 1.9 km^2 area that is primarily contained within the Peach Tree and Lower Surveyors Creek study area. The extent of the TUFLOW model area is shown in **Figure A2** in **Appendix A**.
- The model uses a 4m grid size to define the variation in topography and hydraulic roughness.
- The terrain representation is primarily defined based upon detailed ground survey undertaken by Freeburn Surveyors. The extent of the Freeburn Surveyors topographic survey is shown in **Figure A5** in **Appendix A**. It is considered that the Freeburn Surveyors information represents the best available topographic information covering this section of the catchment.
- Outside of the Freeburn Surveyors topographic information, the terrain representation is largely based on 2002 ALS information. However, some modifications to this terrain representation where completed to reflect significant topographic modifications that have occurred since 2002. The most notable of these is the reconstruction of the Jamison Road / Mulgoa Road intersection in 2011.
- Jamison Creek east of Mulgoa Road and the eastern section of Showground Creek were modelled as 1D channels. All other watercourses were modelled in 2D.
- A full representation of the stormwater drainage network is included as 1D elements. The stormwater system representation is very detailed and provides a significant amount of additional information on top of that which is readily available to Council.
- Most culverts were included in the model as 1D structures (e.g. Mulgoa Road on Showground Creek, Mulgoa Road on Jamison Creek). However, other culverts (e.g. Ski Lake Road culverts, Jamison Road, local access road crossing along Showground Creek) were included in the model as 2D structures.
- Upstream inflows were applied to the TUFLOW model based on previous studies:
- Peach Tree Creek and Nepean River inflows were extracted from a regional RMA-2 model of the Hawkesbury-Nepean river system.
- Jamison Creek and Showground Creek inflows were extracted from the Peach Tree Flood Study (PWD, 1994). The timing of the inflows was adjusted so it coincided with the peak Peach Tree Creek / Nepean River flows.
- Inflows across the local subcatchment were represented using flows extracted from an XP-RAFTS hydrologic model that was developed specifically for the study.
- Downstream boundary conditions for Peach Tree Creek for the 1% AEP and 0.5% AEP events were defined based upon stage hydrographs extracted from the regional RMA-2 model. For the 5% AEP event, a static tailwater level of 23.0 m AHD was adopted based on information contained in the 'Peach Tree Flood Study' (PWD, 1994).

Like the *'Peach Tree Creek Flood Study'* (Public Works, 1994), the study recognised the potential for inundation of the lower Peach Tree Creek catchment from both local catchment runoff as well as the Nepean River. In recognition of the potential for different flooding mechanisms to impact on the precinct, a number of different local catchment and Nepean River flood scenarios were simulated. This included:

1) 1% AEP Nepean River flood with no local catchment inflows
2) 0.5% AEP Nepean River flood with no local catchment inflows
3) 1% AEP Nepean River flood with 5% AEP local catchment flood

4) 5% AEP Nepean River flood with 1% AEP local catchment flood

In each of the above scenarios it was assumed that the peak local catchment inflows occurred at the same time as peak levels within the Nepean River. Like the *'Peach Tree Creek Flood Study'* (Public Works, 1994), this study determined that flooding from the Nepean River generated higher peak flood levels along Peach Tree Creek during floods equal to and greater than the 1% AEP event relative to local catchment runoff.

The TUFLOW model was used to simulate design flood behaviour under three different development scenarios:

- Existing (i.e., 2016) development conditions;
- Development approved to date (i.e., 2016); and
- Future Development (including development approved to date together with anticipated future development across the remainder of the Panthers Precinct)

The results of the modelling showed that the future development across the Panthers precinct would produce negligible changes in flood behaviour during events up to and including the 1% AEP event. Some very small increases in flood levels (i.e. 20mm) were predicted during the 0.5% AEP event.

It is noted that the TUFLOW model was not calibrated against historic flood information. However, the TUFLOW modelling was reviewed by Worley Parsons who completed a similar assessment using the regional Nepean River RMA-2 flood model. The review determined that both models predict equivalent flood behaviour and impacts along Peach Tree Creek. Therefore, although the model was not calibrated, the fact that both models produced similar results provides increased confidence that the models are providing reliable estimates of flood behaviour.

As with the *'Peach Tree Creek Flood Study'* (Public Works, 1994), the TUFLOW model provides negligible information on design flood behaviour east of Mulgoa Road. Nevertheless, it is considered that this TUFLOW model provides the best available flood-related information for areas to the west of Mulgoa Road and that many components from this model can be used to assist in the development of the hydraulic model for the current study, subject to suitable data sharing arrangements between Council and Panthers Group.

3.2.6 Hydrology and Drainage 20% Detailed Design Report – Jane Street and Mulgoa Road Infrastructure Project (2017)

The *'Hydrology and Drainage 20% Detailed Design Report - Jane Street and Mulgoa Road Infrastructure Project'* was prepared by SMEC for Roads and Maritime Services in 2017. The report documents the detailed drainage design of the Jane Street and Mulgoa Road Infrastructure Project (JSMR). The JSMR aims to alleviate congestion and improve traffic flow along Mulgoa Road and Castlereagh Road adjacent to Penrith's CBD by widening the corridor from south of Union Road to south of Museum Drive. To accommodate the widening, the existing rail underbridge over Castlereagh Road will be replaced and three intersections will be upgraded. The existing underbridge at Castlereagh Road is a known flooding hotspot in the area.

The key drainage design features of the JSMR include:

- Pits and pipes used to drain runoff from the road pavement
- A pump mechanism solution has been proposed to drain the flow from the sag section near the Castlereagh Road underbridge into Peach Tree Creek. This will minimise the surcharging of the stormwater network at the underbridge.

The area of the JSMR is significantly flood affected and flooding can occur from local flooding from the Penrith CBD catchment, backwater from Peach Tree Creek and/or overland flooding from Nepean River. The Castlereagh Road underbridge is the most critically flood affected area. Accordingly, the flood mechanisms in the area were investigated as part of this study.

The flood impact assessment of the proposed design was based on modified/updated versions of the Arup Peach Tree Creek TUFLOW model and the Arup Penrith CBD TUFLOW model. The Penrith CBD TUFLOW model is discussed in detail in Section 3.2.2 and is a 2m resolution 1D/2D model developed in 2014.

The Peach Tree Creek TUFLOW model was developed by ARUP based on the 2014 flood study model created by Lyall and Associates, and is a 2m resolution, 2D only TUFLOW model primarily purpose to develop mainstream flood results. It applies the following model boundary conditions:

- DRAINS hydrological model for the local Peach Tree Creek catchments;
- Upstream Peach Tree Creek inflows extracted from the Peach Tree Flood Study (1994) FPLAIN model;
- Downstream boundary conditions extracted from the Nepean River Green Bridge Hydraulic Investigation (2014) TUFLOW model.

SMEC undertook the following updates to the TUFLOW models:

- Additional survey of the drainage infrastructure was undertaken in the vicinity of the Main Western Railway Line underbridge on Castlereagh Road. Updated pipe details were incorporated into the TUFLOW model in this area.
- A more refined DRAINS model, with high resolution definition of sub-catchments for each proposed stormwater inlet pit, was developed for the purposes of the 20% drainage design. The outputs from this DRAINS model were included in the model for model of the 20% detailed design scenario.
- The TUFLOW timestep of the Penrith CBD TUFLOW model was reduced to 0.5 seconds for the 2D domain and 0.25 seconds for the 1D domain.

From a review of the 'Peach Tree Creek Flood Study' (1994), SMEC determined the critical duration storm for Peach Tree Creek and its tributaries to be a 90 minute storm for the 20 year and 100 year ARI events. A 120 minute duration storm was adopted for the 5 year ARI event. The 100 year ARI planning levels in lower reaches of Peach Tree Creek were derived from the 20 year ARI local Peach Tree Creek event in combination with a 100 year ARI Nepean River tail water. Nepean River levels were based on a results for a critical duration for the Nepean River of 72 hours.

Based on the results of the TUFLOW modelling, SMEC determined that flooding in the area of Castlereagh Road is the result of Peach Tree Creek flood levels drowning the outlet of the 375mm pipe currently draining the sag point of the road at the underbridge. Consequently, a pumping solution for the Castlereagh Road Underbridge was recommended to separate flood levels at the underbridge from impacts of the high tailwater in Peach Tree Creek. To inform the appropriate pump size, a conceptual pump of infinite capacity was modelled in TUFLOW in conjunction with upgraded pipe connections to the pump from the underbridge. The pump system has been designed to convey the 10 year ARI flow (2,000l/s) and transfer flows to Peach Tree Creek.

3.2.7 Other Flood Reports

A range of other flood reports were provided by Council to assist in the preparation of the flood study. The other flood reports generally did not contain information specific to the study area or contained data that was superseded by more recent studies. However, the reports did occasionally contain information that could be used to assist in the development of the flood models and/or verify the results produced by the models developed for the current study. The additional reports included:

- *'Surveyors Creek / Peach Tree Creek Hydraulic Study'* (Lyall & Macoun, 1991): This study formed the basis for the subsequent *'Peach Tree Creek Flood Study'* (Public Works, 1994). Although a considerable amount of design flood information is contained in this report, it is largely superseded by the 1994 study.
- *'Peach Tree Creek / Showground Creek Flood Study'* (Lyall & Macoun, 1994): This study builds upon the flood modelling completed as part of the *'Peach Tree Creek Flood Study'* (Public Works, 1994) to provide an improved description of flood behaviour in the vicinity of the Mountain View Retirement Village. The existing model was updated to include an improved representation of Showground Creek. The creek representation was largely based on cross-sections extracted from contour plans. Therefore, it is considered that the cross-sections surveyed as part of the *'Penrith CBD Detailed Overland Flow Flood Study'* (Cardno, 2015) provide a better description of the conveyance characteristics of this channel.
- *'Surveyor Creek, Glenmore Park – Concept Design Report'* (GHD, 1994): summarises the outcomes of flood and water quality modelling to support the design of detention basins, creek channels, road crossings and wetlands across the upper sections of the Surveyors Creek catchment (i.e., south of the M4 motorway). It builds upon previous computer flood modelling documented in the *'Surveyors Creek, Glenmore Park – Plan of Management'* (Land Systems EBC, 1993). The study is focussed on areas to the south of the current study area. Therefore, it does not contain any information specific to the study area. Nevertheless, peak flow estimates documented in this report could be used to assist in validating the results generated by the hydrologic model developed for the current study.
- *'Panthers Redevelopment Project – Flood Study: Buildings Extensions and Carpark at Club Building'* (Lyall & Macoun, 1995): summarises the outcomes of flood modelling investigations to support the extension of the main Panthers building as well as car park expansions. A review of the plans contained in the report against recent aerial imagery indicates that these modifications were not implemented. Therefore, this report provides limited value to the current study.

- *'Glenmore Park Stages 8DEFGHJ: Eastern Branch of Surveyors Creek West Arm – Trunk Drainage Design Report'* (J. Wyndam Prince, 1999): presents the results of a stormwater drainage design for a branch of Surveyors Creek located south of Glenmore Parkway. This channel segment is located approximately 1 km south of the M4 motorway. Furthermore, no design discharges are documented in the report. Therefore, this report is of limited value to the current study.
- *'ESQ1818 Development Panthers Site – Flood Impact Assessment and Water Quality Management Report'* (J. Wyndam Prince, 2016): quantifies the potential flood and water quality impacts associated with the proposed "ESQ1818" development across the northern section of the existing Panthers site. This development was included in the future development scenarios documented in the *'Panthers Precinct Master Plan – Flood Assessment Report'* (J. Wyndham Prince, 2016). Therefore, the report provides limited additional flood information.
- *'Nepean River at Penrith Flood Study'* (NSW Department of Land & Water Conservation, 1997): provides a detailed 2-dimensional hydraulic assessment of flood behaviour along the Nepean River at Penrith. The study provides design flood information for events ranging between a 10% AEP flood and the PMF. Although a considerable amount of information was provided in this report, the results are superseded by the 'Nepean River Flood Study: Exhibition Draft Report' (Advisian, 2017).

3.3 Hydrologic Data

3.3.1 Rain Gauge Data

A number of daily read and continuous (i.e., pluviometer) rainfall gauges are located near the catchment. The location of each gauge is shown in **Figure 2**. Key information for each gauge is summarised in **Table 1**.

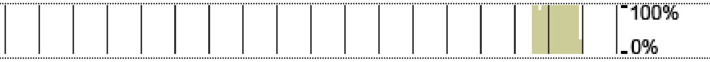
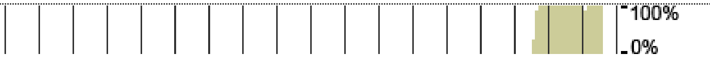
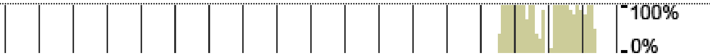
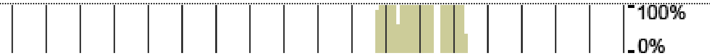
The information provided in **Table 1** indicates that daily rainfall records in the vicinity of the study area are available dating back to 1880 (Emu Plains gauge). However, continuous rainfall records are only available from 1996 onwards (Penrith Lakes AWS).

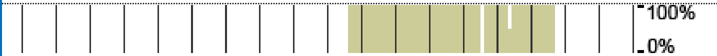
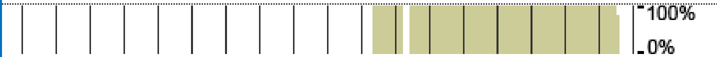
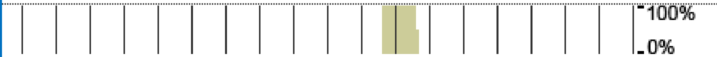
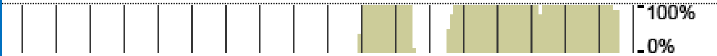
3.3.2 Stream Gauge Data

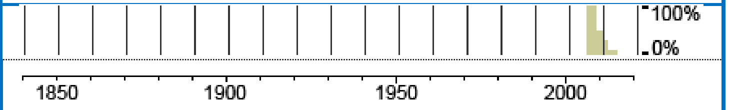
Figure 2 also shows the location of stream gauges located in the vicinity of the Peach Tree and Lower Surveyors Creek study area. Key information for each gauge is summarised in **Table 2**. As shown in **Figure 2**, there are no stream gauges located within the study area.

However, there is a stream gauge located on the Nepean River immediately adjacent to where Peach Tree Creek flows into the Nepean River. Accordingly, information for this stream gauge may assist in defining downstream boundary conditions as part of the historic flood simulations.

Table 1 Available rain gauges in the vicinity of the Peach Tree and Lower Surveyors Creek catchment

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Centroid of Catchment (km)	Temporal Availability and Percentage of Annual Record Complete
67096	Penrith (Glenroy)	Daily	BOM	1917 Jan	1923 Dec	1.5	
567163	Regentville Rural Fire Service	Continuous	SW	1992 Sep	2015 May	2.0	
567158	Orchard Hills (Kingswood Road Reservoir)	Continuous	SW	1991 Aug	2015 May	2.5	
67115	Glenmore Park (Cartwright Cl)	Daily	BOM	1995 Jan	2009 Apr	2.8	
567082	Orchard Hills (Orchard Hills WTW)	Continuous	SW	1991 Aug	2015 May	2.9	
67084	Orchard Hills Treatment Works	Daily	BOM	1970 Dec	2015 Aug	2.9	
67018	Penrith Ladbury Avenue	Daily	BOM	1890 Jan	1995 Oct	3.0	
67067	Emu Plains	Daily	BOM	1911 Jan	1996 Dec	3.9	
67004	Emu Plains	Daily	BOM	1880 Jan	1973 Jun	4.0	
567156	Orchard Hills (Flinders AV)	Continuous	SW	1991 Aug	2015 May	6.3	
67113	Penrith Lakes Aws	Daily	BOM	1995 Sep	2015 Nov	6.7	
		Continuous	BOM	1996 Jan	2015 Nov		
63185	Glenbrook Bowling Club	Daily	BOM	1963 Jan	2013 Jul	6.9	
63206	Wascoe	Daily	BOM	1903 Jan	1911 Dec	7.0	
67024	St Marys Bowling Club	Daily	BOM	1897 Jul	1984 Dec	7.0	
567159	Mount Pleasant (Cranebrook Reservoir)	Continuous	SW	1991 Aug	2015 May	7.1	
67102	St Clair (Juba Close)	Daily	BOM	1985 Sep	2013 Jul	8.4	
67025	St Marys Mwsdb	Daily	BOM	1947 Feb	1973 Apr	8.6	

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Centroid of Catchment (km)	Temporal Availability and Percentage of Annual Record Complete
567087	St Marys STP	Continuous	SW	1990 Jan	2015 May	8.7	
67003	Colyton (Carpenter St)	Daily	BOM	2000 Oct	2008 Feb	8.8	
63230	Blaxland Western Highway	Daily	BOM	1968 May	1980 Sep	9.1	
67083	Mount Druitt Francis St	Daily	BOM	1970 Dec	1976 Jan	10.1	
67066	Erskine Park Reservoir	Daily	BOM	2013 Jul	2015 Nov	10.4	
67068	Badgerys Creek McMasters	Daily	BOM	1936 Jan	1996 Dec	10.5	
67029	Wallacia Post Office	Daily	BOM	1943 Feb	2015 Sep	10.6	
67116	Willmot (Resolution Ave)	Daily	BOM	1995 Oct	2015 Nov	11.7	
63078	Springwood (Journeys End)	Daily	BOM	1946 Jan	1956 Jun	12.2	
63077	Springwood (Valley Heights)	Daily	BOM	1863 Jan	2017 Sep	12.5	
67002	Castlereagh (Castlereagh Rd)	Daily	BOM	1939 Sep	2015 Nov	12.6	
67050	Badgerys Creek School	Daily	BOM	1919 Jan	1929 May	12.8	
63272	Springwood (Euchora)	Daily	BOM	1885 Jan	1905 Dec	13.1	
63286	Winmalee (Pentlands Drive)	Daily	BOM	1985 Jan	2015 Mar	13.3	
67016	Minchinbury	Daily	BOM	1901 Feb	1970 Aug	13.3	
67108	Badgerys Creek AWS	Continuous	BOM	1996 Jan	2015 Nov	13.5	
63183	Valley Heights (Sun Valley Rd)	Daily	BOM	2002 Sep	2011 Oct	13.7	
67118	Oakhurst (Lawton Place)	Daily	BOM	1991 Mar	1999 May	13.9	
67000	Eastern Creek (Wonderland)	Daily	BOM	2000 Feb	2004 Feb	14.6	

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Centroid of Catchment (km)	Temporal Availability and Percentage of Annual Record Complete
67027	Warragamba	Daily	BOM	2005 Feb	2013 Mar	14.9	

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

Table 2 Available stream gauges in the vicinity of the Peach Tree and Lower Surveyors Creek catchment

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Centroid of Catchment (km)	Located within study area?
212201	Penrith Weir	River	SCA	1968 Jan		2.4	Nepean River immediately adjacent to study area
212219	Jellybean Pool	River	SCA	1990 Jan		6.9	Located upstream and west of study area (Nepean River catchment)
212048	Great Western HWY (DWR)	River	SWB	1986 Jan	1995 Jan	6.6	Located in separate catchment (South Creek)
2122971	Mandalong Park	River	SCA	1992 Jan		8.9	Located in separate catchment (South Creek)
212049	Debrincat Ave (DWR)	River	SWB	1986 Jan	1992 Jan	9.5	Located in separate catchment (South Creek)
2122002	Blacks Falls	River	SCA	1990 Jan		11.4	Located in Nepean River catchment downstream of study area
212202	Wallacia	River	SCA	1908 Jan		12.2	Located in Nepean River catchment upstream of study area
212240	Nepean Junction	River	SCA	1967 Jan		12.8	Located in Nepean River catchment upstream of study area
212404	Castlereagh (WQ)	River	MHL			13.4	Located in Nepean River catchment downstream of study area
212241	Warragamba Weir	River	SCA	1980 Jan		13.8	Located in Nepean River catchment upstream of study area
212218	Dodds Rock	River	SCA	1990 Jan		14.1	Located in Nepean River catchment upstream of study area
212320	Mulgoa Rd	River	DNR			14.2	Located in separate catchment (South Creek)

3.4 Topographic Information

The following topographic datasets were provided for use in defining the variation in ground surface elevations across the study area:

- 2011 Light Detection and Ranging (LiDAR) survey
- 2016 Penrith Lakes LiDAR survey
- 2002 Aerial Laser Survey (ALS)

Further detailed information on each topographic dataset is provided below.

3.4.1 2011 LiDAR Survey

LiDAR data was collected across Sydney in February 2011 by the NSW Government's Land and Property Information Department. This included the majority of the Peach Tree and Lower Surveyors Creeks study area. The extent of the 2011 LiDAR coverage is shown **in Figure A5** in **Appendix A**.

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 and provides an average of 1.65 elevation points per square metre.

A digital elevation model (DEM) was developed from the 2011 LiDAR information and is shown in **Figure 3**.

The LiDAR generally provides a good representation of the variation in ground surface elevations across the study area. However, these datasets 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, buildings) are not correctly removed from the raw dataset.

Plate 1 provides an example of the 2011 LiDAR point density in the vicinity of Anakai Drive. **Plate 1** shows a high LiDAR point density across grassed and paved areas but reduced ground points in the vicinity of dense trees / vegetation. **Plate 1** also shows no ground points across buildings. Therefore, it appears that non-ground points have correctly been removed from the 2011 dataset.

Nevertheless, the reduced point density shown in **Plate 1** means that there will be a less detailed representation of the variation in terrain in areas of dense vegetation. Unfortunately, many of the major conveyance areas in the study area (i.e., creek channels) include significant vegetation. Therefore, there is a significant chance that the LiDAR will not provide a reliable description of the flow carrying capacity of these major conveyance areas.

A review of the vertical accuracy of the 2011 LiDAR was completed by comparing the LiDAR elevations against detailed ground survey information. The detailed ground survey information was extracted from the stormwater survey discussed in Section 3.5. This review indicates that in areas of minimal dense vegetation, the average difference in elevation between the LiDAR and ground survey information was -0.004m with a standard deviation of

0.10 metres. Accordingly, this confirms that the LiDAR provides a good representation of the variation in terrain in areas not obscured by vegetation. However, in areas of dense vegetation, the average difference between ground survey and LiDAR survey elevations was in excess of 0.3 metres.

Plate 1 LiDAR data points (yellow crosses) in the vicinity of Anakai Drive

Given the LiDAR does not appear to provide a reliable representation of creek channels obscured by vegetation, it was considered necessary to collect additional survey information along these creeks as part of the project. Further information on the creek cross-section survey that was completed as part of the project is provided in Section 3.9.

It was also recognised that the LiDAR data will not pick up the details of drainage features that are obscured from aerial survey techniques, such as bridge and culvert dimensions. Although some bridge and culvert information is available from past studies and plans, there were some bridges and culverts where no detailed information was available. Therefore, survey of some bridges and culverts was also completed to ensure a reliable representation of these drainage structures were provided. Further details of the hydraulic structure survey is provided in Section 3.9.

3.4.2 2016 LiDAR Survey

LiDAR data was also collected across the Penrith Lakes area in 2016. This included a small section of the Peach Tree and Lower Surveyors Creek study area. The extent of the 2016 LiDAR coverage is provided in **Figure A5**. As shown in **Figure A5**, when the 2016 LiDAR is combined with the 2011 LiDAR, it provides a complete topographic representation of the study area.

A statistical assessment of the 2016 LiDAR was completed by Atlass (2016). This was completed using 50 ground survey points. This assessment determined that the average difference between the ground survey and LiDAR points was 0.00 metres with a maximum difference of 0.05 metres and a minimum difference of -0.07 metres. Accordingly, the 2016 LiDAR appears to afford an improved level of vertical accuracy relative to the 2011 LiDAR. Therefore, it is recommended that the 2016 LiDAR be used in preference to the 2011 data where data overlaps exist.

It should still be noted that the 2016 LiDAR is subject to the same limitations as the 2011 LiDAR. That is, it provides a less reliable description of the terrain in areas of dense vegetation and will need to be supplemented with creek cross-section survey to ensure these major conveyance areas are reliably defined.

3.4.3 2002 ALS

Aerial Laser Survey (ALS) was collected across the Penrith City Council LGA in 2002. This includes the full extent of the Peach Tree and Lower Surveyors Creeks study area (refer **Figure A5**). Specific metadata for the ALS could not be uncovered. Therefore, the horizontal and vertical accuracy of the data could not be confirmed. However, a review of roadway cross-section information indicates that the 2011 LiDAR provided a more reliable description of the roadway geometry relative to the 2002 ALS in areas not obscured by vegetation.

Moreover, as the ALS data was collected in 2002, it will not include any topographic modifications that have occurred since this date. As the 2011 and 2016 LiDAR data sets collectively provide a complete coverage of the study area and were collected more recently, it is recommended that they are used in preference to the 2002 ALS data to describe contemporary topographic conditions across the study area. However, the 2002 ALS may prove useful in replicating historic topographic conditions as part of the model calibration process.

3.5 Stormwater Survey

A detailed survey of the stormwater system contained within the Peach Tree and Lower Surveyors Creek study area was completed by ThinkSpatial on behalf of Penrith City Council. The survey includes all stormwater pit and pipes located in road reserves and drainage easements. This provides information on 3,144 stormwater pits and 3,392 stormwater pipes. The extent of the surveyed stormwater pits and pipes is shown in **Figure A4** in **Appendix A**.

A range of information was collected for each stormwater pit and pipe as part of the survey to ensure the flow carrying capacity of the stormwater system could be fully defined in the computer model. This included pit and pipe invert elevations, lintel and grate sizes as well as pipe sizes. The resulting pit and pipe layers were supplied in GIS format and could be modified to a format suitable for inclusion in the computer flood model.

The stormwater survey information was reviewed against the stormwater GIS layers as well as aerial imagery and it was determined to provide a sound description of the stormwater drainage system across the majority of the study area. The survey report that was supplied with the stormwater information noted that the stormwater system was incomplete in some areas. This included pit lids that could not be opened/lifted as well as pits and pipes on private property. In general, pipes draining out of private properties included the downstream pipe segment only with no information about upstream connections contained within private property (refer **Plate 2**).

Plate 2 Example of stormwater pipes on private property with no upstream connections/pits

There are also other instances of an incomplete/disconnected drainage system, as shown in **Plate 3**. However, a review of the notes included with the survey data indicates that these are old pipes that do not connect to any pits (i.e., they were decommissioned as part of previous stormwater upgrades). These pipes will need to be removed from the dataset before application to the TUFLOW model.

Plate 3 Example of incomplete drainage system on Mulgoa Road

The stormwater system review also determined that there were fourteen pipes with adverse slopes (pipes with a downstream invert elevation that was higher than the upstream invert). In general, the adverse slopes were mild (i.e., <-1%), but there were instances of more significant differences:

- Pipe 15919: downstream invert 0.54m higher than upstream invert;
- Pipe 14329: downstream invert 0.46m higher than upstream invert;
- Pipe 14037: downstream invert 0.41m higher than upstream invert;
- Pipe 13522: downstream invert 0.27m higher than upstream invert;

A review of each of the above pipes shows that they are generally located on major roads that were surveyed at night, which reduces the reliability of the survey. In such cases, the pit invert elevations are likely to be more reliable than pipe inverts. In most cases, if the downstream pit invert elevation was used instead of the pipe invert elevations, it would rectify (or significantly reduce) the adverse pipe slope issues.

With the exception of the limited issues outlined above, the stormwater survey information is considered to provide a reliable representation of the stormwater drainage system across the Peach Tree and Lower Surveyors Creek study area and is suitable for application to the computer flood model.

3.6 Geographic Information System (GIS) Data

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

- Aerial Photography – provides ortho-rectified aerial imagery collected in 2014 and 2016.
- Contours: provides ground surface elevation contours at 0.5 metre intervals derived from the 2002 ALS and 2011 LiDAR.
- Drainage Charted – shows the location of key components of the drainage system including open channels, stormwater pipes and pits, headwalls and culverts. The dataset was generally compiled from paper plans and maps and the accuracy and completeness of the dataset is unknown. The extent of the drainage information extracted from the plans and maps is shown in red in **Figure A4**. This information is considered to be superseded by the stormwater drainage survey information discussed in Section 3.5.
- Drainage Asset Survey – shows the location and properties of a selection of stormwater pits and pipes. The data was collected by Council’s asset department but only includes pits that were visible from the surface. The extent of the stormwater information collected as part of the asset survey is shown in blue in **Figure A4**. This information is considered to be superseded by the stormwater drainage survey information discussed in Section 3.5.
- Easements – shows the locations of drainage easement. The extent of the drainage easements is shown in **Figure 5**.

In general, the GIS layers provide a suitable basis for preparing report figures as well as informing the computer flood model development. However, as noted above, the quality of the drainage information is questionable and the detailed stormwater survey information described in Section 3.5 is considered more appropriate to use.

3.7 Remote Sensing

In addition to providing ground point elevations, the LiDAR also provides non-ground points (e.g., buildings, trees) as well as other information including point intensity and multiple return information. This information can be used with aerial photography to assist with the identification of different land uses across the catchment. This, in turn, can be used to assist in defining the spatial variation in different land uses across the catchment which can inform Manning's 'n' roughness coefficients and rainfall losses in the computer flood models.

This technique of land use classification was based on research documented in a paper prepared by Ryan titled *'Using LiDAR Survey for Land Use Classification'* (2013) and was applied based upon the 2011 LiDAR and 2014 aerial imagery. The classification algorithm divided the study area into the following land use classifications:

- Buildings;
- Water;
- Trees;
- Grass;
- Concrete; and
- Roads.

It should be noted that perfect accuracy cannot be expected from any automated classification, particularly when the LiDAR and aerial imagery date from different periods (i.e., 2011 & 2014). Errors can also arise due to shadowing effects. As a result, manual updates to the remote sensing outputs was completed to ensure a reliable representation of the spatial variation in land use was provided across the catchment.

The final remote sensing output is shown in **Figure 4**.

3.8 Engineering Plans

Penrith City Council provided design and work-as-executed plans for sixty drainage structures located within and upstream of the study area. The location of the drainage structures contained in the plans are shown in **Figure A1** in **Appendix A**.

The age and quality of the information contained in the plans is variable. In addition, details of most structures contained in the plans were collected as part of the stormwater survey described in Section 3.5. Accordingly, where stormwater survey information is available, it is preferable to use this dataset in preference to the information contained in the plans.

Nevertheless, some major drainage structures located upstream of the study area were not included as part of the stormwater survey. This includes the major culvert crossings of the M4 Motorway as well as outlet details for major upstream detention basins. Each of these structures will have a significant impact on flood conditions across the study area and it is considered important to include a presentation of these structures in the hydrologic model developed for the study. Therefore, the information in these plans was used to assist in deriving stage-storage and stage-outflow relationships in the hydrologic model.

Design plans of the Northern Road upgrade were also provided to Council from Roads & Maritime Services. The plans provide detailed design information for the upgraded roadway extending from south of the M4 Motorway north to the intersection of Jamison Road and the Northern Road. This includes details of all existing and proposed cross-drainage structures (e.g., culverts). Given the impending construction of these works, it is considered appropriate to include these drainage details in the design flood simulations but they should be omitted from the historic/calibration simulations.

3.9 Survey

3.9.1 General

To enable development of a computer model capable of providing reliable estimates of flood behaviour within the catchment, it was necessary to collect additional information describing major conveyance features including creeks, culverts and bridges. Consulting surveyors, Metropolis City Surveyors, collected the additional survey information.

Further information on the survey that was completed specifically for the project is presented below.

3.9.2 Creek Cross-Sections

As discussed in Section 3.4, LiDAR can provide a less reliable description of the variation in terrain in areas of dense vegetation, including the major creeks within the study area. Therefore, cross-sections were surveyed along each of the major creeks where vegetation was prevalent to ensure a reliable description of the conveyance capacity of these waterways could be provided in the computer model.

Cross-sections were collected at approximately 50 metre intervals along each creek. This resulted in the survey of fifty-seven (57) cross-sections. The location where cross-sections were surveyed is shown on **Figure 6**.

Photographs were also collected looking upstream and downstream of each cross-section to assist with defining Manning's "n" roughness coefficients in the computer model.

3.9.3 Hydraulic Structures

The details of forty (40) hydraulic structures (i.e., culverts and bridges) were also collected as part of the survey. The location of each structure that was surveyed is shown on **Figure 6**.

Key characteristics of each bridge were collected as part of the survey (e.g., pier sizes, bridge deck elevations, details of hand rails) as well as details of the creek channel directly below the bridge to ensure the conveyance capacity could be reliably defined.

Key characteristics of each culvert were also collected as part of the survey including invert elevations, culvert dimensions, roadway elevations as well as the details of any handrails. Cross-sections of the upstream and downstream channel were also collected to ensure potential hydraulic losses associated with flow contracting into and expanding out of the culvert could be defined in the computer model.

Photographs were also taken of each bridge and culvert to assist in defining Manning's "n" roughness coefficients in the computer model as well as the extent of any debris accumulation and blockage.

3.10 Community Consultation

3.10.1 General

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

Negligible historic flood information is available for the study area. However, it was considered that the community may be able to provide information of past floods to assist with the computer model validation. 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.

3.10.2 Flood Study Website

A flood study website was established for the duration of the study. The website address is: http://peachtree.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).

3.10.3 Community Information Brochure and Questionnaire

A community information brochure and questionnaire were prepared and distributed to all residential and business properties in the catchment. This resulted in brochures and questionnaires being distributed to approximately 7,700 addresses. 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 740 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 B1**, 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 at least 20 years.

- 20% of respondents have experienced some form of inundation or disruption as a result of flooding in the study area. This includes (also refer **Plate 4**):
  - 86 respondents have had their front or back yard inundated;

- 78 respondents have experienced traffic disruptions;
- 36 respondents have had their garage inundated; and,
- 9 respondents have had their house or business inundated above floor level.

The spatial distribution of respondents that have reported past flooding problems is shown in **Figure B1** in **Appendix B** (refer red dots).

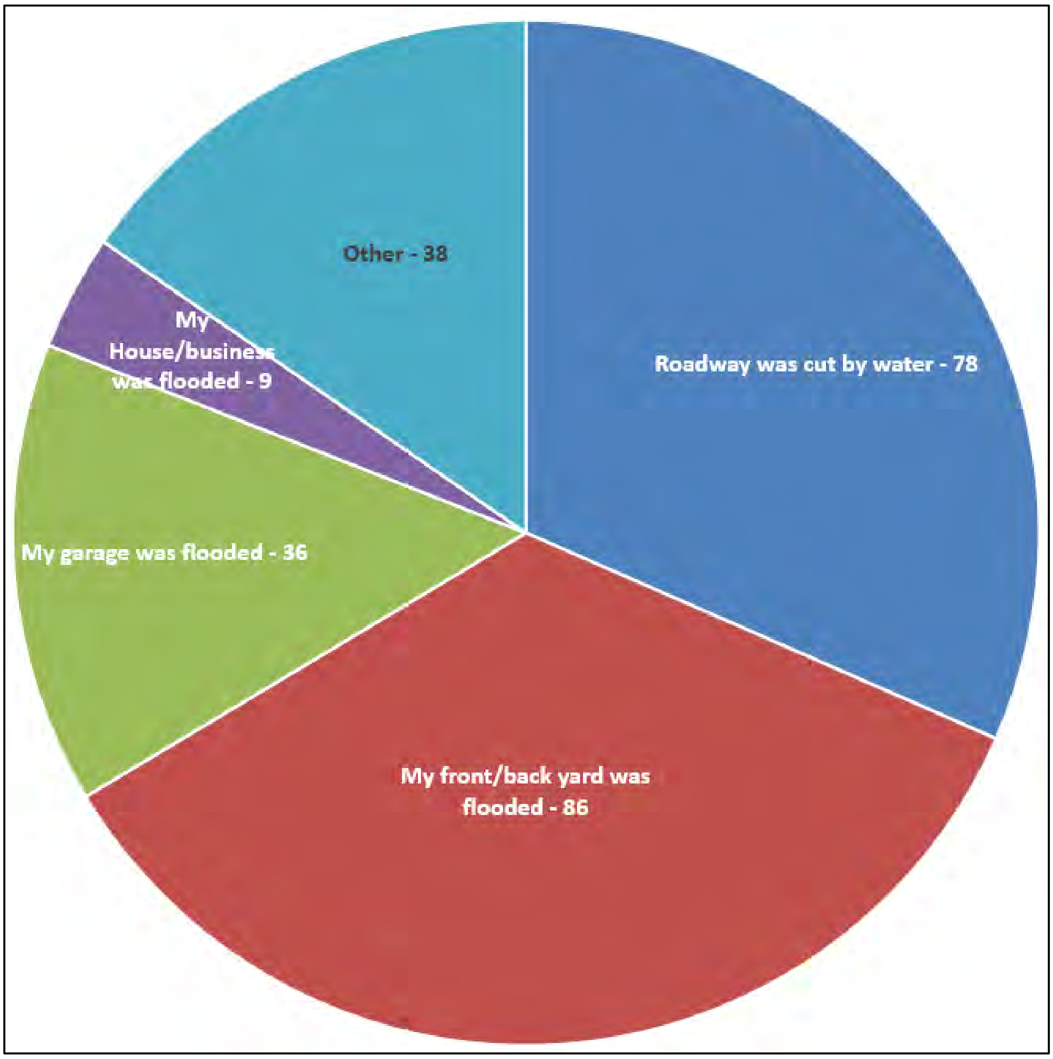


Plate 4 Type of Flood Impact Reported by Questionnaire Respondents

- Flooding problems were reported in the following streets in multiple questionnaire responses:
  - Anakai Street
  - York Road / Jamison Road
  - Evan Street
  - Ladbury Avenue
  - Teme Place
  - Henderson Crescent
  - Lyn Circuit

- Parsons Avenue
- Banool Avenue
- Smith Street
- Greenhills Avenue

- Several respondents noted that flooding can often occur in conjunction with sewer overflows.

- A number of respondents believe inundation in the catchment is exacerbated by:
  - Limited capacity of the exiting stormwater system (63 respondents)
  - Blockage of the creek, stormwater inlets and/or drains (49 respondents)
  - Insufficient creek capacity (37 respondents)
  - Overland flow obstructions (e.g., fences, buildings) (21 respondents)

A number of respondents provided photos of past floods. A selection of these photographs are provided in **Appendix C**. The photos generally show shallow depths of water across front and back yards as well as a number of streets. Anakai Drive, in particular, features in a large number of photographs.

A number of respondents provided information on floodwater depths and flow characteristics from past floods. Information on floods that occurred in January 2016 and February 2012 tended to be the most prolific and the information from these events is considered suitable for model calibration/validation purposes. However, those respondents that have lived in the area for a significant time claimed that the August 1986 event was the most significant flood in recent memory. Several respondents also recalled significant flooding in August 1990 and March 1978. The earliest report of flooding was from 1943.

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 discharges, flood level and flow velocity.

Two computer models were developed to simulate flood behaviour across the Peach Tree and Lower Surveyors Creeks catchment:

- A XP-RAFTS hydrologic model was developed to simulate the transformation of rainfall into runoff across the catchment; and,
- A TUFLOW hydraulic model was developed to simulate how the runoff would be distributed/move across the study area.

The following sections describe the model development process.

4.2 XP-RAFTS Model Development

4.2.1 Subcatchment Parameterisation

The Peach Tree and Lower Surveyors Creeks catchment was subdivided into 944 subcatchments based on the alignment of major streams, topographic divides and the location of key infrastructure (e.g., culvert crossings). The subcatchments were delineated with the assistance of the CatchmentSIM software using a 2 metre Digital Elevation Model (DEM). A total of 944 subcatchments were delineated. The subcatchment layout is presented in **Figure 7**.

Key hydrologic properties including area, impervious proportion, roughness and average vectored slope were calculated automatically for each subcatchment using CatchmentSIM in conjunction with detailed remote sensing land use information (refer Section 3.7). The spatial distribution of the different land use types is shown in **Figure 4**. As shown in **Figure 4**, the study area was subdivided into six different material types:

- Buildings;
- Water;
- Trees;
- Grass;
- Roadways; and,
- Concrete.

Percentage impervious and Manning's 'n' roughness values were assigned to each land use (refer **Table 3**) and were used to calculate weighted average percentage impervious and pervious 'n' values for each subcatchment. The adopted subcatchment parameters are summarised in **Appendix D**.

Table 3 Adopted Impervious Percentage and Manning's 'n' Values for Hydrologic Model

Land Use Description	Manning's 'n'	Impervious (%)
Buildings	0.025	100
Roads	0.018	100
Concrete	0.015	100
Trees	0.100	0
Water	0.030	100
Grass	0.035	0

Effective Impervious Area

Historically, impervious areas in hydrologic models were represented as the "total impervious area" (TIA). This concept assumes that with the exception of the initial wetting of the catchment, all impervious areas contribute fully to runoff. However, research dating back to the 1970s (e.g., Cherkaver, 1975, Beard and Shin, 1979) highlights the importance of using the "Effective Impervious Area" (EIA) in preference to the TIA to better account for impervious areas that are not directly connected to the drainage system (referred to as indirectly connected impervious areas).

An example of an indirectly connected impervious area is a foot path which is adjoined by a grassed area. In instances such as this, any runoff from the footpath will flow onto the grassed area and this runoff will have an additional opportunity to infiltrate into the underlying soils, thereby reducing the contribution of runoff.

Accordingly, Book 5 of ARR2016 advocates the use of EIA when modelling urbanised catchments to ensure urban runoff volumes and peak flows are not overestimated. Although ARR2016 presents a range of approaches for estimating the EIA, the most straight forward approach is estimating the EIA as a percentage of the TIA. Section 3.4.2.2 of Book 5 outlines that EIA will typically be 50% to 70% of the TIA. That is, only 50% to 70% of the total impervious area is directly connected to the drainage system. The remaining 30% to 50% of the impervious area is, therefore, indirectly connected and has additional infiltration opportunity.

For this study, the 70% adjustment factor (i.e., the most conservative factor) was initially trialled. However, verification of the runoff volumes generated using this adjustment factor relative to a "direct rainfall" TUFLOW model indicated that runoff volumes were being underestimated using this factor. Therefore, a range of alternate adjustment factors were trialled to better reflect the runoff volumes generated by the direct rainfall model. An adjustment factor of 85% was ultimately selected. That is, the total impervious areas that were calculated for each subcatchment were multiplied by 0.85 to develop a revised "EIA version" of the model.

4.2.2 Stream Routing

The sub-catchment area, roughness, slope and percentage impervious parameters that are input into the XP-RAFTS model are used by the model to estimate the transformation of rainfall excess into runoff for each subcatchment. In addition to local subcatchment runoff, most sub-catchments will also carry flow from upstream catchments along the main watercourses. The flow along the watercourses in XP-RAFTS is represented using a "link" between successive sub-catchment "nodes".

For this study, time delay lag routing was employed to represent the routing of runoff along the main watercourses into downstream sub-catchments. The time delay value for each stream segment was calculated by dividing the stream length by an average stream velocity. The average stream velocity was defined using peak design flow velocity grids that were generated as part of the *'Penrith Overland Flow Flood "Overview Study"'* (Cardno, 2006) for the 5% AEP, 1% AEP and PMF floods. Accordingly, the average velocity from the 5% AEP, 1% AEP and PMF floods was determined at each grid cell and then the average velocity contained within each subcatchment was calculated.

4.2.3 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 & Runoff' (Geoscience Australia, 2016). 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.

4.2.4 Detention Basins

The Peach Tree and Lower Surveyors Creeks catchment includes a number of detention basins. The basins are designed to attenuate downstream flows from the local catchment by temporarily storing runoff. Due to the potential for the basins to impact on downstream flows, they were incorporated as flood detention basins in the XP-RAFTS model.

The representation of flood storage basins in XP-RAFTS requires the storage characteristics of the basin to be defined. The storage characteristics were defined using a stage-storage relationship. The stage-storage relationships were developed from the 2011 LiDAR information. For those basins with a permanent water body, it was assumed that no storage is provided below the water surface. The location of each basin included in the XP-RAFTS model is shown in **Figure 7**.

4.3 TUFLOW Model Development

4.3.1 Model Extent

A 2-dimensional hydraulic computer model of the Peach Tree and Lower Surveyor Creeks catchment was developed using the TUFLOW software (version 2017-09-AC). 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 extent of the TUFLOW model area is shown in **Figure 8**. As shown in **Figure 8**, the TUFLOW model extends across the full extent of the catchment located north of the M4 Western Motorway. However, some sections of the catchment located south of the Motorway were also included to allow representation of:

- The potential interaction of floodwaters in flood storages located south of the Motorway.
- The potential for floodwater to "back-up" from the Nepean River along School House Creek and enter the study area through the Motorway culverts.

4.3.2 Grid Size

The TUFLOW software uses a grid to define the spatial variation in topography, hydrologic and hydraulic properties (e.g., Manning's 'n' roughness, rainfall losses) across the model 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 the 2011 and 2016 LiDAR data. Elevations in the vicinity of the Panthers World of Entertainment were defined based on detailed ground survey collected as part of the *'Panthers Precinct Master Plan – Flood Assessment Report'* (2016).

4.3.3 Manning's "n" Values

The TUFLOW software uses land use information to define the hydraulic (i.e., Manning's 'n') properties for each grid cell in the model. The remote sensing information described in Section 3.7 was used as the basis for defining the variation in land use across the TUFLOW model (refer **Figure 4**). This land use information, in turn, was used as the basis for assigning the variation on Manning's "n" roughness values across the model area.

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 and 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 or density, topographic irregularities and flow obstructions. All of these factors are typically aggregated into a single Manning's "n" value for each material type and representative 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 or vegetation height and the material is rigid (McCarten, 2011).

In an urban catchment, the depth of flow will frequently 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). Therefore, Manning's 'n' values obtained from literature are generally no longer valid for shallow flow depths.

Research completed by McCarten (2011) and others (e.g., Engineers Australia, 2012) indicates that Manning's "n" values will not be "static" and will vary with flow regime or 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 or 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 or obstruction height. The Manning's "n" calculations are included in **Appendix E** and the final Manning's 'n' values for each material type at each depth are summarised in **Table 4**.

Table 4 Depth Varying Manning's 'n' Roughness Values

Material Description	**Depth$_1$** *(metres)*	**n_1**	**Depth$_2$** *(metres)*	**n_2**
Buildings*	<0.3	0.03	>0.3	1.0
Water	0.03 for all depths			
Trees	0.10 for all depths			
Grass	0.035 for all depths			
Concrete	0.15 for all depths			
Roadways	0.18 for all depths			

NOTE: * please refer to section 1.2.7 for a more detailed description of building representation approach

As shown in **Table 4**, a constant Manning's "n" was applied to all land uses with the exception of buildings. The Manning's "n" value assigned to buildings was treated differently to the other land uses across the catchment. The main goal of the Manning's "n" value assigned to buildings was to represent the significant impediment to flow afforded by buildings. Further information on the representation of buildings in the model is provided in Section 4.3.7.

4.3.4 Creek Channels

Major conveyance areas that would not be well represented by the 2 metre grid or the DEM (e.g., narrow or heavily vegetated creek channels) were included within a 1-dimensional domain that was embedded within the 2-dimensional domain. The location of creek channels represented in 1D is shown in **Figure 8**.

The flow carrying characteristics of each creek channel segment are represented in TUFLOW using cross-sections. The creek cross-sections were sourced from the following:

- Peach Tree and Lower Surveyors Creeks: Surveyed cross-sections gathered by Metropolis City Surveyors (refer Section 3.9.2).
- Showground channel and Racecourse channel: surveyed creek information extracted from the *'Penrith CBD Detailed Overland Flow Flood Study' (2015)* and the *'Panthers Precinct Master Plan – Flood Assessment Report' (2016).*

The location of cross-sections that were included within the 1D domain is shown in **Figure 8**.

4.3.5 Culverts and Bridges

Culverts and bridges can have a significant influence on flood behaviour. Therefore, all bridges and culverts within the study area were represented within the TUFLOW model as 1D hydraulic structures. The location of culverts and bridges that were included within the TUFLOW model is shown in **Figure 8**.

For circular or rectangular culverts, the surveyed dimensions and invert elevations of the structures were included directly in the TUFLOW model. For irregular culverts (e.g., arch culverts), the shape of each crossing was defined using a flow height versus flow width relationship. The surveyed structure information was either extracted from the following sources (in order of priority):

- Structure survey (refer Section 3.9.3)
- Stormwater survey (refer Section 3.5)
- *'Penrith CBD Detailed Overland Flow Flood Study' (2015)*
- *'Hydrology and Drainage 20% Detailed Design Report - Jane Street and Mulgoa Road Infrastructure Project' (2017)*
- work-as-executed plans provided by Council (refer Section 3.8)

An entrance loss coefficient of 0.5 and an exit loss coefficient of 1.0 was adopted for all culverts.

The catchment also includes a number of bridge crossings. The available waterway area beneath the bridge deck was specified using a surveyed cross-section of the underlying channel. Energy losses were defined using a water height versus loss coefficient relationship that was developed based upon procedures outlined in *'Hydraulics of Bridge Waterways'* (Bradley, 1978). The bridge loss calculations are included in **Appendix E**.

4.3.6 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 stormwater system in the TUFLOW model to ensure the interaction between piped stormwater and overland flows was reliably represented.

The full stormwater system contained within the catchment was included within the TUFLOW model as a dynamically linked 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.

The properties of the stormwater system (e.g., pits types/sizes, pipe lengths/diameters) were defined from a number of different data sources. This included (in order of priority):

- Detailed stormwater survey (refer Section 3.5);
- *'Panthers Precinct Master Plan – Flood Assessment Report' (2016)*
- *'Penrith CBD Detailed Overland Flow Flood Study' (2015)*

When combined, these datasets provided a detailed description of the key attributes of most stormwater pits and pipes within the study area located on public land. The extent of the stormwater system included within the TUFLOW model is shown in **Figure 8**.

It was noted that some sections of the stormwater system were not complete. This was most common where pipe systems entered private property and no upstream pit was defined. In instances such as this, a grated inlet pit was included at the upstream end of each pipe. Where an actual pit was visible in the aerial imagery, the pit and upstream end of the pipe were moved to this location. When a pit was not visible (e.g., in areas of dense vegetation), the pit and pipe location was not altered.

Once all stormwater pits were included in the TUFLOW model, inlet capacity curves were prepared to define the pit inflow capacity with respect to water depth for each pit type. 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., grated, side entry, combination);
- The different topographic locations (e.g., sag or on-grade); and,
- The different grate dimensions and lintel sizes.

The inlet capacity curves that were developed for each pit type are provided in **Appendix H**. A total of 155 different pit inlet capacity curves were developed.

Hydraulic "losses" throughout the stormwater system were estimated using the Engelhund loss approach (BMT WBM, 2015). This loss approach automatically accounts for the following loss components at each stormwater pit for each model time step:

- Pit entrance loss;
- Loss associated with a drop in elevation between inlet and outlet pipes;
- Loss associated with a change in flow direction between the inlet and output pipes; and,
- Pit exit loss.

4.3.7 Building Representation

The Peach Tree and Lower Surveyors Creeks catchment incorporates significant urban areas. This urbanisation creates many overland flow obstructions. The most significant impediment to overland flow in an urban environment is buildings. Available research indicates that buildings have a considerable influence on flow behaviour in an urban environment by significantly deflecting flows irrespective of whether a 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 was represented as a complete flow obstruction. This is shown conceptually in **Plate 5**. The 0.3 metre height was selected as most houses within the study area include two steps up to the front door.

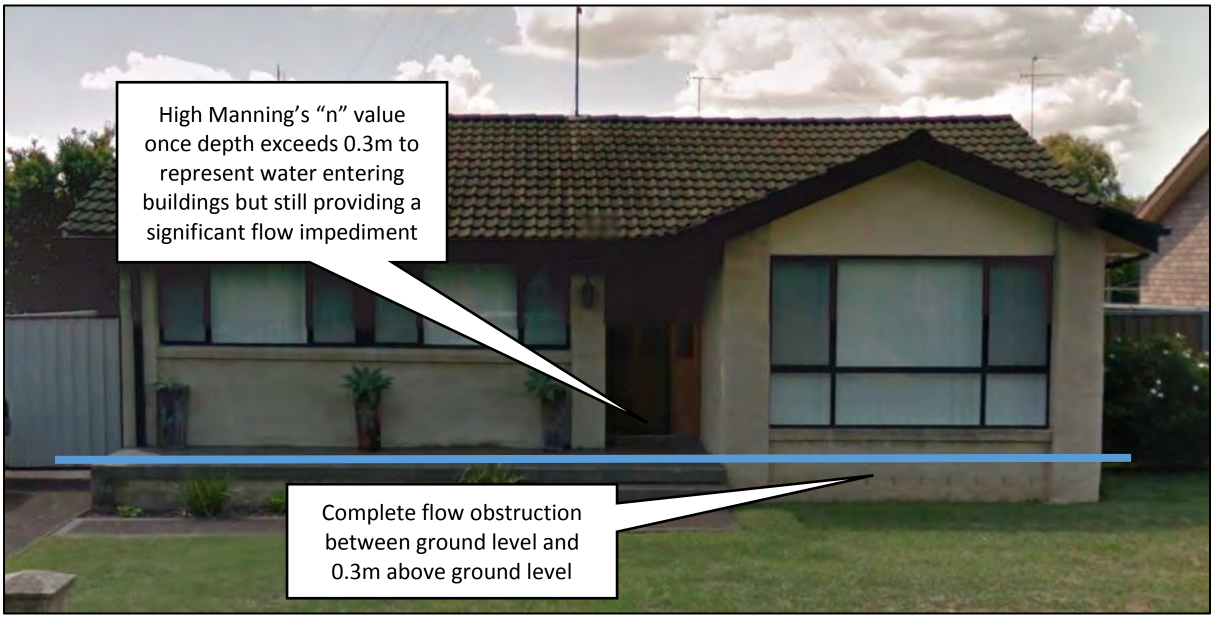


Plate 5 Conceptual representation of buildings in TUFLOW model

Once the water level exceeds the floor level of each building, it was allowed to "enter" the building. The floor level of each building was approximated by assuming the floor was elevated 0.3 metres above the adjoining ground elevation. A high Manning's "n" value of 10.0 was adopted to reflect the significant impediment to flow afforded by the many flow obstructions contained with a typical house (e.g., walls, doors, furniture etc). This is also shown conceptually in **Plate 5**.

4.3.8 Detention Basins and Water Storages

As discussed in Section 4.2.4, the Peach Tree and Surveyors Creek catchment incorporates a number of formal detention basins and water features (e.g., lakes in Cables Wake Park) that may attenuate downstream flows during rainfall events. Therefore, a representation of each storage was included in the TUFLOW model.

The absence of any water level monitoring gauges within each basin or dam means that the normal operating water level (or range of operating water levels) of each storage is not known. In the absence of any water level information, it was assumed that all "wet" water storages (e.g., farm dams) were full at the start of each simulated flood. No water was included within "dry" detention basins.

4.3.9 Pumps

Two pumps are installed to pump water from the low point in the Mulgoa Road underpass into the Penrith Homemaker Centre. No information could be uncovered regarding the capacity of each pump. Therefore, Penrith City Council provided photographs of the identification tags of each pump so that pump make/model could be identified in an effort to obtain pump capacity information from the pump manufacturer. The identification tags are shown in **Plate 6** and **7**.

Plate 6 Identification Tags for Pump #1

Plate 7 Identification Tags for Pump #2

The text included on the tags is difficult to read. However, the pump manufacturer was determined to the "Grundfos" and model numbers could be identified. However, pump curves for those particular pump makes and models were not available from the Grundfos website.

It was noted that the tag for pump #1 included specification of a power (5.5 KW) and a weight (17kg) (refer circled text in **Plate 6**). Therefore, pump curves from other pump manufacturers with similar power/weight characteristics were reviewed with the understanding that similar performance should be afforded. A pump curve for the Franklin Electric FPS-14-11 (5.5kW/16.6kg) was ultimately selected. The pump curve for this pump is included in **Plate 8**.

As shown in **Plate 8**, this pump provides a peak flow capacity of approximately 17.75 m^3/hour (0.3 m^3/s). It was assumed that both of the Mulgoa Road pumps had similar performance characteristics to those shown in **Plate 8** and "turned on" once the water depth in the underpass exceeded 0.05 metres.

The pumps were included within the TUFLOW model. The TUFLOW representation allows a pump discharge-head curve to be defined for each pump along with operational controls describing when the pumps turn on and off. As no operation controls were available for the pumps, it was assumed that the pumps would "turn" as soon as there was any water within the underpass.

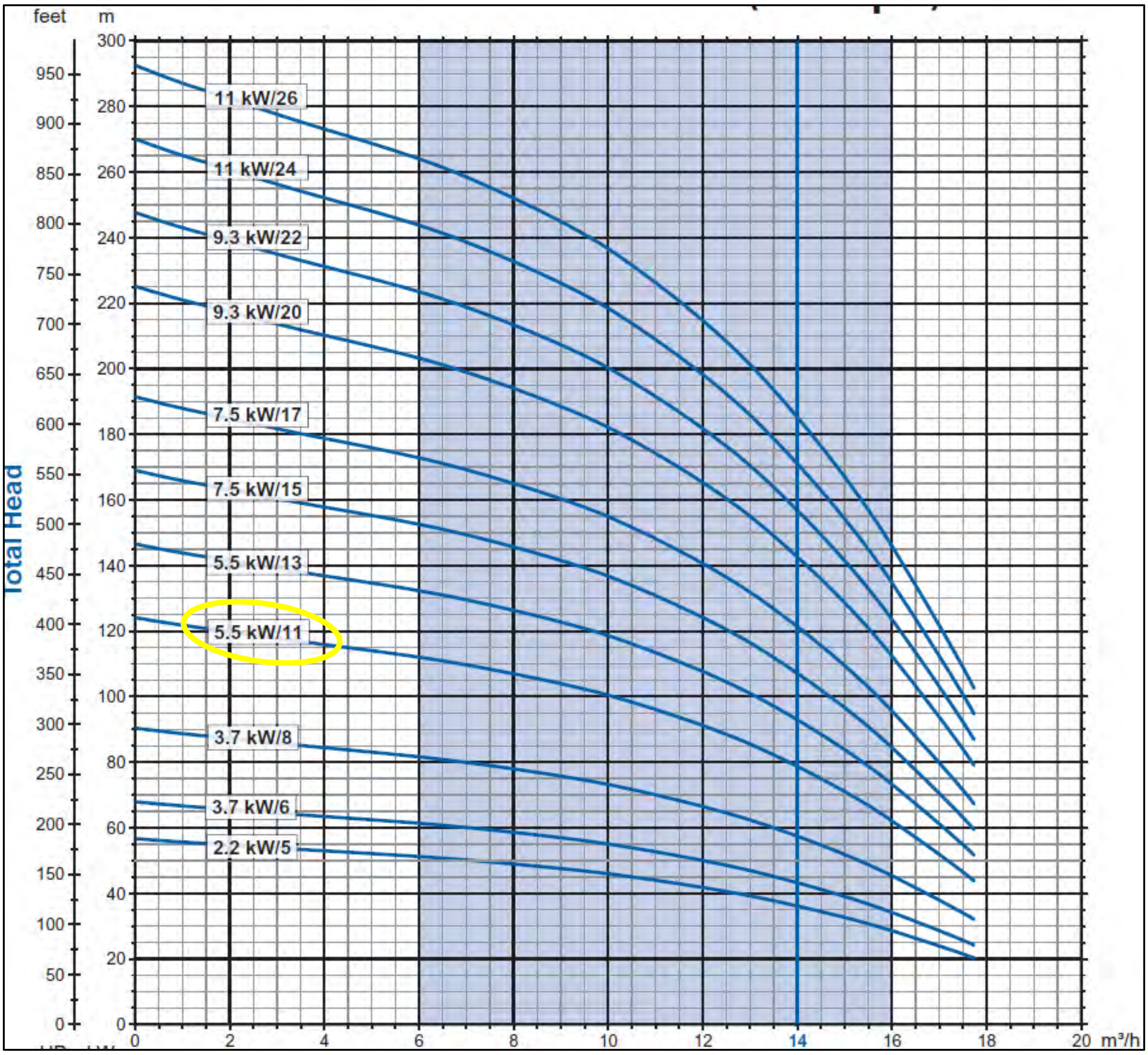


Plate 8 Adopted Pump Curve

No information was available describing where the water from the underpass is pumped to. Therefore, it was assumed that the pumps discharged to an above ground basin contained located on Wolseley Street within the Homemaker Centre (located about 50 metres north-west of the underpass).

5 Computer Model Validation

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 as well as blockage of hydraulic structures. Accordingly, the model should be calibrated using rainfall, 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 the hydrologic model. Simulated flows are extracted from the model results at locations where recorded flow hydrographs 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. The calibrated flows from the hydrologic model are then routed through the hydraulic model and simulated flood levels and depths are compared against reported flood level depths from the historic flood. Again, the hydraulic model parameters are adjusted until the best correlation between simulated and observed flood behaviour is achieved.

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

However, descriptions of flood behaviour were provided by the community as part of the community consultation for a number of historic floods. This included descriptions of floodwater depths as well as photographs of past floods. Moreover, there are several rainfall gauges located within close proximity to the catchment. Therefore, it was possible to validate the performance of the computer model by routing recorded rainfall from the nearby gauges through the hydrologic model. The flows from the hydrologic model can then be routed through the hydraulic model and simulated floodwater depths can be validated against floodwater depths and flood photographs provided by the community.

A number of anecdotal reports of flooding were provided by the community as part of the questionnaire responses (refer Section 3.10). The February 2012 as well as the January 2016 floods were the events most frequently reported on by the community. Accordingly, these events were selected as the primary validation events.

A limited number of anecdotal reports of flooding were also provided for a flood that occurred in February 2006. However, as this event occurred over 10 year ago and significant changes have occurred across the catchment during this period, it was considered unlikely that catchment conditions at the time of this flood would be well represented by the models which have been developed to reflect contemporary catchment conditions. Nevertheless, an assessment of the 2006 event was included. However, a greater priority was placed on the 2012 and 2016 events.

Further information on outcomes of the model validation is provided below.

5.2 2012 Flood

5.2.1 XP-RAFTS Modelling

Rainfall

The February 2012 flood occurred as a result of rain falling over an extended period from about 10:30am on 9th February 2012 to 04:00am on 10th February 2012. The most intense period of rain occurred over a 6-hour period starting around 3:00pm on 9th February 2012.

Accumulated daily rainfall totals for each rainfall gauge that was operational during the 2012 event were used to develop a rainfall isohyet (i.e., rainfall depth contour) map for the event, which is shown in **Figure 9**.

The isohyet map indicates that there was some spatial variation in rainfall across the catchment during the 2012 event. It indicates that between 96 and 140 mm of rain fell across the catchment, with greater depths of rainfall experienced in the northern portion of the catchment. In recognition of the variation in rainfall across the catchment during this event, the isohyets shown in **Figure 9** were used as the basis for defining spatially varying rainfall across the catchment as part of the 2012 flood simulation. Approximately 107mm of rainfall was applied across the XP-RAFTS model subcatchment located south of the M4 Motorway and about 117mm of rainfall applied across subcatchments located north of the Motorway.

The temporal (i.e., time-varying) distribution of rainfall was determined based on the closest active continuous rainfall gauge during this event. The closest continuous gauge with data for the 2012 event was determined to be the Regentville Rural Fire Service gauge (Gauge #567163), which is located immediately adjacent to the catchment. The location of the gauge is shown in **Figure 9** and the pluviograph for the gauge is presented in **Appendix I**.

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Appendix I** and indicates that the 2012 event was in the order of a 20% AEP design rainfall event.

Rainfall Losses

As discussed in Section 4.2.3, the initial-continuing loss model was employed to represent rainfall losses across the catchment. A continuing loss rate of 2.5 mm/hour based upon recommendations in *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987) was adopted. A review of rainfall records indicates that 14 mm of rain fell in the 24 hours prior to the main downpour. Therefore, no initial loss was applied to the model as the catchment would likely have been saturated prior to main rainfall event.

Results

The XP-RAFTS model was used to simulate rainfall-runoff behaviour for the 2012 flood based upon the rainfall and rainfall loss information presented in the preceding sections. This enabled discharge hydrographs to be generated for each subcatchment. A summary of peak discharges for each XP-RAFTS model subcatchment for the 2012 event are included in **Appendix J.**

The hydrographs generated by the XP-RAFTS model were subsequently routed through the TUFLOW model. Further discussion on the TUFLOW model simulation, are provided below.

5.2.2 TUFLOW Modelling

Inflow Boundary Conditions

As discussed, the XP-RAFTS model was used to simulate the transformation into runoff and generate discharge hydrographs at discrete locations across the full extent of the Peach Tree and Surveyors Creek catchment. The TUFLOW model extends across only the lower section of the Surveyors Creek catchment. Therefore, the total flows from the upstream sections of Surveyors Creek catchment as well as flows from the local subcatchments located north of the M4 Motorway must be accounted for. Accordingly, 'total' inflow hydrographs (i.e., hydrographs describing the total upstream contributing flow) were used to define the design inflows from those subcatchments located south of the M4 Motorway. In addition, 'local' discharge hydrographs (representing flows from the local subcatchments only) were also extracted from XP-RAFTS and were used to represent inflows for those subcatchment located north of the M4 Motorway. The local flow hydrographs were applied to the TUFLOW model at the outlet of each subcatchment.

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 condition is typically defined as a known water surface elevation (i.e., stage). The downstream boundary of the computer model is located at the confluence of Peach Tree Creek and the Nepean River. Accordingly, the water level across the downstream reaches of the catchment will be driven by the prevailing water level along the Nepean River at the time of the flood.

There is a stream gauge located on the Nepean River immediately adjacent to the Peach Tree Creek confluence (SCA Gauge 212201). Therefore, water level information from this stream gauge was extracted for the February 2012 event and was used to define the downstream boundary condition for the historic flood simulation.

Blockage

There is potential for blockage of stormwater inlets to occur during storms (refer **Plate 9**). Accordingly, blockage factors were assigned to all stormwater pits as part of the calibration simulations to reflect the reduced inflow capacity that would occur with partial pit blockage.

The stormwater pit blockage factors were assigned based on Council’s current pit blockage policy, which is summarised in **Table 5**. The pit blockage factors summarised in **Table 5** were applied for all validation and design flood simulations. The impact of no blockage as well as complete blockage of stormwater pits was assessed as part of the model sensitivity analysis.

Plate 9 View showing blockage of a stormwater pit

Table 5 Adopted Stormwater Pit Blockage Factors

Pit Type	Blockage Factor
Side entry (Sag)	20%
Grated (Sag)	50%
Combination (Sag)	Side inlet capacity only (i.e., complete blockage of grate)
Letterbox (Sag)	50%
Side entry (On-Grade)	20%
Grated (On-Grade)	50%
Combination (On-Grade)	10%

Comparison Between Observed and Simulated Flood Behaviour Results

Validation of the TUFLOW computer model was attempted based upon ten (10) anecdotal reports of flood behaviour for the 2012 event. In general, the anecdotal reports of flooding describe floodwater depths at discrete locations across the study area.

Peak floodwater depths were extracted from the results of the 2012 flood simulation and are included on **Figure 10**. A comparison between the peak flood depths generated by the TUFLOW and the flood depths reported by the community for the 2012 flood is also provided in **Figure 10**.

The flood depth comparison is also summarised in **Table 6**. The 'confidence level' that was reported by the community for each reported floodwater depth is also provided in **Table 6** and provides an indication of the flood depth reliability provided by the respondent, i.e.,:

- High = exact
- Medium = better than 0.1m
- Low = better than 0.5m.

Table 6 Comparison between simulated and observed floodwater depths for the 2012 flood

Response #	Reported Floodwater Depth* (m)	Confidence Level#	Simulated Floodwater Depth (m)	Difference (m)
1	>2.0	Medium	2.64	0.64
65	0.5	Low	0.26	-0.24
261	0.15-0.2	High	0.11	-0.04
317	0.04	Medium	0.03	-0.01
444	0.10	High	0.13	0.03
456	1.2	Medium	1.10	-0.10
467	0.12	-	0.11	-0.01
652	0.05	-	0.03	-0.02
654	0.08-0.12	High	0.08	0.00
671	0.30	Medium	0.22	-0.08

NOTE:
* Flood depths are based upon interpretation of photographs and flood descriptions provided by the community. Therefore, they should be considered approximate only.
# Flood depth confidence level is the confidence level reported by the community as part of the questionnaire responses.

As shown in **Table 6**, the majority of respondents reported either a medium or high level of confidence. A stronger emphasis was placed on reproducing floodwater depths that were reported with a high level of confidence. However, it was noted that some reports of flooding did not include an associated confidence level.

The flood level comparison provided in **Table 6** shows that the TUFLOW model is generally reproducing the reported depths of inundation to within 0.1 m. Some more significant differences were noted, including:

- Response #1: Difference is 0.64 metres. However, it was noted that the questionnaire response stated that the floodwater depth was at least 2 metres. Therefore, a precise flood depth is not provided for this flood mark and the simulated floodwater depth (2.64 metres) agrees with the reported flooding depth.

- Response #65: The response states that 0.5m of water covered the roadway. However, a precise location was not included. It is also noted that the respondent reported a "low" confidence level for this flood mark.

Overall, it is considered that the outcomes of the validation show that the TUFLOW model is providing a good reproduction of descriptions of flood behaviour for the 2012 flood.

5.3 2016 Flood

5.3.1 XP-RAFTS Modelling

Rainfall

The January 2016 flood occurred as a result of rainfall that commenced about 9:00am on 4th January 2016. During the following 56 hours, a total of 132mm of rain fell. The most intense downpour occurred over a 10-hour period commencing at 11:00 on 4th January 2016 (47mm of rain fell during this period).

Accumulated daily rainfall totals for each rainfall gauge that was operational during the 2016 event were used to develop a rainfall isohyet (i.e., rainfall depth contour) map for the event, which is shown in **Figure 11**. As there was minimal spatial variation in rainfall across the catchment during the 2016 event, a uniform rainfall depth of 140 mm was adopted.

The temporal (i.e., time-varying) distribution of rainfall was applied based on the closest continuous rainfall gauge. The closest continuous gauge with data for the 2016 event was determined to be the Regentville Rural Fire Service gauge (Gauge #567163). The location of the gauge is included in **Figure 11** and the pluviograph for the gauge is presented in **Appendix I**.

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Appendix I** and indicates that the 2016 event less severe than a 50% AEP design rainfall event.

Rainfall Losses

As discussed in Section 4.2.3, the initial-continuing loss model was employed to represent rainfall losses across the catchment. A continuing loss rate of 2.5 mm/hour based upon recommendations in *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987) was adopted. A review of rainfall records indicates that no rain fell in the 24 hours prior to the main downpour. Therefore, an initial loss of 10/1 mm was applied to the pervious/impervious subareas in the model as the catchment would likely have been dry prior to main rainfall event.

Results

The XP-RAFTS model was used to simulate rainfall-runoff behaviour for the 2016 flood based upon the rainfall and rainfall loss information presented in the preceding sections. This enabled discharge hydrographs to be generated for each subcatchment. A summary of peak discharges for each XP-RAFTS model subcatchment for the 2016 event are included in **Appendix J.**

5.3.2 TUFLOW Modelling

Inflow Boundary Conditions

As for the 2012 simulation, a combination of "local" and "total" flow hydrographs were extracted from the XP-RAFTS model and used to define inflow boundary conditions for the TUFLOW model. For further information on how inflows were applied to the TUFLOW model, please refer to Section 5.2.2.

Downstream Boundary Conditions

The Nepean River stream gauge located immediately adjacent to the Peach Tree Creek confluence (SCA Gauge 212201) was used to describe the time variation in water level at the downstream TUFLOW model boundary for the 2016 event.

Blockage

Partial blockage was assigned to all stormwater pits as part of the 2016 flood simulation in accordance with Council's blockage policy (refer Section 5.2.2).

Comparison Between Observed and Simulated Flood Behaviour Results

Validation of the TUFLOW computer model was attempted based upon twelve (12) reports of flood behaviour for the 2016 event. In general, the anecdotal reports of flooding describe floodwater depths at discrete locations across the study area. However, some floodwater depths were also estimated based upon photographs of the 2016 flood (refer **Appendix C**).

Peak floodwater depths were extracted from the results of the 2016 flood simulation and are included on **Figure 12**. A comparison between the peak flood depths generated by the TUFLOW model and the flood depths reported by the community and shown in flood photos for the 2016 flood is provided in **Figure 12**. The flood depth comparison is also summarised **Table 7**.

The flood level comparison provided in **Table 7** shows that the TUFLOW model generally provides a reasonable reproduction of recorded floodwater depths. In most cases, the TUFLOW model produces peak depths that are within 0.2 metres of recorded depths and levels. The only significant difference occurs, again, at response #1 where the respondent states that the floodwater depth was at least 2 metres. Therefore, it is considered that the simulated flood depth agrees with this reported flooding depth.

Overall, it is considered that the TUFLOW model is providing a reasonable reproduction of reported flood levels.

5.4 2006 Flood

5.4.1 XP-RAFTS Modelling

Rainfall

The February 2006 flood occurred as a result of an intense downpour that fell within a 4-hour period starting around 6:00pm on 26th February 2006. During this period, up to 130 mm of rain fell. Accumulated daily rainfall totals for each rainfall gauge that was operational during the 2006 event were used to develop a rainfall isohyet (i.e., rainfall depth contour) map for the event, which is shown in **Figure 13**.

Table 7 Comparison between simulated and observed floodwater depths for the 2016 flood

Response #	Reported Floodwater Depth* (m)	Confidence Level#	Simulated Floodwater Depth (m)	Difference (m)
1	>2.0	Medium	2.47	0.47
257	0.2	High	0.05	-0.15
261	0.15-0.2	High	0.10	-0.05
317	0.04	Medium	0.01	-0.03
436	0.1	High	0.03	-0.07
467	0.12	-	0.10	-0.02
545	0.30	-	0.13	-0.17
649	0.30	Medium	0.14	-0.16
656	0.30	Medium	0.15	-0.15
131	0.30	From photo	0.19	-0.11
211	0.20	From photo	0.13	-0.07
211	0.20	From photo	0.17	-0.03

NOTE:
* Flood depths are based upon interpretation of photographs and flood descriptions provided by the community. Therefore, they should be considered approximate only.
# Flood depth confidence level is the confidence level reported by the community as part of the questionnaire responses.

The isohyet map indicates that there was some spatial variation in rainfall across the study area during the 2006 event. It indicates that between 65 and 130 mm of rain fell across the catchment, with greater depths of rainfall experienced in the northern portion of the catchment. In recognition of the variation in rainfall across the catchment during this event, the isohyets shown in **Figure 13** were using as the basis for defining spatially varying rainfall across the catchment as part of the 2006 flood simulation. A total of 86mm of rainfall was applied in the XP-RAFTS model for those subcatchments located south of the M4 Motorway and 125mm of rainfall was applied to those XP-RAFTS subcatchment located north of the M4 Motorway.

The temporal (i.e., time-varying) distribution of rainfall was applied based on the closest continuous rainfall gauge. The closest continuous gauge with data for the 2006 event was determined to be the Regentville Rural Fire Service gauge (Gauge #567163), which is located immediately outside the catchment, on the southern side of the M4 Motorway. The location of the gauge is shown in **Figure 13** and the pluviograph for the gauge is presented in **Appendix I**.

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Appendix I** and indicates that that the 2006 event was between a 5% AEP and 2% AEP design rainfall event.

Rainfall Losses

The rainfall hyetograph presented in **Figure I3** in **Appendix I** indicates that the main down pour during the 2006 event was preceded by negligible rainfall. As a result, the catchment would have been relatively "dry" prior to the main rainfall event. Therefore, an initial loss of 10 mm/1mm was applied to pervious and impervious sections of the catchment to represent rainfall losses during with the initial "wetting" of the catchment.

A continuing loss rate of 2.5 mm/hour was adopted for pervious sections of the catchment and a continuing loss rate of 0 mm/hour was adopted for impervious areas.

Results

The XP-RAFTS model was used to simulate rainfall-runoff behaviour for the 2006 flood based upon the rainfall and rainfall loss information presented in the preceding sections. This enabled discharge hydrographs to be generated for each subcatchment. A summary of peak discharges for each XP-RAFTS model subcatchment for the 2006 event are included in **Appendix J.**

5.4.2 TUFLOW Modelling

Inflow Boundary Conditions

As for the 2012 and 2016 simulations, a combination of "local" and "total" flow hydrographs were extracted from the XP-RAFTS model and were used to define inflow boundary conditions for the TUFLOW model. For further information on how inflows were applied to the TUFLOW model, please refer to Section 5.2.2.

Downstream Boundary Conditions

The Nepean River stream gauge located immediately adjacent to the Peach Tree Creek confluence (SCA Gauge 212201) was used to describe the time variation in water level at the downstream TUFLOW model boundary for the 2006 event.

Blockage

Partial blockage was assigned to all stormwater pits as part of the 2006 flood simulation in accordance with Council's blockage policy (refer Section 5.2.2).

Comparison Between Observed and Simulated Flood Behaviour Results

Validation of the TUFLOW computer model was attempted based upon two (2) anecdotal reports of flood behaviour for the 2006 event. The validation was undertaken by routing the historic rainfall described in Section 5.3.1 through the TUFLOW model and comparing reported and simulated flood levels at each location.

Peak floodwater depths were extracted from the results of the 2006 flood simulation and are included on **Figure 14**. A comparison between the peak flood depths generated by the TUFLOW and the flood depths reported by the community for the 2006 flood is also provided in **Figure 14**. The flood depth comparison is also summarised in **Table 8**.

The flood level comparison provided in **Table 8** shows that the TUFLOW model generally provides a reasonable reproduction of recorded floodwater depths. In all cases the TUFLOW model produces peak depths that are within 0.08 metres of recorded depths and levels.

Table 8 Comparison between simulated and observed floodwater depths for the 2006 flood

Response #	Reported Floodwater Depth* (m)	Confidence Level#	Simulated Floodwater Depth (m)	Difference (m)
255	0.12	High	0.10	0.02
695	0.3	High	0.22	-0.08

NOTE:
* Flood depths are based upon interpretation of photographs and flood descriptions provided by the community. Therefore, they should be considered approximate only.
# Flood depth confidence level is the confidence level reported by the community as part of the questionnaire responses.

Therefore, it is considered that TUFLOW model is providing a good reported flood behaviour for the 2006 flood.

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 often expressed as an Annual Exceedance Probability (AEP).

The AEP of a flood flow or level or depth at a particular location is the probability that the flood flow or level or 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 be equalled or exceeded, on average, once in a 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 Peach Tree and Lower Surveyors Creek catchment are outlined in the following sections.

6.2 Hydrology

Design hydrology was defined as part of the flood study using the 2016 revision of Australian Rainfall and Runoff (ARR2016) (Geoscience Australia, 2016). The following sections describe each of the hydrologic inputs that were derived based upon ARR2016 as well as the outputs.

6.2.1 Rainfall

Revised design rainfall was established as part of the 2016 revision of Australian Rainfall and Runoff. The revised design rainfall takes advantage of more rainfall gauges and nearly 30 years of additional data, as well as more modern statistical analysis techniques. This provides an improved representation of design rainfall.

Point design rainfall depths were downloaded from the Bureau of Meteorology's 2016 IFD webpage. The design rainfall intensities were extracted at two locations in an attempt reflect any spatial variation in rainfall information:

- Centroid of catchment upstream of the M4: 33.7974°S, 150.6930°N
- Centroid of catchment downstream of the M4: 33.7632°S,150.6894°N

A copy of the design rainfall information downloaded from the Bureau of Meteorology's is included in **Appendix K**. The design rainfall information is also summarised in **Table 9** at the centroid of the catchment downstream (i.e., north) of the M4 motorway.

Table 9 Point Design Rainfall Depths north of M4 Motorway

Storm Duration	Rainfall Depth (mm)								
	50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.2%AEP	PMP
5 min	7.60	10.6	12.8	14.9	17.8	20.1	22.9	27.0	-
10 min	12.2	17.2	20.8	24.3	29.1	32.8	37.4	44.1	-
15 min	15.2	21.6	26.0	30.4	36.4	41.1	46.9	55.2	140
20 min	17.4	24.7	29.7	34.8	41.6	47.0	53.6	63.2	-
25 min	19.1	27.0	32.5	38.1	45.6	51.4	58.6	69.1	-
30 min	20.5	28.9	34.8	40.7	48.7	55.0	62.7	73.9	200
45 min	23.6	33.0	39.7	46.3	55.4	62.6	71.4	84.1	260
1 hour	25.9	36.0	43.1	50.3	60.2	68.1	77.6	91.5	310
1.5 hour	29.3	40.4	48.2	56.2	67.3	76.2	86.9	102	390
2 hour	32.0	43.9	52.3	61.0	73.0	82.7	94.3	111	460
3 hour	36.5	49.7	59.2	69.0	82.6	93.7	107	126	550
4.5 hour	42.1	57.2	68.1	79.4	95.2	108	123	145	-
6 hour	46.9	63.8	76.1	88.7	106	121	138	163	740
9 hour	55.2	75.5	90.3	106	127	144	164	194	-
12 hour	62.4	85.9	103	121	145	164	187	220	-
18 hour	74.3	104	125	147	176	199	227	267	-
24 hour	83.9	118	143	168	202	229	261	308	-
30 hour	92.0	131	159	187	225	254	290	341	-
36 hour	98.9	142	172	203	244	275	314	370	-
48 hour	110	159	194	230	275	310	353	417	-

It is noted that the Bureau of Meteorology's 2016 IFD webpage does not currently have design rainfall information published for events rarer than the 1% AEP for durations less than 24 hours. In the absence of this information, Section 3.6.3 of Book 8 of ARR2016 was used to derive rainfall estimates for these rarer events. This involved applying "growth curve" factors to the 1% AEP rainfall as follows:

- 0.2% AEP rainfall = 1.140 x 1% AEP rainfall
- 0.5% AEP rainfall = 1.344 x 1% AEP rainfall

As part of the flood study it was also necessary to define flood characteristics for the Probable Maximum Flood (PMF). The PMF is considered to be the largest flood that could conceivably occur across a particular area.

The PMF is estimated by routing the Probable Maximum Precipitation (PMP) through the XP-RAFTS model. The PMP is defined as the greatest depth of rainfall that is meteorologically possible at a specific location.

PMP depths were derived 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 GSDM PMP calculations are included in **Appendix K** and the calculated rainfall depths are summarised in **Table 9**.

6.2.2 Areal Reduction Factors

The design rainfall intensities presented in the preceding section are only applicable for catchment areas of up to 1 km^2. Therefore, ARR2016 includes areal reduction factors that recognise that there is unlikely to be a uniformly high rainfall intensity across all sections of large catchments.

The primary input variable to calculate the areal reduction factors is the contributing catchment area. One of the main difficulties in applying the areal reductions factors for a flood study such as this is the fact that the contributing catchment area varies considerably across the study area. For example, the contributing catchment areas vary from less than 1 km^2 at the upstream end of each major subcatchment (and smaller tributaries) up to 24.5 km^2 at the downstream end of the catchment. Therefore, to fully apply the correct areal reductions factors, it would be necessary to calculate the catchment area draining to the outlet of each subcatchment, determine the reduction factor for each subcatchment then adjust the point rainfall intensities individually for each subcatchment. This would result in a significant increase in the number of design storms that need to be simulated with associated increases in simulation times and processing effort. Therefore, it was considered more appropriate to select a single representative contributing catchment area to develop a single set of areal reduction factors for application to the study area.

As a first step, the subcatchments where the contributing catchment area was less than versus greater than 1 km^2 (i.e., the area threshold where reduction factors begin to be applied) was investigated. The outcomes of this assessment are presented in **Plate 10** with yellow subcatchments having a contributing catchment area less than 1 km^2 and red subcatchments having a contributing catchment area greater than 1 km^2.

The information presented in **Plate 10** shows that most subcatchments in the study area have a contributing upstream catchment of less than 1 km^2 (approximately 84% of the study area has a contributing catchment area of less than 1 km^2). Therefore, application of no areal reductions would be appropriate for approximately 84% of the study area.

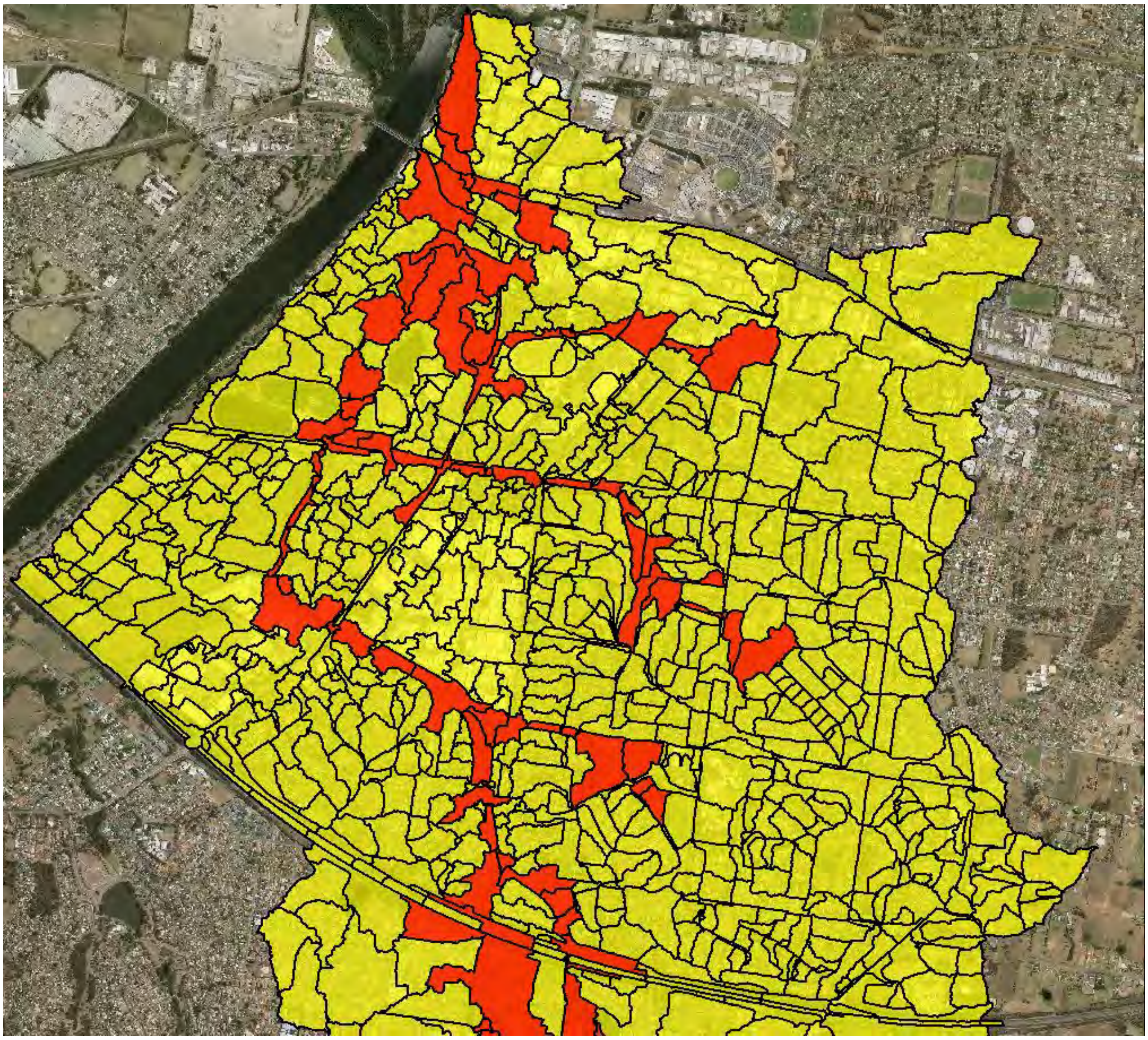


Plate 10 Subcatchments where the contributing catchment area is less than 1 km^2 (yellow) and greater than 1 km^2 (red)

Although application of no reduction factor across the downstream sections of the catchment will result in design discharges being overestimated, this is not considered unreasonable for the following reasons:

- The “worst case” flooding across the lower catchment areas is dominated by Nepean River inundation during large design floods. Therefore, Nepean River flooding is likely to “usurp” any relatively small overestimates of design discharges from the local catchment.
- The reduction factors for the critical durations would typically be less than 10% (and in most bases less than 5%). Therefore, the reductions would not change peak discharges along these waterways significantly.

Based on this assessment, it was considered reasonable to apply no areal reduction factors to the point rainfall intensities as the majority of the study area comprises catchments areas that are less than 1 km^2 and the reduction factors for other areas would be relatively small.

6.2.3 Rainfall Losses

ARR2016 introduced a revised approach for defining pervious rainfall losses for design flood simulations. Although the same initial/continuing loss approach is retained in ARR2016 that was specified inARR1987, ARR2016 employs a variable initial rainfall loss that varies accordingly to the storm severity and duration.

The ARR2016 initial rainfall losses are calculated by subtracting median pre-burst rainfall losses from the overall storm loss for the area. This aims to recognise that the most intense "downpour" is frequently preceded by rainfall that would serve to "wet" the catchment, thereby reducing the potential for rainfall during the main "burst" to infiltrate into the underlying soils (i.e., the median pre-burst rainfall depth is intended to reflect the "lead up" rainfall).

The overall storm loss data and pre-burst rainfall data for the study area was sourced from the ARR2016 data hub. The input data was derived at the centroid of the overall catchment (150.689^{O}E, 33.779^{O}S). The ARR2016 data hub download for this location is included in **Appendix K**.

An overall storm loss of 46 mm is defined for the study area by ARR2016. However, the ARR2016 data hub notes that this storm loss is for rural areas only and should not be applied in urban areas. A review of Section 3.5.3.2.1 of Book 5 of ARR2016 suggests that for catchments with an urban component, the pervious storm initial loss should be 60 to 80% of the rural storm initial loss to account for the reduced infiltration potential across catchments with an urban proportion (most notably from indirectly connected impervious areas). For this study, the 60% factor was adopted providing a storm initial loss of 27.6 mm (46mm x 0.6).

The adjusted storm initial loss is subsequently adjusted by subtracting a median pre-burst rainfall depth (which varies based on storm duration and AEP) from the adjusted storm loss. For example, the "burst" initial loss for the 1% AEP, 120-minute storm would be calculated as:

- Burst initial loss = adjusted storm initial loss – median pre-burst rainfall depth

  Burst initial loss = 27.6mm – 1.9mm

  Burst initial loss = 25.7mm

It was noted that no pre-burst rainfall losses are provided on the ARR2016 data hub for storm durations less than 1 hour. Therefore, it was assumed that the pre-burst rainfall losses for the 1 hour storm also applied for storm durations less than 1 hour. The resulting "burst" initial rainfall losses for the study area are summarised in **Table 10**. As shown in **Table 10**, "burst" initial rainfall losses vary between 7.5 mm and 27.6 mm. No preburst rainfall information is available for the 0.5% AEP and 0.2% AEP events. Therefore, the 1% AEP burst losses were also used for the 0.5% AEP and 0.2% AEP events.

Continuing loss rates are used in ARR2016 in a similar manner to how they were used in ARR1987. The ARR2016 data hub specifies a continuing loss rate of 3.4 mm/hour for the study area for rural areas. Again, ARR2016 only recommends applying this rainfall loss rate for rural catchments. For pervious and indirectly connected impervious areas within an urban proportion, Section 3.5.3.2.2 of Book 5 of ARR2016 recommends a continuing loss rate for south-eastern Australia of between 1 and 3 mm/hour (with a value of 2.5 mm/hour being recommended for most applications). The outcomes of the model validation (refer Section 5) as well as the calibration/validation results for other nearby catchments (e.g., Penrith CBD, College, Orth & Werrington Creeks) also suggests that a continuing loss rate of 2.5 mm/hour is appropriate for the area. Therefore, this continuing loss rate was also used as part of the current study for pervious and indirectly connected impervious areas.

Table 10 ARR2016 Initial Rainfall Losses

Storm Duration	Initial Loss (mm)						
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP up to 0.2% AEP	PMP
< 1 hour	25.9	26.2	26.4	26.7	25.0	23.7	0
1 hour	25.9	26.2	26.4	26.7	25.0	23.7	0
1.50 hour	27.0	26.6	26.4	26.1	26.5	26.7	0
2 hours	27.6	27.4	27.3	27.2	26.4	25.7	0
3 hours	25.9	24.4	23.5	22.6	24.0	25.1	0
6 hours	23.8	16.7	12.0	7.5	10.1	12.1	0
12 hours	26.1	21.8	19.0	16.3	10.8	6.6	0
18 hours	26.2	21.3	18.1	15.0	11.7	9.2	0
24 hours	27.6	23.5	20.8	18.2	16.2	14.6	0
36 hours	27.6	25.5	24.1	22.7	21.9	21.3	0
48 hours	27.6	27.6	27.6	27.6	27.1	26.8	0
72 hours	27.6	27.6	27.6	27.6	27.6	27.6	0

For impervious areas, Section 3.5.3.1.2 of Book 5 of ARR2016 recommends a storm initial loss of 1 mm and a continuing loss rate of 0 mm/hr. The continuing loss rate of 0 mm/hr was adopted directly, however, the storm loss of 1 mm again needs to be adjusted to a burst loss by subtracting the preburst rainfall. This yielded an impervious burst loss of 0mm for all storm durations.

For the PMP, Section 4.2.2.3 of Book 8 of ARR2016 recommends that for sub-humid areas of south-eastern Australia, a pervious burst loss of 0 mm should be adopted for shorter duration PMP events. Section 4.3.4.3 of the same book also recommends a pervious continuing loss rate of 1 mm/hr for south-eastern Australia. For impervious areas, the 0mm initial loss and 0 mm/hr continuing loss rate that was utilised for the other design flood simulations was also retained for the PMP simulations.

6.2.4 Temporal Patterns

One of the most significant differences between ARR2016 and ARR1987 is in the use of storm temporal patterns (i.e., the patterns describing the distribution of rainfall throughout the storm). ARR1987 used a single temporal pattern for each AEP/storm duration while ARR2016 uses 10 temporal patterns for each AEP/storm duration. Therefore, ARR2016 requires simulation of a minimum of ten times more storms than ARR1987.

The temporal patterns for the study area were downloaded from the ARR2016 data hub and were used to simulate the temporal distribution of rainfall for each design storm. In accordance with ARR2016 for catchments with an area less than 75 km^2, the "point" temporal patterns rather than "areal" temporal patterns were selected to describe the temporal variation in rainfall.

ARR2016 groups the temporal patterns into "frequent", "intermediate" and "rare" groupings, which were applied to each design storm as follows:

- Frequent temporal patterns: 50% AEP and 20% AEP
- Intermediate temporal patterns: 10% AEP and 5% AEP
- Rare temporal patterns: 2% AEP, 1% AEP, 0.5% AEP and 0.2% AEP

For the PMP, a single temporal pattern was adopted for each PMP storm simulation in line with the approach recommended in the *'Generalised Short Duration Method'* (GSDM) (Bureau of Meteorology, 2003).

6.2.5 Results

The XP-RAFTS model was subsequently used to simulate rainfall runoff processes for the complete suite of design storm. The design 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP storms were simulated using the XP-RAFTS model. The PMF was also simulated.

As outlined in Section 6.2.4, a suite of ten temporal patterns were used to represent the temporal variation in rainfall for each design flood frequency up to and including the 0.2% AEP and duration. The peak discharges from the full suite of temporal patterns for each design event were reviewed to determine the most representative temporal pattern for each storm duration. The temporal pattern that generated the peak discharge immediately above the median discharge was selected as the most representative temporal pattern for each subcatchment. This process was completed for all AEPs and storm durations. The peak discharges generated by the representative temporal pattern were then reviewed across all storm durations for a particular AEP and the storm duration that produced the highest peak design discharge was selected as the duration and critical discharge for a particular subcatchment.

The resulting critical storm durations and peak discharges for each subcatchment are presented in **Appendix L**. The results of the hydrologic analysis indicate that the critical durations across the study areas typically vary between 20 minutes (for smaller, urbanised subcatchments in the upper catchment areas) and 6 hours (for the lower catchment areas located west of Mulgoa Road). Peak flows were also extracted at key locations across the study area and are presented in **Table 11.**

Box plots were also prepared to better display the full range of results produced as part of the ARR2016 hydrologic analysis. The box plots are provided in **Appendix L** at key locations across the study area and present the following information:

Table 11 Peak Design Discharges at Key Locations

Location		XP-RAFTS ID	Peak Discharge (m^3/s)								
			50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.2%AEP	PMP
Surveyors Creek	Downstream of M4 Motorway	117	22.4	23.1	23.7	24.9	28.1	30.2	32.1	36.1	317.3
	Ikin Street	130	25.3	43.2	54.8	68.0	81.3	91.6	103.4	126.3	808.3
	Mulgoa Road	159	32.0	53.1	70.3	85.3	98.6	119.1	137.0	166.0	984.3
	Peach Tree Creek Confluence	181	37.8	55.6	76.3	90.0	109.7	128.9	154.2	179.5	1053
Peach Tree Creek	Jamison Road	211	39.9	58.0	82.1	96.0	120.1	135.1	161.4	194.3	1113
	Great Western Highway	310	56.1	93.9	115.1	136.3	182.5	217.7	252.9	308.1	1465
	Nepean River Confluence	1	58.3	100.9	120.9	141.8	185.7	228.2	270.3	324.7	1529
Racecourse Channel	The Northern Road	301	1.2	2.2	2.7	3.7	5.3	6.9	8.0	10.4	36.9
	Greenway Drive	326	3.7	5.3	7.2	10.6	13.5	17.1	21.9	25.8	95.9
	Evan Street	165	4.6	6.9	9.3	11.9	13.9	16.5	20.4	25.2	84.1
	Racecourse Road	317	5.5	8.7	11.5	14.9	19.8	23.5	28.3	35.0	127.8
	York Road	201	8.3	13.0	18.2	23.6	32.3	39.3	46.0	57.3	218.3
	Mulgoa Road	351	12.3	18.2	24.3	32.6	44.0	52.7	63.1	81.8	303.3
Showground Channel	Station Street	308	7.1	10.0	11.6	14.8	18.9	23.6	28.2	36.1	138.3
	Mulgoa Road	233	8.9	12.7	15.9	20.0	27.5	33.4	40.0	49.6	193.0
	Peach Tree Creek confluence	250	9.0	13.5	17.5	21.7	29.1	35.1	41.8	51.5	214.1

- Median discharge for each storm duration (represented by the blue horizontal line contained within each green box);
- Average discharge for each storm duration (defined by the "✳");
- The first and third quartiles (defined by the green box), which illustrated the 25th percentile and 75th percentile discharge values;
- The highest and lowest discharge value (represented by the "T" attached to the end of the green box)
- The critical storm duration is highlighted in yellow

The results of the hydrologic analysis also indicate a large number of unique combinations of critical durations and temporal patterns across the study area. More specifically:

- 50% AEP: 44 unique critical storms;
- 20% AEP: 50 unique critical storms;
- 10% AEP: 54 unique critical storms;
- 5% AEP: 52 unique critical storms;
- 2% AEP: 58 unique critical storms;
- 1% AEP: 47 unique critical storms;
- 0.5% AEP: 54 unique critical storms; and,
- 0.2% AEP: 58 unique critical storms.

6.3 Hydraulics

6.3.1 Boundary Conditions

Inflows

As discussed in the previous section, an XP-RAFTS hydrologic model was used to simulate the transformation of rainfall into runoff and generate discharge hydrographs throughout the study area. The discharge hydrographs generated by the XP-RAFTS models were used to define upstream (i.e., inflow) boundary conditions for the TUFLOW models.

However, as noted above, a large number of unique critical durations and temporal patterns were determined as part of the hydrologic analysis (417 unique combinations of AEP, duration and temporal patterns). Although the XP-RAFTS model runs in a matter of seconds and can run a large number of storms in a relatively short amount of time, the hydraulic model takes several hours to run a single storm. Therefore, it was not considered feasible to run all unique combinations of storm durations and temporal patterns through the hydraulic model in a timely manner.

Therefore, the assessment of critical durations and temporal patterns was restricted to a selection of "focus" locations. A focus location was defined as a major watercourse or a location where preliminary design flood simulations showed a major overland flow path. A total of 24 focus locations were identified and are shown in **Plate 11**.

Once the assessment of critical durations and temporal patterns was reduced from every subcatchment (i.e., >900 locations) down to 24 focus locations, the number of unique durations was significantly reduced (less than 20 unique combination per design flood). However, this was still considered to be too many simulations to undertake efficiently (noting that the future floodplain risk management study will require simulation of each design flood multiple times for each of the potential flood risk mitigation measures).

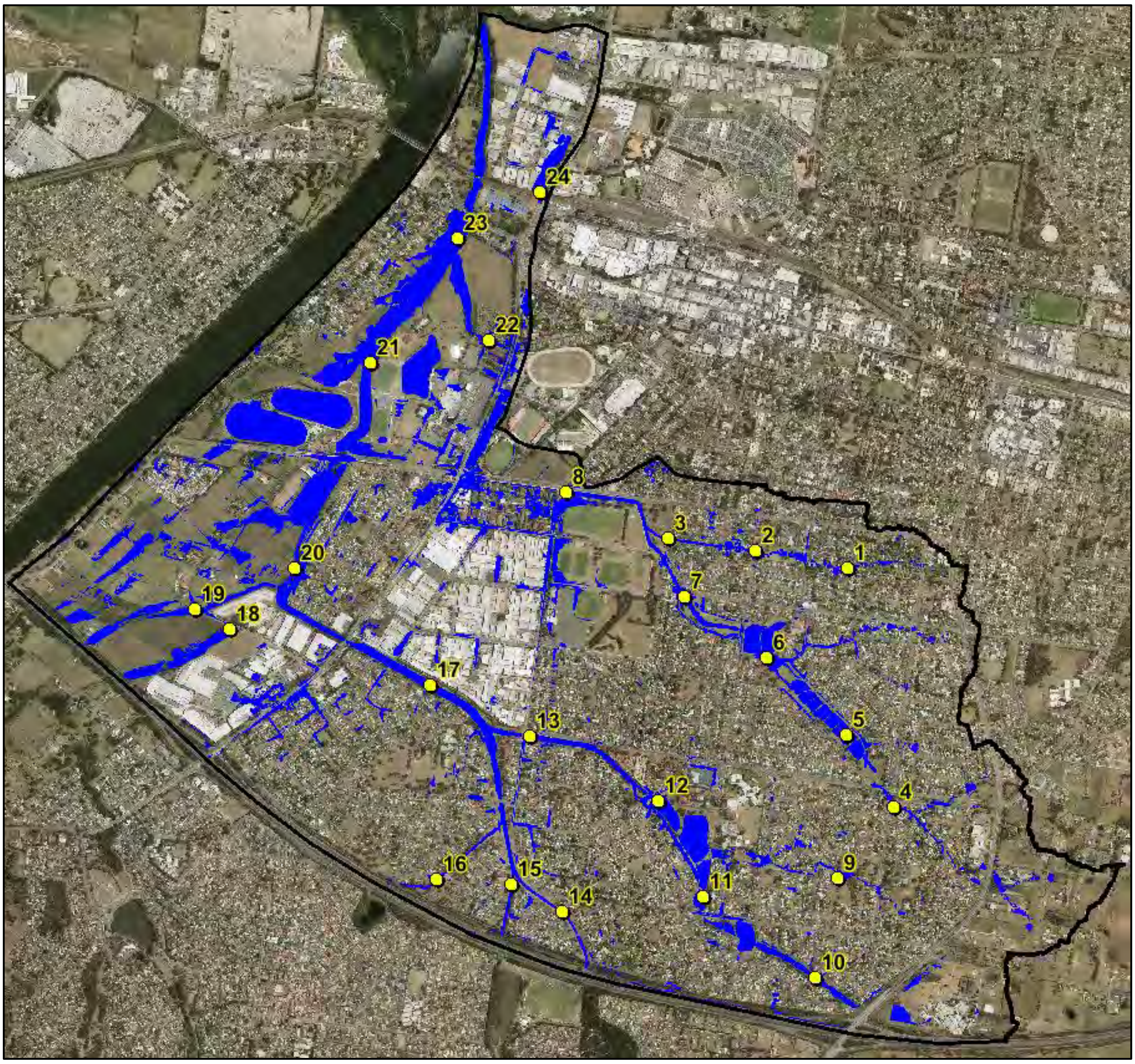


Plate 11 “Focus” locations (yellow) selected for critical duration & temporal pattern analysis

Therefore, the XP-RAFTS results were further reviewed to determine if a reduced number of temporal patterns and durations could be applied without significantly impacting on the overall hydrologic outcomes. The peak discharges generated by the most commonly “critical” temporal patterns and durations were compared against the peak discharges generated by the “correct” temporal patterns and durations for each subcatchment to determine the difference in peak discharge that would results from using a reduced set of temporal patterns. A preference was given to adopting temporal patterns and durations that produced a peak discharge slightly higher than the critical discharge in preference to a lower discharge to ensure a conservative estimate of flood behaviour was being provided.

The outcomes of this assessment are summarised in **Appendix L**. **Appendix L** reproduces the "actual" critical discharges (i.e., based on consideration of all durations and temporal patterns) for each subcatchment but also includes the peak discharges that would be generated based upon the reduced number of temporal patterns. The temporal patterns and storm durations that were ultimately selected for each AEP are summarised in **Table 12**.

Table 12 Adopted temporal patterns and storm durations for hydraulic analysis

Design Storm	Storm Durations and Temporal Pattern ID				
	10 mins	45 mins	60 mins	120 mins	360 mins
50% AEP	4389				4741
20% AEP	4386			4645	4741
10% AEP	4369		4567	4621	4730
5% AEP			4567		4731
2% AEP		4528		4619	4529
1% AEP		4534		4619	4529
0.5% AEP			4463	4619	4529
0.2% AEP			4463	4431	4529

As shown in **Table 12**, the critical ARR2016 durations varied considerably with storm durations of between 10 minutes and 360 minutes being adopted. The shorter duration tended to be critical across the smaller contributing catchment areas (e.g., overland flow path areas) while the longer durations tended to be critical across the mainstream flow areas.

The peak discharges summarised in **Appendix K** show that the peak discharges generated by the adopted temporal patterns are typically lower than the "actual" discharges (i.e., based upon considered of all durations and temporal patterns) across areas away from the main flow paths and slightly higher than the actual discharges along major flow paths and watercourses. The average difference in peak discharges at each of the "focus locations" was typically less than 5%. Therefore, although adopting a reduced set of temporal patterns is providing conservative ARR2016 discharge estimates, the discharges are not significantly inflated. Therefore, it is considered that the reduced set of temporal patterns and durations is reasonable for application to the hydraulic model as part of the ARR2016 analysis.

Nepean River Boundary Conditions

The Peach Tree and Lower Surveyors Creek catchment drains into the Nepean River. Accordingly, the prevailing water level within the Nepean River can have a significant impact on flood behaviour across the downstream catchment area. Therefore, it is important to define a reliable Nepean River boundary condition as part of the design flood simulations. At the same time, it was also considered important to note that the goal of the current study is to define flood behaviour for the Peach Tree and Lower Surveyors Creek and not re-define flood behaviour for the Nepean River (which has been completed as part of the 'Nepean River Flood Study: Exhibition Draft Report' (Advisian, 2017)).

As noted in Section 3.2.3, it is unlikely that floods of equivalent frequency will occur simultaneously in the Nepean River and Peach Tree and Lower Surveyors Creek catchments due to the different characteristics of each catchment. As part of past studies in the catchment, it was assumed that a 5% AEP Nepean River flood was occurring in conjunction with a 1% AEP flood within the Peach Tree and Lower Surveyors Creeks catchment. This equates to a peak Nepean River water level 22.73 mAHD at the confluence of the Nepean River and Peach Tree and Surveyors Creek catchment increasing along the length of the Nepean River to 25.58 mAHD at the Mulgoa Creek confluence.

As part of the hydraulic analysis for other nearby catchments (e.g., College, Orth & Werrington Creeks catchment), it was assumed that floods of equivalent severity were occurring in the local catchment and receiving watercourse during all events up to and including the 5% AEP event. For local catchment events rarer than the 5% AEP, the 5% AEP tailwater level was retained. Accordingly, it was considered appropriate to retain the 5% AEP Nepean River tailwater for all local catchment events equal to and greater than the 5% AEP event.

It was considered desirable to maintain consistency with other studies by assuming floods of equivalent severity were occurring in the local catchment and Nepean River for floods more frequent that the 5% AEP event. However, no design flood level information is available for the Nepean River for floods more frequent than the 5% AEP event. Therefore, the 5% AEP Nepean River level was also used to define tailwater levels for local catchment floods more frequent than the 5% AEP event. That is, the 5% AEP Nepean River water level was adopted as the downstream tailwater level for all design flood simulations.

The combination of local catchment and Nepean River floods that were adopted for each design flood is summarised in **Table 13**.

Table 13 Adopted Nepean River Boundary Conditions

Design Flood Event	Peach Tree & Surveyors Creek Flood Event	Nepean River Flood Event
50% AEP	50% AEP	5% AEP
20% AEP	20% AEP	5% AEP
10% AEP	10% AEP	5% AEP
5% AEP	5% AEP	5% AEP
2% AEP	2% AEP	5% AEP
1% AEP	1% AEP	5% AEP
0.5% AEP	0.5% AEP	5% AEP
0.2% AEP	0.2% AEP	5% AEP
PMF	PMF	5% AEP

The impact of alternate tailwater levels on design flood behaviour was subsequently assessed as part of the sensitivity analysis (refer Section 8.2.6).

6.3.2 Hydraulic Structure Blockage

Culvert and Bridge Blockage

'Base' blockage factors for each bridge and culvert in the study were estimated based upon recommendations in Chapter 6 of Book 6 of 'Australian Rainfall & Runoff' (Geoscience Australia, 2016). This document also recommends adjusting the 'base' blockage factors up or down depending on the severity of the event (i.e., higher blockage factors during larger floods and lower blockage factors during smaller floods). A summary of the blockage scenarios that were adopted for each design flood is provided in **Appendix F** and is also summarised below:

- Low Blockage Scenario – 50% AEP, 20% AEP and 10% AEP events
- Medium Blockage Scenario – 5%, 2%, 1% and 0.5% AEP events
- High Blockage Scenario – 0.2% AEP and PMF events

The only exception to this was at each of the M4 Motorway culverts. As these structures have a significant impact on flows entering the study area (both from the upper Surveyors Creek catchment as well as flows from the Nepean River during large floods), it was considered important to ensure a conservative estimate of flood behaviour was being provided. As a result, no blockage was assigned to any of the M4 Motorway culverts.

The impact of no blockage as well as complete blockage of culverts and bridges was assessed as part of the sensitivity analysis (refer Section 8.2.5).

Stormwater Blockage

Blockage factors were also assigned to stormwater pits based on Council's current pit blockage policy, which is summarised in **Table 14**. The impact of no blockage as well as complete blockage of stormwater pits was assessed as part of the model sensitivity analysis.

Table 14 Adopted Stormwater Pit Blockage Factors

Pit Type	Blockage Factor
Side entry (Sag)	20%
Grated (Sag)	50%
Combination (Sag)	Side inlet capacity only (i.e., complete blockage of grate)
Letterbox (Sag)	50%
Side entry (On-Grade)	20%
Grated (On-Grade)	50%
Combination (On-Grade)	10%

6.3.3 Model Updates

Ground surface elevations within the TUFLOW model have largely been defined based upon 2011 LiDAR supplemented with 2016 LiDAR as well as ground survey. However, modifications to the existing terrain is expected across some areas of the catchment in the near future.

Therefore, the terrain description provided by the LiDAR datasets will not necessarily provided a reliable description of the future landform.

Therefore, areas where terrain modifications are expected in the near future were incorporated into the “design” terrain representation. This included:

- Panthers World of Entertainment: The Panthers site was modified to reflect the ultimate development of this area. This was defined based upon TUFLOW modelling that was completed for the *‘Panthers Precinct Master Plan – Flood Assessment Report’* (J. Wyndam Prince, 2016).
- The Northern Road: The future widening of The Northern Road from south of the M4 Motorway north to the intersection of Jamison Road and the Northern Road was also included in the model based upon design plans provided by Roads and Maritime Services. This included terrain modifications as well as modifications to drainage structures (e.g., culverts).

It should be noted that the design terrain information may not reflect the final topography. Therefore, the results shown in these areas are considered to be approximate only and are subject to further confirmation once each development is finalised.

6.4 Results

6.4.1 Design Flood Envelope

As discussed, a range of storm durations and temporal patterns were simulated for each design flood to ensure the critical discharge was being simulated across all critical sections of the catchment.

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.

6.4.2 Presentation of Results

The results of the flood simulations were reviewed and was noted a number of properties were impacted by shallow water depths that would not present a significant flood hazard. Therefore, it was considered 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 was adopted as the filter criteria 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) (2016), 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.

Accordingly, flood model results were only presented in the maps and figures where the depth of inundation was predicted to exceed 0.15 metres. However, it was noted the application of a depth threshold in isolation generated a number of "puddles". In many cases the puddles were isolated and did not form part of an overland flow path. Therefore, an additional filter was applied whereby all "puddles" less than 100 m^2 in size were also removed from the presentation of results if they did not align with an overland flow path.

It should also be noted that the TUFLOW model results were also "clipped" to the study area boundary. That is, results are not displayed in areas outside of the study area (e.g., Penrith CBD and south of the M4 Motorway).

6.4.3 Field Verification

Preliminary floodwater depth maps were prepared for the 1% AEP flood based upon the depth and area filter criteria outlined in Section 6.4.2. The preliminary maps were subject to an initial desktop review to determine if the mapped inundation depths and extents appeared realistic.

In areas where the desktop analysis proved inconclusive, "ground truthing" was completed to confirm the veracity of the modelling results. The ground truthing involved undertaking a field review of locations where there was some uncertainty associated with the preliminary mapping results. This aimed to confirm whether the modelling results were realistic in the first instance and whether the results should be retained or removed across these areas. In a number of cases the modelling results were considered to overestimate floodwater depths, particularly in areas where there were relatively narrow flow paths between buildings that could not be well represented in the model. Consequently, the ground truthing resulted in the preliminary modelling results being removed from the final flood mapping across a number of locations and/or the model being modified to better reflect field conditions.

The outcomes of the field verification are summarised in **Appendix R**.

6.4.4 Peak Depths, Levels and Velocities

Results were extracted from the final design flood envelopes and were used to prepare a range of flood mapping for the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and PMF events. This includes:

- Floodwater Depths: **Figures 15** to **23**
- Floodwater Levels: **Figures 24** to **32**
- Floodwater Velocities: **Figures 34** to **42**

Peak floodwater surface profiles were also extracted along Peach Tree Creek and the Lower Surveyors Creek channels and are provided in **Figure 33**.

Peak flood levels, depths and velocities were also extracted at twenty-four discrete locations across the study area and are provided in **Table 15**, **Table 16** and **Table 17** respectively. The locations where the results were extracted are shown in **Plate 12**.

Plate 12 Tabulated flood result locations

6.4.5 Stormwater System Capacity

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

Table 15 Peak Design Flood Levels at Various Locations across the Catchment

Location (refer to Plate 12)	**Peak Flood Level (mAHD)**								
	50% AEP	**20% AEP**	**10% AEP**	**5% AEP**	**2% AEP**	**1% AEP**	**0.5% AEP**	**0.2% AEP**	**PMF**
1	44.63	44.86	45.04	45.06	45.09	45.17	45.23	45.29	45.80
2	NA	37.92	37.94	37.95	37.99	38.03	38.07	38.11	38.43
3	32.65	32.71	32.75	32.79	32.82	32.85	32.89	32.95	33.40
4	NA	49.39	49.60	49.65	49.78	49.84	49.95	50.02	50.81
5	44.31	44.56	44.59	44.63	44.70	44.73	44.79	44.84	45.33
6	38.72	39.02	39.21	39.31	39.40	39.47	39.52	39.60	40.18
7	NA	34.32	34.58	34.69	34.76	34.88	34.97	35.01	35.78
8	27.81	28.06	28.41	28.61	28.83	28.85	29.06	29.12	30.13
9	49.25	49.36	49.58	49.75	49.84	49.90	49.94	49.97	50.35
10	50.00	50.06	50.11	50.14	50.17	50.22	50.25	50.28	50.58
11	40.36	40.85	40.97	41.05	41.14	41.18	41.22	41.30	41.76
12	36.96	37.21	37.33	37.42	37.52	37.63	37.71	37.77	38.35
13	31.67	31.76	31.94	32.18	32.39	32.65	32.85	32.92	34.24
14	34.76	35.12	35.23	35.27	35.34	35.42	35.51	35.54	37.56
15	33.72	34.05	34.46	34.54	34.66	34.74	34.83	34.94	37.13
16	36.22	36.26	36.31	36.33	36.38	36.43	36.45	36.49	36.97
17	29.26	29.42	29.49	29.58	29.65	29.73	29.85	29.93	31.57
18	26.59	26.59	26.59	26.59	26.59	26.59	26.60	26.62	27.36
19	25.79	26.01	26.08	26.14	26.16	26.17	26.25	26.29	27.20
20	25.70	26.20	26.42	26.63	26.79	26.90	27.01	27.08	27.55
21	23.05	23.30	23.44	23.55	23.66	23.77	23.89	24.01	26.96
22	25.44	25.68	25.92	26.00	26.03	26.05	26.09	26.13	27.04
23	22.86	22.99	23.08	23.16	23.24	23.35	23.49	23.59	26.80
24	24.22	24.28	24.35	24.36	24.38	24.44	24.60	24.78	26.46

Table 16 Peak Design Flood Depths at Various Locations across the Catchment

Location (refer to Plate 12)	**Peak Flood Depth (metres)**								
	50% AEP	**20% AEP**	**10% AEP**	**5% AEP**	**2% AEP**	**1% AEP**	**0.5% AEP**	**0.2% AEP**	**PMF**
1	0.20	0.43	0.59	0.61	0.64	0.74	0.78	0.84	1.36
2	0.00	0.17	0.19	0.20	0.24	0.28	0.33	0.36	0.68
3	0.36	0.43	0.47	0.50	0.53	0.56	0.61	0.66	1.12
4	0.00	0.28	0.50	0.55	0.67	0.73	0.84	0.92	1.70
5	0.68	0.93	0.97	1.01	1.08	1.11	1.17	1.22	1.70
6	0.58	0.87	1.06	1.16	1.26	1.32	1.37	1.46	2.04
7	0.00	0.30	0.56	0.66	0.74	0.85	0.94	0.99	1.75
8	0.82	1.07	1.41	1.61	1.83	1.86	2.07	2.13	3.14
9	0.69	0.80	1.03	1.20	1.29	1.34	1.39	1.42	1.79
10	0.98	1.03	1.09	1.11	1.15	1.20	1.22	1.26	1.56
11	0.76	1.25	1.38	1.46	1.55	1.58	1.63	1.71	2.16
12	0.67	0.92	1.04	1.13	1.23	1.33	1.42	1.48	2.05
13	0.29	0.38	0.55	0.80	1.01	1.27	1.47	1.54	2.86
14	1.48	1.85	2.02	2.06	2.12	2.15	2.30	2.33	4.28
15	0.79	1.12	1.50	1.58	1.71	1.81	1.87	1.99	4.20
16	0.24	0.28	0.32	0.35	0.39	0.45	0.46	0.50	0.99
17	0.85	1.02	1.06	1.15	1.22	1.32	1.42	1.50	3.17

Location (refer to Plate 12)	**Peak Flood Depth (metres)**								
	50% AEP	**20% AEP**	**10% AEP**	**5% AEP**	**2% AEP**	**1% AEP**	**0.5% AEP**	**0.2% AEP**	**PMF**
18	0.30	0.30	0.30	0.30	0.30	0.30	0.31	0.33	1.07
19	0.46	0.68	0.69	0.75	0.77	0.84	0.86	0.90	1.86
20	1.51	2.01	2.23	2.44	2.60	2.71	2.82	2.89	3.36
21	3.31	3.56	3.74	3.86	3.97	4.03	4.19	4.32	7.22
22	1.43	1.67	1.92	1.99	2.02	2.04	2.08	2.12	3.03
23	3.52	3.65	3.73	3.81	3.90	4.00	4.14	4.24	7.45
24	2.19	2.25	2.32	2.33	2.36	2.41	2.58	2.75	4.43

Table 17 Peak Velocities at Various Locations across the Catchment

Location (refer to Plate 12)	**Peak Velocity (m/s)**								
	50% AEP	**20% AEP**	**10% AEP**	**5% AEP**	**2% AEP**	**1% AEP**	**0.5% AEP**	**0.2% AEP**	**PMF**
1	0.19	0.24	0.28	0.22	0.17	0.19	0.23	0.25	0.51
2	0.00	0.07	0.11	0.11	0.18	0.28	0.36	0.44	0.92
3	0.18	0.20	0.24	0.33	0.49	0.66	0.94	1.20	2.49
4	0.00	0.17	0.24	0.17	0.36	0.44	0.88	1.11	2.15
5	0.10	0.32	0.40	0.49	0.58	0.62	0.66	0.68	1.09
6	0.03	0.05	0.13	0.16	0.26	0.29	0.33	0.43	1.06
7	0.00	0.15	0.18	0.21	0.31	0.67	0.84	0.91	1.38
8	0.89	0.93	0.96	0.89	1.17	1.05	1.11	1.13	1.19
9	0.53	0.71	0.75	0.38	0.36	0.51	0.56	0.56	0.90
10	0.16	0.20	0.24	0.24	0.30	0.39	0.44	0.49	0.92
11	0.61	0.71	0.95	0.56	0.69	0.80	0.79	0.90	1.46
12	0.06	0.07	0.09	0.06	0.16	0.16	0.26	0.34	1.19
13	0.47	0.84	0.92	0.93	0.92	0.93	1.05	1.13	2.14
14	2.68	2.68	2.64	2.56	2.65	2.75	2.68	2.74	3.28
15	1.96	2.44	2.78	2.75	2.77	2.54	2.80	2.84	3.78
16	0.74	0.86	0.99	1.09	1.12	1.16	1.35	1.48	1.93
17	2.02	2.29	2.35	2.48	2.59	2.72	2.86	2.97	4.89
18	0.66	0.66	0.66	0.66	0.67	0.66	0.67	0.67	0.67
19	0.11	0.19	0.22	0.18	0.18	0.19	0.20	0.24	1.27
20	0.98	1.20	1.29	1.39	1.45	1.49	1.53	1.55	1.91
21	0.63	0.85	0.93	1.01	1.10	1.18	1.22	1.30	1.80
22	0.57	0.74	0.97	1.13	1.19	1.23	1.32	1.37	3.62
23	0.76	1.08	1.23	1.35	1.45	1.59	1.76	1.87	3.10
24	0.03	0.05	0.06	0.03	0.02	0.03	0.04	0.04	1.45

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 95% full during the 10% AEP event and 100% full during the 5% AEP event, the pipe capacity would be defined as "10% AEP".

A nominal return period was also calculated for each pit based on one of the following "failure" criteria:

- AEP at which the pit begins to surcharge;
- AEP at which the water depth at the pit exceeds 0.2 metres;

The resulting stormwater capacity maps are presented in **Figure 43**. As shown in **Figure 43**, the pit and pipe capacities are colour coded based on the nominal capacity that was calculated. Furthermore, different symbols have been applied to each pit to define whether the pit first "fails" via ponding depth or surcharge.

The information presented in **Figure 43** shows that the capacity of the system varies considerably across the catchment. Some sections of the stormwater system have a capacity of less than the 50% AEP flood while other sections of the stormwater system are able to convey flows in excess of the 1% AEP event. But, overall, the majority of the stormwater system (i.e., >60%) is predicted to have a capacity of no greater than the 20% AEP event. **Figure 43** also indicates that the pipe capacity rather than pit capacity appears to be the limiting factor in the performance of the stormwater system.

6.4.6 Inundated Properties

The number of properties inundated during each design flood was also determined. This information is summarised in **Table 18** (there are 6,505 properties contained within the study area). The information presented in **Table 18** indicates that 14% of properties located within the catchment will be at least partly inundated to a depth of at least 0.15 metres at the peak of the 1% AEP flood. This is predicted to increase to well over 40% during the PMF. Accordingly, major flooding has the potential to impact a significant number of properties within the catchment.

Table 18 Number of Inundated Properties

Event	Number of Inundated Properties	Percentage of Total Number of Properties
50% AEP	327	5%
20% AEP	406	6%
10% AEP	561	9%
5% AEP	589	9%
2% AEP	731	11%
1% AEP	910	14%
0.5% AEP	1119	17%
0.2% AEP	1325	20%
PMF	2999	46%

6.5 Results Verification

The XP-RAFTS and TUFLOW models developed as part of this study was validated against recorded and observed flood information for three historic floods. In general, the models were found to provide a good reproduction of historic flood mark elevations. However, the outcomes of the calibration only provide evidence that the model is providing a reliable representation of flood behaviour at isolated locations (i.e., at recorded flood mark locations).

Therefore, additional verification of the models was completed by comparing the results generated by each model against past studies. Further details on the outcomes of the model verification is presented below.

It should be noted that the 2016 revision of Australian Rainfall & Runoff has been used as part of the current study to define catchment hydrology. All previous studies have used the 1987 version of Australian Rainfall & Runoff. Accordingly, it is not possible to complete an "apples with apples" comparison as part of this verification due to the different hydrologic approaches. Nevertheless, it is still considered possible to ensure the models are producing realistic estimates of design flood behaviour using this verification approach.

6.5.1 Comparison with Past Studies

A number of flooding and drainage investigations have previously been prepared to define flood behaviour across various parts of the Peach Tree and Lower Surveyors Creeks catchment. This includes:

- Peach Tree Creek Flood Study (PWD, 1994)
- Penrith CBD Detailed Overland Flow Flood Study (Cardno, 2015)
- Penrith Overland Flow Flood "Overview Study" (Cardno, 2006).

Hydrology

Peak 1% AEP discharges were extracted from the above reports (where available) and were compared against peak 1% AEP discharges produced by the XP-RAFTS model. The peak discharge comparison is provided in **Table 19**.

Table 19 Comparison between peak 1% AEP discharges

Location	Peak 1% AEP Discharge (m^3/s)		
	Peach Tree Creek Flood Study (1994)	Penrith CBD Detailed Overland Flow Flood Study (2015)	Current Study (XP-RAFTS)
Surveyors Creek downstream of Mulgoa Road	140	N/A	127
Racecourse Channel @ Mulgoa Road	48	30.9	52.6
Showground Channel @ Mulgoa Road	40	17.2	34.0

The comparison presented in **Table 19** indicates the XP-RAFTS model is generally producing lower peak 1% AEP discharges relative to the 'Peach Tree Creek Flood Study' (PWD, 1994) and higher discharges relative to the 'Penrith CBD Detailed Overland Flow Flood Study' (Cardno, 2015).

It should be noted that the 'Penrith CBD Detailed Overland Flow Flood Study' flow results were extracted from the "direct rainfall" TUFLOW model, which accounts for minor storages across the upstream catchment (e.g., behind road embankments), which is not explicitly represented in the XP-RAFTS modelling. Accordingly, the cumulative impact of these small storages is the likely reason for the lower peak discharge estimates.

It is difficult to determine the precise reason for the differences between the XP-RAFTS model peak discharges with those documented in the 'Peach Tree Creek Flood Study' (PWD, 1994) given the different hydrologic approaches. However, it is noted that the ARR2016 1%AEP rainfall depths are lower than the equivalent ARR1987 rainfall depths for storm durations less than 2 hours, which may be contributing to the differences (along with the revised ARR2016 hydrologic procedures). Nevertheless, the differences in peak discharges are not substantial and indicates that the XP-RAFTS model is producing realistic design discharge estimates.

Hydraulics

Peak 1% AEP levels were also extracted from the previous reports and were compared against peak 1% AEP flood levels produced by the TUFLOW model developed for the current study at various locations across the catchment. The water level comparisons are provided in **Table 20**.

Table 20 Verification of 1% AEP water levels

Location (refer to Plate 12)	**Peak Water Level (mAHD)**			
	Peach Tree Creek Flood Study (1994)	**Penrith Overland Flow Flood "Overview Study" (2006)**	**Penrith CBD Detailed Overland Flow Flood Study (2015)**	**Current Study (TUFLOW)**
1	N/A	45.20	N/A	45.17
2	N/A	38.07	N/A	38.03
3	N/A	32.89	N/A	32.84
4	N/A	49.96	N/A	49.84
5	N/A	44.56	N/A	44.73
6	N/A	39.41	N/A	39.47
7	N/A	34.80	N/A	34.88
8	N/A	29.26	28.99	28.85
9	N/A	49.67	N/A	49.90
10	N/A	49.43	N/A	50.22
11	N/A	40.97	N/A	41.18
12	N/A	37.64	N/A	37.63
13	N/A	32.92	N/A	32.65
14	N/A	35.90	N/A	35.42
15	N/A	34.49	N/A	34.74
16	N/A	36.14	N/A	36.43
17	N/A	29.98	N/A	29.73
18	27.1	26.56	N/A	26.59 (27.20)*
19	26.5	26.20	N/A	26.17 (26.69)*

Location (refer to Plate 12)	Peak Water Level (mAHD)			
	Peach Tree Creek Flood Study (1994)	Penrith Overland Flow Flood "Overview Study" (2006)	Penrith CBD Detailed Overland Flow Flood Study (2015)	Current Study (TUFLOW)
20	26.5	26.94	N/A	26.90 (26.90)*
21	25.9	N/A	N/A	23.77 (26.06)*
22	25.9	25.62	N/A	26.05 (26.06)*
23	25.9	N/A	N/A	23.35 (26.06)*
24	N/A	25.11	24.55	24.44

NOTE: * to enable a meaningful comparison between the TUFLOW model results and results documented in the 'Peach Tree Creek Flood Study' (1994), peak flood levels from the 1% AEP Nepean River tailwater with 5% AEP local catchment flood sensitivity simulations (refer Section 8.2.6) are included in parenthesis

The Penrith Overland Flow Flood "Overview Study" (Cardno, 2006) provides the most comprehensive flood level information across the catchment. The comparison provided in **Table 20** indicates that both studies generate similar peak 1% AEP flood levels (i.e., levels generally agree to better than 0.3 metres). The current study generally produces slightly lower peak flood levels. This is considered to associated with the current study including a full representation of the stormwater pipe system, which will result in less flow travelling overland (and consequently lower overland flood levels). There are some locations where the TUFLOW model developed for the current study is predicting higher peak design flood levels relative to the 2006 study. These are typically located downstream of major roadway embankment (e.g., M4 Motorway) and are likely associated with some of the major cross-drainage structures not being represented in the 2006 study (consequently some of the embankments serve as detention basins, resulting in lower downstream flood levels).

Only limited coincidental flood level information is available for the 'Penrith CBD Detailed Overland Flow Flood Study' (Cardno, 2015). The available information indicates that the TUFLOW model developed for the current study generates a slightly lower levels at both locations (although the differences are no greater than 0.2 metres). The lower levels are considered to be primarily hydrology related (i.e., ARR2016 generating lower discharges relative to ARR1987).

Flood level information for the 'Peach Tree Creek Flood Study' (PWD, 1994) is primarily restricted to the catchment area located west of Mulgoa Road. Comparisons are further complicated by the fact that flood levels in the 1994 study comprise an "envelope" incorporating a 1% AEP Nepean River tailwater. To enable a more meaningful comparison to be completed, peak flood levels generated as part of a 1% AEP tailwater with 5% AEP local catchment runoff sensitivity simulation completed for the current study (refer Section 8.2.6) were extracted and are included in parenthesis in **Table 20**. When comparing the peak 1994 water levels with the 1% AEP tailwater sensitivity flood levels, all flood levels agree to within 0.2 metres. The flood levels generated by the TUFLOW model are typically higher than the 1994 study and are most likely associated with the blockage factors adopted as part of the current study (the 1994 study did not incorporate blockage factors for hydraulic structure resulting in lower flood levels upstream of structures).

It was noted a more significant difference in 1% AEP flood levels occurs at Location ID 20 where the TUFLOW model developed for the current study is predicting a 1% AEP flood level that is 0.4 metres higher than the 'Peach Tree Creek Flood Study' (PWD, 1994) flood level. This particular location is where the hydraulic gradients "steepens" noticeably. Therefore, if the flood level comparison point was shifted ~80 metres downstream it would still be located within the same 'Peach Tree Creek Flood Study' "cell" and the TUFLOW and 1994 flood study flood levels would compare favourably.

6.5.2 Comparison with Alternate Modelling Approaches

Direct Rainfall Model

As discussed, two separate computer models have been developed as part of the study to define flood behaviour across the Peach Tree and Lower Surveyors Creeks catchment. It was considered important to undertake additional validation of these models by comparing the results generated by the XP-RAFTS and TUFLOW models against an alternate computer modelling approach.

"Direct rainfall" (DR) TUFLOW models have been employed in a number of recent flood studies across the Penrith City Council LGA. The direct rainfall models involve application of rainfall directly to the hydraulic model (i.e., the hydrology and hydraulics are combined into a single model). Unfortunately, the long run times associated with the DR TUFLOW models means that they cannot be efficiently used to simulate the thousands of storms required under ARR2016. However, it was still considered that a DR version of the Peach Tree and Lower Surveyors Creeks catchment could be developed and used to verify the results of the RAFTS/TUFLOW model based upon the reduced set of 1% AEP ARR2016 storms discussed in the preceding sections.

The direct rainfall model was used to re-simulate the 1% AEP flood based upon ARR2016 hydrology. Flow hydrographs were extracted from the direct rainfall model at two locations and were compared against flow hydrographs extracted from the TUFLOW model with hydrology defined by the XP-RAFTS model. The locations where the hydrographs were extracted included:

- Surveyors Creek upstream of Mulgoa Road (reflecting a "main stream" flooding area); and,
- Overland flow path at Evan Street (immediately south-west of Penrith South Public School reflecting an overland flooding area).

The hydrograph comparisons are provided in **Plate 13** and **Plate 14**. The comparison indicates that the overall shape of the hydrographs at both locations are similar. The XP-RAFTS hydrographs tend to rise more rapidly and generate peak discharges that are higher than the direct rainfall versions. This is most likely associated with minor depression storage in the direct rainfall version of the model that is not reflected in the XP-RAFTS version of the model. Overall, the XP-RAFTS and TUFLOW models are generating conservative, but realistic, discharge estimates with the peak discharges agreeing to better than 10%.

The peak flood levels and pipe flows generated by the direct rainfall model were also compared against peak flood levels and pipe flows generated by the RAFTS/TUFLOW models. This is presented in the form of a flood level difference map in **Plate 15** (the difference map is provided for a part section of the Penrith South Public School flow path). Greens/blues indicate the direct rainfall version of the model is generating lower levels while oranges/reds indicates the direct rainfall version is generating higher levels

This difference map (as well as inspection of differences away from this specific flow path) shows that, in general, the XP-RAFTS version of the model is producing higher levels along the major flow path but lower levels/extents across areas away from the major flow paths as well as in the vicinity of major overland flow obstructions, such as buildings. Most of these differences appear to be associated with water in the direct rainfall model getting "trapped" behind buildings (resulting in a localised build-up of water behind buildings and less water reaching the main flow path). It is likely that the extent of the trapped water is being exaggerated by the direct rainfall model (i.e., the model is not providing a true representation of the narrow flow paths between buildings).

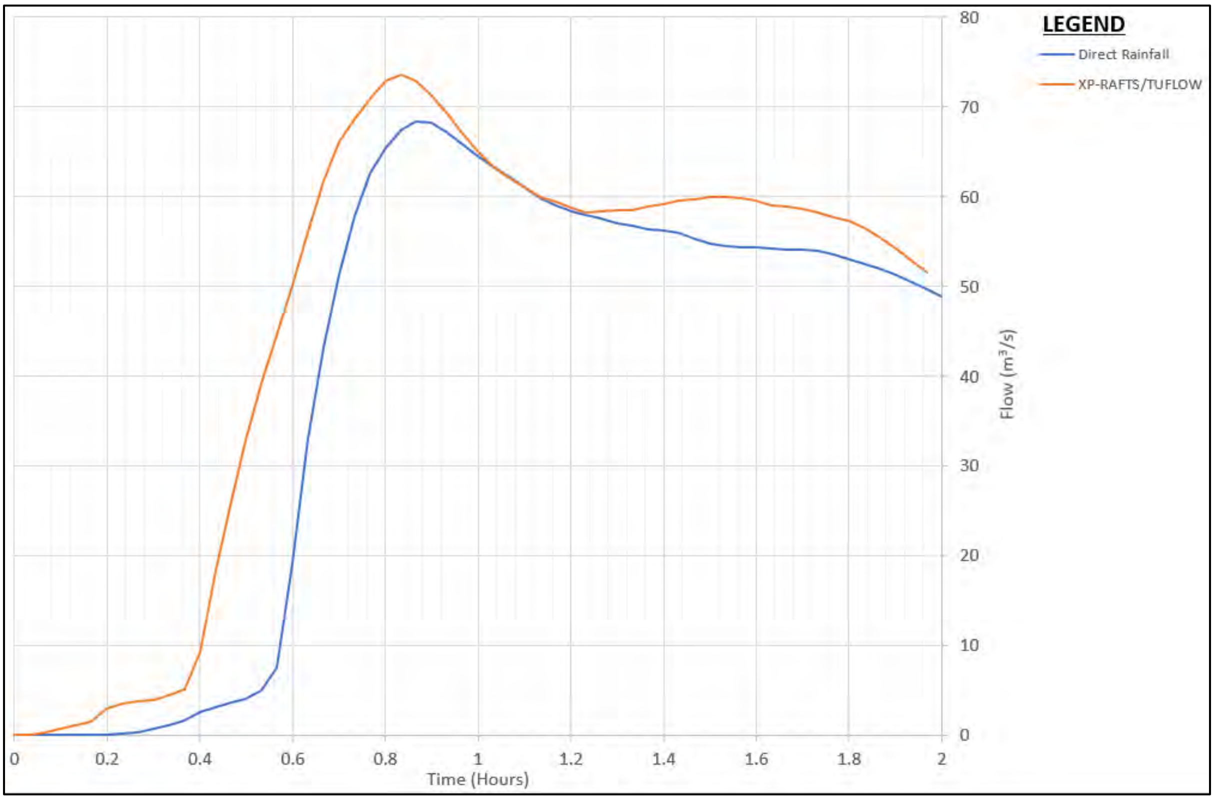


Plate 13 1% AEP Direct Rainfall Hydrograph Comparison for Surveyors Creek upstream of Mulgoa Road

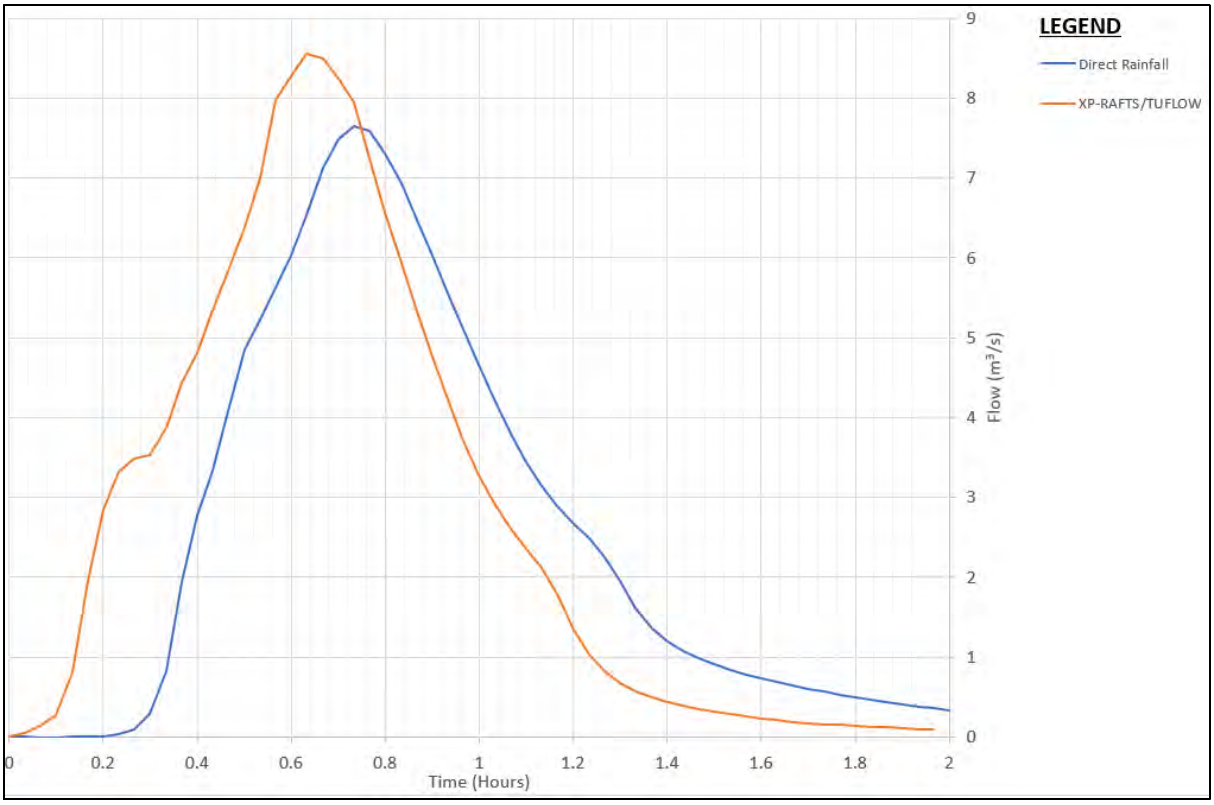


Plate 14 1% AEP Direct Rainfall Hydrograph Comparison for overland flow path near Penrith South Public School

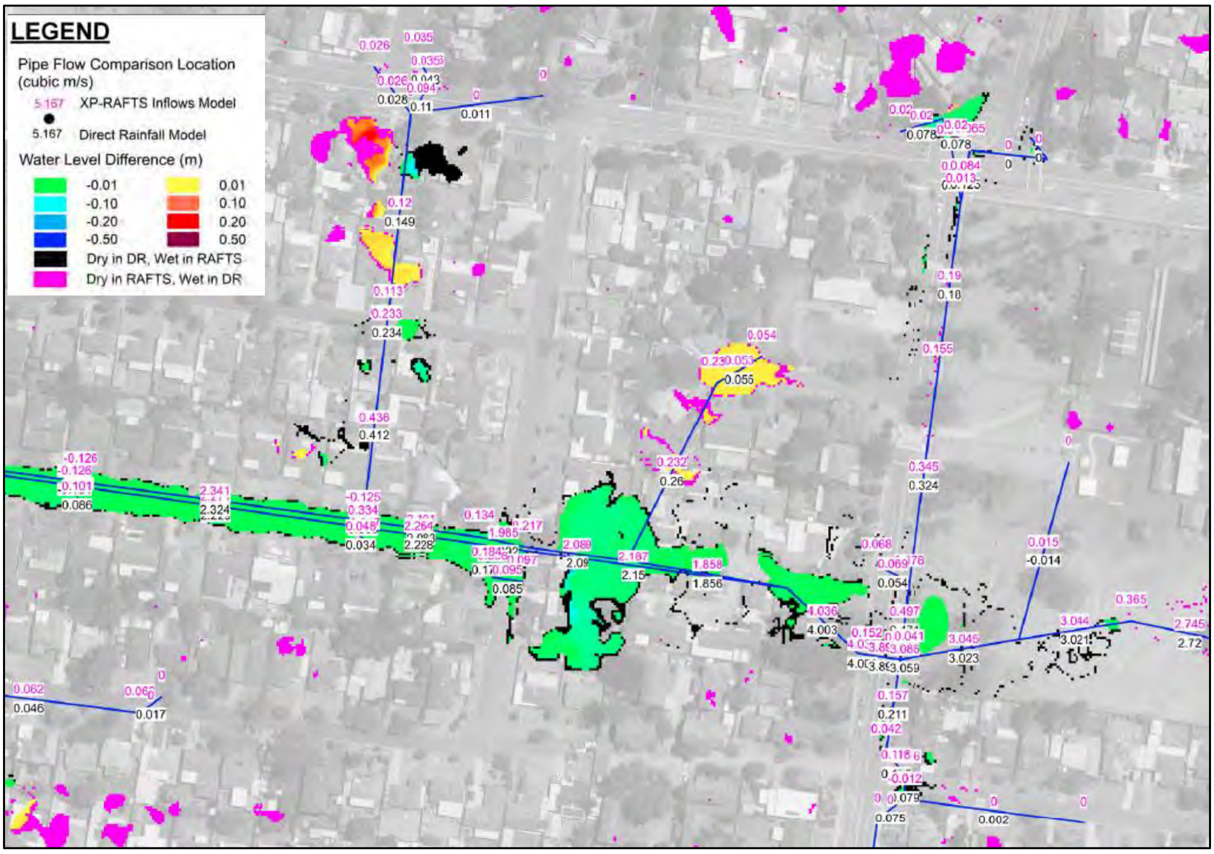


Plate 15 Direct rainfall and XP-RAFTS/TUFLOW flood level difference and pipe flow map near Penrith South Public School

The pipe flow comparison shows that the peak pipe flows along the major overland flow paths are very similar with the XP-RAFTS version of the model producing slightly higher pipe flows relative to the direct rainfall model (although the differences are generally less than 0.02 m^3/s, which is considered to be negligible).

Overall, the results of the direct rainfall verification shows that the XP-RAFTS and TUFLOW models are providing realistic descriptions of design flood behaviour.

6.5.3 Model "Health"

The TUFLOW software automatically reports mass balance errors for the 1D domain, 2D domain and overall model as part of each simulation. Generally, it is desirable to keep mass balance errors below ±1% to ensure that water is not being artificially "added" or "removed" from the model domain. High mass balance errors are an indicator of poor model health and can often be linked to poor model setup/schematisation.

A review of the time variation in 1D, 2D and overall mass balance errors was completed following each simulation. This review determined that:

- Overall mass balance error did not exceed 0.9% for any simulation
- 1D mass balance error did not exceed 0.1% for any simulations
- 2D mass balance error did not exceed 0.6% for any simulations

Therefore, the mass balance error for all simulations is less than the desired ±1% and indicates that the model is "healthy".

6.5.4 Peer Review

As discussed in the preceding section of this report, the TUFLOW computer model provided a good reproduction of historic flood information and also compare favourably against other flood-related studies. However, to further ensure that the model was appropriately setup and parameterised, an additional interval peer review of the model was completed.

The outcomes of the review are summarised in **Appendix S**.

6.5.5 Summary

The outcomes of the results verification presented in this section indicates that the TUFLOW and XP-RAFTS models developed for this study are generally producing hydraulic and hydrologic results that compare favourably with past studies as well as alternate calculation approaches.

Some more notable differences were identified at isolated locations. However, most of these differences are likely associated with the different hydrologic approaches (i.e., ARR2016 versus ARR1987), simplifications with past models (e.g., past models not including all stormwater infrastructure / drainage structures) or differences in modelling assumptions (e.g., blockage). Overall, it is considered that the TUFLOW model results presented in this study provide an improved contemporary representation of hydrologic and hydraulic processes across the Peach Tree and Lower Surveyors Creeks catchments.

7 FLOOD HAZARD AND HYDRAULIC CATEGORIES

7.1 Flood Hazard

Flood hazard defines the potential impact that flooding will have on vehicles, people and property across different sections of the floodplain. More specifically, it describes the potential for floodwaters to cause damage to property or loss of life / injury (Australian Government, 2014).

For this study, the variation in flood hazard across the catchment was defined using flood hazard vulnerability curves presented in Section 7.2.7 of Chapter 7 of Book 6 of 'Australian Rainfall & Runoff' (Geoscience Australia, 2016). This approach was selected over the hazard categorisation defined in the FDM (2005) as it is believed to represent the latest approach to flood hazard definition and provides better correlation between risk to life and flood hazard. The hazard curves are reproduced in **Plate 16** and are also described in **Table 21**.

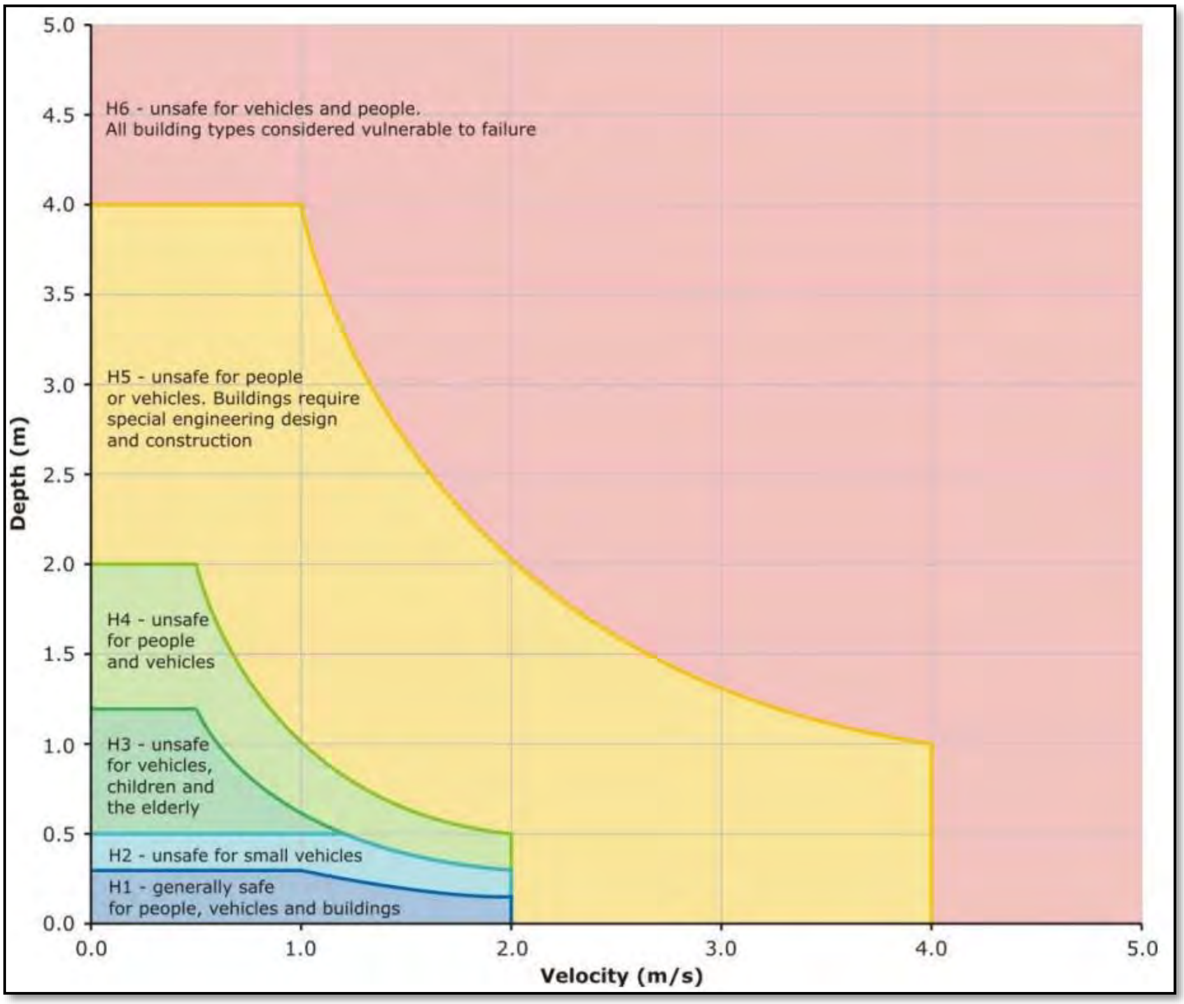


Plate 16 Flood hazard vulnerability curves (Geoscience Australia, 2016)

As shown in **Plate 16**, the hazard curves assess the potential vulnerability of people, cars and structures based upon the depth and velocity of floodwaters at a particular location. Therefore, peak depth, velocity and velocity-depth product outputs generated by the TUFLOW model were used to map the variation in flood hazard across the catchment based on the hazard criteria shown in **Plate 16** for the 5% AEP, 1% AEP, 0.5% AEP flood as well as the PMF. The resulting hazard category maps are shown in **Figures 44** and **47**.

Table 21 Description of Adopted Flood Hazard Categories (Geoscience Australia, 2016)

Hazard Category	Description
H1	Relatively benign flood conditions. Generally safe for vehicles, people and buildings.
H2	Unsafe for small vehicles
H3	Unsafe for vehicles, children and the elderly
H4	Unsafe for vehicles and people of all ages & levels of mobility
H5	Unsafe for vehicles and people. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure
H6	Unsafe for vehicles and people. All building types considered vulnerable to failure.

The hazard maps indicate that during the 1% AEP flood, the flood hazard across most urban areas is predicted to remain at H3 or below. However, across dedicated channels and detention areas, the flood hazard is predicted to increase to H6 in some locations.

During the PMF, many roads and urban overland flow paths would be exposed to a hazard of H4 or above. This indicates that cars and people would be exposed to a significant flood risk

7.2 Flood Emergency Response Classifications

In an effort to understand the potential emergency response requirements across different sections of the catchment, the Office of Environment and Heritage (formerly Department of Environment and *Climat*e Change), in conjunction with the State Emergency Service (SES) developed the "Flood Emergency Response Planning Classification of Communities" (2007). This guideline provides a flow chart that can be used to classify different sections of the catchment into different flood emergency response precincts (refer **Plate 17**)

Each allotment within the catchment was classified based upon the flow chart provided in **Plate 17** for the 5% AEP, 1% AEP, 0.5% AEP and PMF. This was completed in an automated fashion using proprietary software based upon consideration of:

- whether evacuation routes/roadways get "cut" (a 300mm depth threshold was used to define a "cut" road);
- whether evacuation routes continuously rise out of the floodplain (based upon roadway alignments provided by Council's and a 1m LiDAR-based DEM developed for this study);
- 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 (a lot was considered inundated when there was less than 100 m^2 of "dry" land area available);

- if evacuation by car was not possible, whether evacuation by walking was possible (a 800mm depth threshold was used to define when a route could not be traversed by walking).

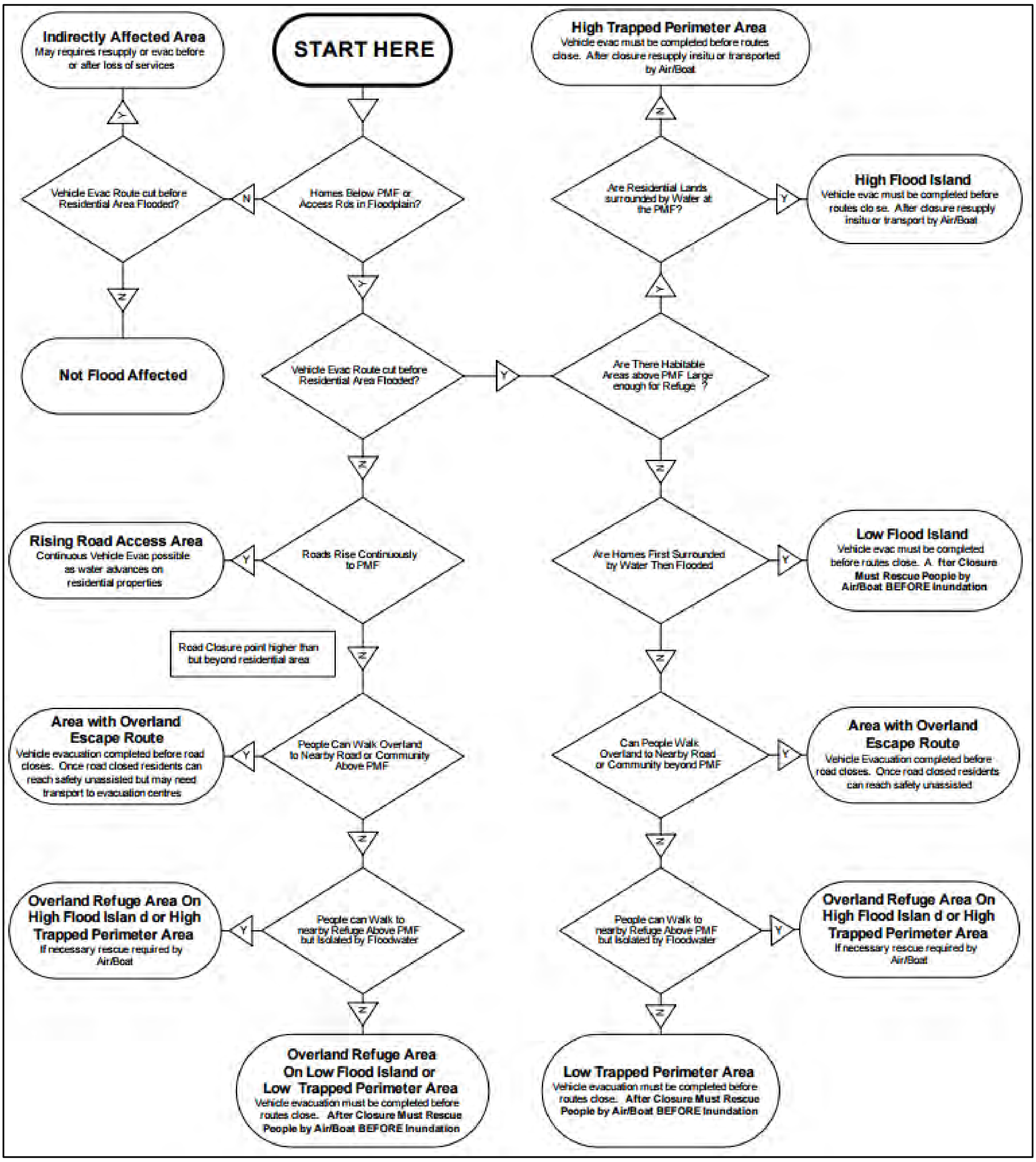


Plate 17 Flood Emergency Response Classification Flow Chart (Department of Environment & Climate Change, 2007)

The resulting classifications for each design flood are provided in **Figures 48** to **51**. The flood emergency response classifications were also completed for an "enveloped" 1% AEP local catchment flood and a 1% AEP Nepean River flood (discussed in more detail in Section 9.2). This is presented in **Figure 50**.

A range of other datasets were also generated as part of the classification process to assist the SES. This includes the locations were 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. A selection of this roadway inundation information is presented in **Figures P1** to **P4** in **Appendix P**.

The emergency response classifications indicate that the most common classifications during smaller design floods are:

- Rising road areas: this classification indicates that evacuation routes grade up and out of the floodplain
- Overland refuge area: this classification indicates that evacuation by vehicle cannot be achieved but evacuation via wading is possible

During larger events (e.g., PMF), much more of the study area fall under the 'low flood island' and 'low trapped perimeter' classification. These classifications indicate that evacuation is lost early in the flood and each lot is eventually inundated. Consequently, a PMF would isolate and inundate a large number of properties and would present a significant emergency response requirement if evacuation is not complete early.

The road inundation information indicates that during floods up to and including the 0.5% AEP event, roadways would not be cut across the downstream sections of the catchment until approximately 6 hours after the commencement of rainfall. Across the upper catchments areas, roadways are predicted to be cut approximately 45 minutes after the initial onset of rainfall. Accordingly, very little warning time would typically be available during large floods. During the PMF, roadways would be cut across most of the catchment approximately 15 minutes after the onset of rainfall.

It was noted a revised process for categorising emergency response precincts had been developed in 2014 (Australian Emergency Management Institute, 2014). However, the new classifications are yet to be widely adopted by the SES across New South Wales. Nevertheless, the SES may move towards this new classification system in the future. Therefore, the new emergency response classifications were also prepared for the 5% AEP, 1% AEP floods as well as the PMF and these are presented in **Appendix T** to assist with future emergency response planning. Also included in **Appendix T** are the national emergency response classifications for the "enveloped" 1% AEP local catchment and 1% AEP Nepean River floods.

7.3 Hydraulic Categories

The NSW Government's *'Floodplain Development Manual'* (NSW Government, 2005) also characterises flood prone areas according to the hydraulic categories presented in **Table 22.** 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.

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.

In an effort to provide quantitative criteria, Howell et al (2003) suggested that floodways can be defined using a combination of velocity depth product and velocity outputs. The criteria proposed by Howell et al is summarised in **Table 22** and was adopted for the current study. However, an additional criterion was added so that all areas contained within a major creek (i.e., from top of bank to top of bank) were also defined as floodways.

Flood storage areas were then defined as those areas located outside of floodways but where the depth of inundation was greater than 0.2 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.

All other areas that were predicted to be flooded but were not classified as flood storage or floodway were designated as "flood fringe" areas.

Table 22 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.	• Minimum top of bank to top of bank (for main stream areas) **AND** • VxD >= 0.25 m^2/s AND V >= 0.25 m/s **OR** • V >= 1.0 m/s
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.	• If not FLOODWAY and D >=0.2 m
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.	• Remaining areas after FLOODWAY and FLOOD STORAGE are defined

The resulting hydraulic category maps for the 5% AEP, 1% AEP and 0.5% AEP floods as well as the PMF are shown in **Figures 53** to **56**.

Figures 53 to **55** indicate that during events up to and including the 0.5% AEP event, the floodways are typically contained to formal waterways/creeks with formal detention areas typically being classified as flood storage areas. Much of the local inundation along roadways would be classified as flood fringe.

However, during the PMF (refer **Figure 56**), the floodway extents are much more significant, with many roadways serving as major flow conveyance areas. In some areas, flow paths between buildings would also be classified as floodway. Those areas not classified as floodway would largely fall under the flood storage classification during the PMF, particularly those areas located west of Mulgoa Road.

8 SENSITIVITY AND CLIMATE CHANGE ANALYSIS

8.1 Overview

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

As outlined in Section 5, computer models are typically validated using recorded rainfall, stream flow and/or flood mark information. Validation 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. Validation is completed in an attempt to ensure the adopted model parameters are generating realistic estimates of flood behaviour.

As discussed in Section 5 and Section 6.6, the XP-RAFTS and TUFLOW models were validated against recorded and observed flood information for three historic events and were further verified against results documented in past studies. In general, this information confirmed that the models were providing realistic descriptions of flood behaviour across the catchment.

Nevertheless, it is important to understand how any uncertainties and variability 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 / Storm Loss

An analysis was undertaken for the 1% AEP storms 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 altering the "storm" rainfall loss in the XP-RAFTS model by ±20%. Specifically, the pervious storm losses were changed from the "design" value of 27.6mm to:

- "Wet" catchment: 18.4mm; and,
- "Dry" catchment: 36.8mm.

The median preburst rainfall was subsequently subtracted from the adjusted storm loss following the procedure summarised in Section 6.2.3 to develop revised burst losses for each storm duration. The revised burst losses were subsequently applied to the XP-RAFTS model and were used to re-simulate each of the 1% AEP and 5% AEP storms in accordance with

ARR2016. The revised 1% AEP and 5% AEP discharges were extracted from the results of the modelling and are included in **Appendix M**. Peak 1% AEP and 5% AEP discharges for the "base" conditions are also included in **Appendix M** for comparison.

The peak discharge comparison indicates that increasing the storm loss (reflecting a dryer catchment) decreases peak design discharges at most subcatchment locations. More specifically, peak 1% AEP discharges are predicted to reduce by about 12% at the "focus" locations shown in **Plate 18**, while 5% AEP discharges are predicted to reduce by about 5%, on average, at the focus locations.

Conversely, reducing the storm loss (reflecting a wetter catchment) is predicted to increase peak design discharges at most locations. Peak 5% AEP discharges are predicted to increase by 15%, while peak 1% AEP discharges are predicted to increase by about 14% at the focus locations.

Accordingly, the peak discharges do appear to be sensitive to the adopted storm losses.

The revised discharge hydrographs were then applied to the TUFLOW model and the TUFLOW model was used to re-simulate the 5% AEP and 1% AEP floods with the modified storm losses. Peak water levels were extracted from the results of the modelling and were compared against peak 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. The difference mapping is presented in **Plates O1** to **O4** in **Appendix O**. Decreases in "design" flood levels associated with the parameter change are shown in shades of blue and increases in flood levels are shown in shades of yellow and red.

The difference mapping was statistically analysed to determine the magnitude of changes in peak 5% AEP and 1% AEP water levels across areas of significant inundation depth (i.e., greater than 0.15 metres). The outcomes of this statistical assessment are shown in **Table 23** and **Table 24**. As shown in **Table 23** and **Table 24**, the flood level differences are reported as a series of percentiles. For example, the lower storm rainfall loss 90th percentile value of 0.06 metres indicates that 90% of the inundated areas are predicted to be exposed to changes in existing 1% AEP flood level of less than or equal to 0.06 metres (the 50th percentile values correspond to the median difference).

Peak 5% AEP and 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented **Table 25** and **Table 26**. The location where the flood levels were extracted from are shown in **Plate 18**.

The difference mapping shows that a lower initial loss value will produce increases in 1% AEP flood levels that are primarily concentrated along the main creek lines and overland flow paths. Conversely, the higher initial loss values will generate decreases in 1% AEP water levels that are again concentrated along the main creek lines and overland flow paths. The magnitude of the differences is typically less than 0.2 metres with the median (i.e., 50th percentile) difference being ±0.02 metres. The most significant differences are concentrated in the vicinity of major structures (e.g., culverts).

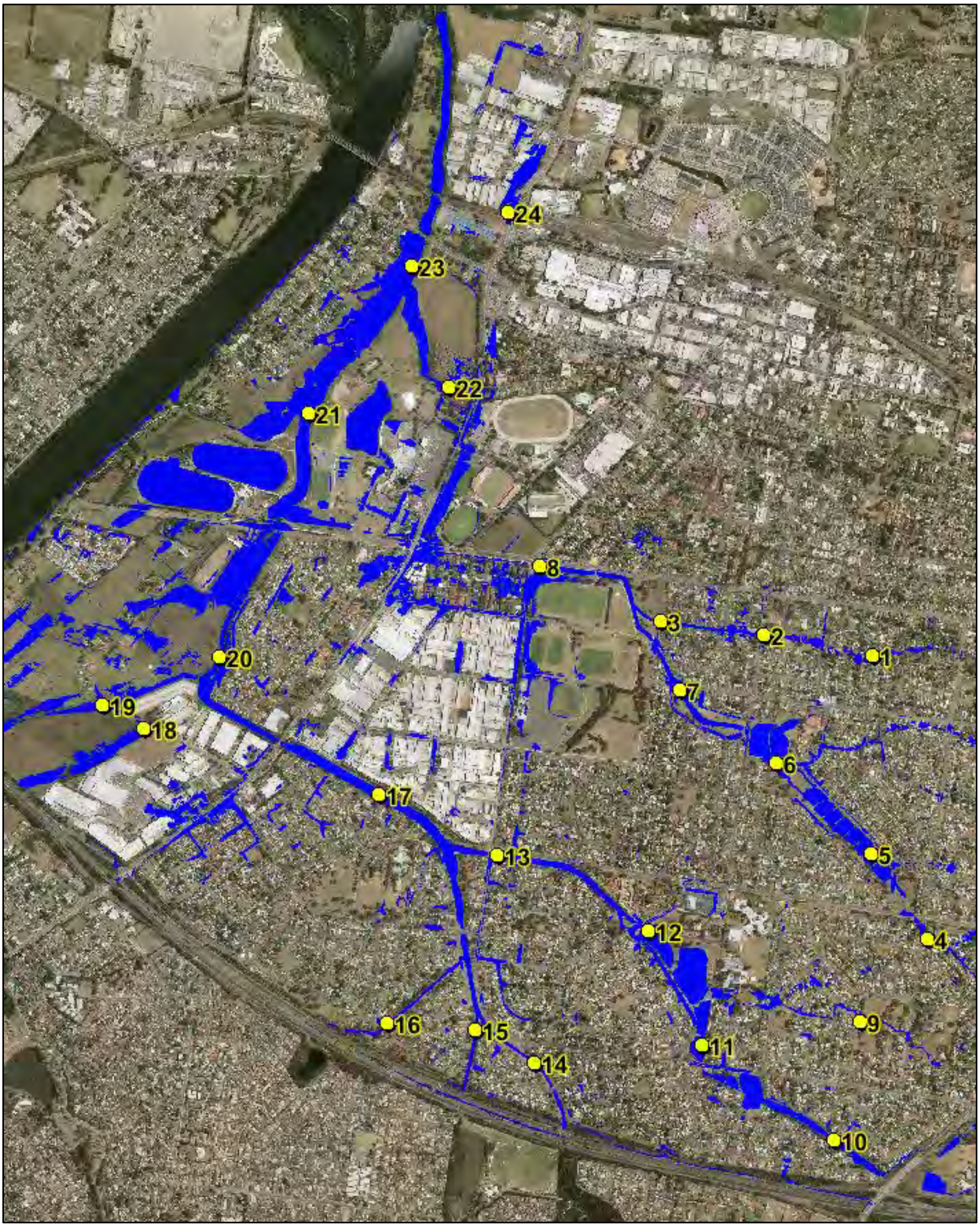


Plate 18 Flood level comparison locations

Table 23 Percentile Change in 5% AEP Flood Levels Associated with Changes to Model Input Parameters

Sensitivity Analysis	Percentile Change in Flood Level (m)								
	1st	5th	10th	25th	50th	75%	90th	95th	99th
Lower Storm Rainfall Loss	-0.02	0.00	0.00	0.00	0.01	0.03	0.05	0.08	0.27
Higher Storm Rainfall Loss	-0.34	-0.07	-0.04	-0.02	-0.01	0.00	0.00	0.00	0.00
Lower Continuing Loss Rates	-0.02	0.00	0.00	0.00	0.00	0.02	0.03	0.05	0.12
Higher Continuing Loss Rates	-0.20	-0.07	-0.04	-0.02	-0.01	0.00	0.00	0.00	0.01
Temporal pattern that generates higher discharge	-0.19	-0.02	-0.02	0.00	0.01	0.02	0.03	0.04	0.13
Temporal pattern that generates lower discharge	-0.29	-0.21	-0.18	-0.13	-0.05	-0.01	0.00	0.00	0.04
Manning's "n" reduced by 20%	-0.21	-0.11	-0.09	-0.03	0.00	0.01	0.03	0.07	0.34
Manning's "n" increased by 20%	-0.04	0.00	0.00	0.01	0.02	0.08	0.14	0.18	0.24
No Blockage of Structures	-0.06	-0.02	-0.01	0.00	0.00	0.01	0.01	0.01	0.02
Complete Blockage of Structures	-1.02	-0.13	-0.05	0.00	0.06	0.39	1.18	1.42	1.71
1% AEP Nepean River tailwater	0.00	0.00	0.00	0.00	0.58	1.67	1.89	1.90	2.83

Table 24 Percentile Change in 1% AEP Flood Levels Associated with Changes to Model Input Parameters

Sensitivity Analysis	Percentile Change in Flood Level (m)								
	1st	5th	10th	25th	50th	75%	90th	95th	99th
Lower Storm Rainfall Loss	0.00	0.00	0.00	0.01	0.02	0.05	0.06	0.07	0.11
Higher Storm Rainfall Loss	-0.12	-0.08	-0.07	-0.06	-0.02	-0.01	0.00	0.00	0.00
Lower Continuing Loss Rates	-0.01	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.04
Higher Continuing Loss Rates	-0.05	-0.02	-0.02	-0.01	0.00	0.00	0.00	0.00	0.03
Temporal pattern that generates higher discharge	-0.02	-0.01	0.00	0.00	0.01	0.08	0.10	0.12	0.16
Temporal pattern that generates lower discharge	-0.16	-0.10	-0.07	-0.04	-0.02	0.00	0.01	0.03	0.12
Manning's "n" reduced by 20%	-0.17	-0.10	-0.09	-0.01	0.00	0.01	0.02	0.03	0.10
Manning's "n" increased by 20%	-0.04	-0.01	0.00	0.01	0.01	0.06	0.09	0.10	0.15
No Blockage of Structures	-0.04	-0.02	-0.01	0.00	0.00	0.01	0.02	0.02	0.04
Complete Blockage of Structures	-0.50	-0.17	-0.05	0.00	0.06	0.38	1.07	1.32	1.47
1% AEP Nepean River tailwater	0.00	0.00	0.00	0.00	0.00	1.25	1.78	1.89	2.79
0.2% AEP Nepean River tailwater	0.00	0.00	0.00	0.00	1.70	3.54	4.27	4.35	4.54
PMF Nepean River tailwater	0.00	0.00	0.00	0.00	4.60	6.32	6.92	7.16	7.44

Table 25 Peak 5% AEP Flood Levels from Sensitivity Simulation at Various Locations across the Catchment

Location (refer to Plate 18 for locations)	**Peak Flood Level** (mAHD)											
	"Base" Case	Lower Storm Loses	Higher Storm Loses	Lower Continuing Loses	Higher Continuing Loses	Higher Q Temporal Pattern	Lower Q Temporal Pattern	Lower Manning's "n"	Higher Manning's "n"	No Blockage	Complete Blockage	1% AEP Nepean River tailwater
1	45.06	45.10	45.00	45.06	45.05	45.10	45.00	45.05	45.06	45.06	45.12	44.62
2	37.95	37.98	37.93	37.95	37.95	37.96	37.93	37.95	37.95	37.95	38.04	37.87
3	32.79	32.81	32.76	32.79	32.79	32.80	32.75	32.78	32.79	32.79	32.87	32.63
4	49.65	49.73	49.61	49.65	49.65	49.70	49.52	49.66	49.66	49.65	49.83	49.45
5	44.63	44.68	44.61	44.63	44.63	44.64	44.55	44.63	44.64	44.64	44.63	44.56
6	39.31	39.31	39.30	39.32	39.29	39.30	39.21	39.32	39.33	39.31	39.37	39.26
7	34.69	34.73	34.60	34.71	34.69	34.70	34.55	34.73	34.74	34.68	34.86	34.58
8	28.62	28.88	28.46	28.72	28.65	28.65	28.34	28.70	28.43	28.63	29.20	28.05
9	49.75	49.84	49.45	49.76	49.74	49.79	49.44	49.75	49.76	49.67	49.89	49.31
10	50.14	50.18	50.09	50.14	50.13	50.16	50.06	50.13	50.15	50.14	49.99	50.03
11	41.05	41.05	41.05	41.05	41.04	41.05	40.98	41.05	41.06	41.05	41.02	40.97
12	37.42	37.43	37.41	37.43	37.40	37.38	37.37	37.44	37.45	37.41	37.59	37.39
13	32.18	32.39	31.95	32.18	32.17	32.26	31.88	32.19	32.19	32.16	32.99	31.78
14	35.34	35.29	35.31	35.30	35.28	35.30	35.20	35.01	35.24	35.30	35.15	35.08
15	34.54	34.54	34.54	34.56	34.50	34.56	34.42	34.49	34.61	34.54	34.75	34.57
16	36.33	36.38	36.31	36.33	36.33	36.36	36.28	36.31	36.35	36.33	36.30	36.31
17	29.58	29.59	29.57	29.61	29.56	29.58	29.46	29.51	29.69	29.59	29.90	29.52
18	26.59	26.59	26.59	26.59	26.59	26.59	26.59	26.60	26.58	26.59	26.59	27.20
19	26.14	26.15	26.12	26.15	26.12	26.14	26.14	26.13	26.19	26.15	26.17	26.69
20	26.63	26.65	26.61	26.68	26.58	26.64	26.33	26.42	26.84	26.63	25.59	26.68
21	23.54	23.55	23.54	23.57	23.52	23.57	23.38	23.44	23.69	23.56	24.61	26.06
22	26.00	26.02	25.95	26.00	26.00	26.00	25.94	26.00	25.99	26.01	25.64	26.08
23	23.16	23.16	23.15	23.17	23.14	23.17	23.04	23.09	23.26	23.16	24.60	26.08
24	24.36	24.39	24.32	24.36	24.35	24.39	24.29	24.35	24.36	24.36	24.47	26.02

Table 26 Peak 1% AEP Flood Levels from Sensitivity Simulation at Various Locations across the Catchment

Location (refer to Plate 17 for locations)	**Peak Flood Level** (mAHD)													
	"Base" Case	Lower Storm Loses	Higher Storm Loses	Lower Continuing Loses	Higher Continuing Loses	Higher Q Temporal Pattern	Lower Q Temporal Pattern	Lower Manning's "n"	Higher Manning's "n"	No Blockage	Complete Blockage	1% AEP Nepean River tailwater	0.2% AEP Nepean River tailwater	PMF Nepean River tailwater
1	45.17	45.19	45.14	45.17	45.17	45.17	45.10	45.17	45.19	45.17	45.25	45.17	45.17	45.17
2	38.03	38.05	38.00	38.03	38.03	38.04	38.00	38.03	38.04	38.03	38.10	38.03	38.03	38.03
3	32.84	32.86	32.83	32.84	32.84	32.86	32.83	32.84	32.85	32.84	32.93	32.84	32.84	32.84
4	49.84	49.88	49.80	49.85	49.84	49.86	49.79	49.85	49.85	49.84	49.97	49.84	49.84	49.84
5	44.73	44.76	44.71	44.73	44.73	44.74	44.70	44.73	44.74	44.73	44.74	44.73	44.73	44.73
6	39.46	39.48	39.43	39.47	39.46	39.46	39.35	39.47	39.48	39.46	39.45	39.46	39.46	39.46
7	34.88	34.92	34.80	34.89	34.87	34.90	34.79	34.90	34.90	34.87	34.91	34.88	34.88	34.88
8	28.85	28.98	28.78	28.97	29.01	29.02	28.98	29.01	28.87	28.92	29.28	28.85	28.90	31.55
9	49.90	49.91	49.86	49.90	49.89	49.91	49.85	49.89	49.90	49.88	49.97	49.90	49.90	49.90
10	50.22	50.23	50.20	50.23	50.22	50.22	50.18	50.21	50.23	50.23	50.10	50.22	50.22	50.22
11	41.18	41.19	41.15	41.18	41.17	41.18	41.09	41.17	41.19	41.18	41.08	41.18	41.18	41.18
12	37.63	37.67	37.56	37.63	37.62	37.65	37.56	37.63	37.65	37.63	37.62	37.63	37.63	37.63
13	32.65	32.75	32.53	32.65	32.64	32.77	32.43	32.67	32.65	32.62	33.01	32.65	32.65	32.65
14	35.43	35.48	35.35	35.44	35.41	35.50	35.35	35.39	35.47	35.43	35.32	35.43	35.43	35.43
15	34.73	34.77	34.69	34.74	34.73	34.79	34.68	34.66	34.81	34.73	34.87	34.73	34.73	34.73
16	36.43	36.44	36.42	36.44	36.43	36.43	36.38	36.42	36.45	36.44	36.41	36.43	36.43	36.43
17	29.73	29.78	29.68	29.73	29.72	29.83	29.69	29.62	29.84	29.75	29.94	29.73	29.73	31.52
18	26.59	26.59	26.59	26.59	26.59	26.59	26.59	26.60	26.59	26.59	26.60	27.19	28.68	31.61
19	26.17	26.20	26.17	26.17	26.16	26.24	26.20	26.16	26.24	26.18	26.24	26.67	28.65	31.58
20	26.90	26.94	26.83	26.90	26.88	26.98	26.84	26.72	27.01	26.91	25.70	26.90	28.73	31.60
21	23.77	23.82	23.70	23.77	23.75	23.86	23.75	23.67	23.85	23.78	24.75	25.03	27.44	30.27
22	26.05	26.06	26.04	26.05	26.05	26.04	26.04	26.07	26.04	26.06	25.72	26.10	28.40	31.21
23	23.35	23.40	23.28	23.35	23.33	23.43	23.31	23.28	23.41	23.36	24.78	24.77	26.90	29.65
24	24.44	24.46	24.43	24.44	24.44	24.44	24.45	24.43	24.45	24.44	24.82	25.69	27.62	30.51

8.2.2 Continuing Loss Rate

An analysis was also undertaken to assess the sensitivity of the results generated by the TUFLOW model to variations in the adopted continuing loss rates. Accordingly, the continuing loss rates within the TUFLOW model 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 0.5mm/hr for impervious areas.
- Decreased Continuing Loss Rates: 1.5mm/hr for pervious areas and 0mm/hr for impervious areas.

The modified continuing loss rates were subsequently applied to the XP-RAFTS model and were used to re-simulate each of the 1% AEP and 5% AEP storms in accordance with ARR2016. The revised 1% AEP and 5% AEP discharges were extracted from the results of the modelling and are included in **Appendix M**. Peak 1% AEP and 5% AEP discharges for the "base" conditions are also included in **Appendix M** for comparison.

The peak discharge comparison indicates that increasing the continuing loss rate reduces peak design discharges at most locations. However, the reductions are predicted to be fairly minor, with peak 5% AEP discharges reducing by about 2% and peak 1% AEP discharge reducing by 1%, on average.

Reducing the continuing loss rate is predicted to increase peak design discharges at most locations. Again, the changes are predicted to be small with peak 5% AEP discharges increasing by about 2% and peak 1% AEP discharge increasing by about 1%, on average.

Accordingly, the hydrologic model does not appear to be particularly sensitive to changes in the adopted continuing loss rate.

The revised discharge hydrographs were then applied to the TUFLOW model and the TUFLOW model was used to re-simulate the 5% AEP and 1% AEP floods with the modified continuing loss rates. 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. The difference mapping is presented in **Plates O5** to **O8** in **Appendix O**.

The difference mapping was statistically analysed to determine the magnitude of changes in peak 5% AEP and 1% AEP water levels across areas of significant inundation depth (i.e., greater than 0.15 metres). The outcomes of this statistical assessment are shown in **Table 23** and **Table 24**.

Peak 5% AEP and 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented **Table 25** and **Table 26**.

The results of the sensitivity analysis show that the TUFLOW model is relatively insensitive to changes in continuing loss rates. More specifically, **Table 24** shows that only small changes in 1% AEP flood levels are predicted with the modified continuing loss rates. In all cases, the 99th percentile change in 1% AEP flood levels are predicted to be less than 0.05 metres.

Therefore, it can be concluded that any uncertainties associated with the adopted continuing loss rates are unlikely to have a significant impact on the results generated by the TUFLOW model.

8.2.3 Temporal Pattern

The box plots presented in **Appendix L** shows that the temporal (i.e., time varying) distribution of rainfall can have a notably impact on peak design discharges throughout the catchment. For example, the peak 1% AEP discharge for Surveyors Creek at Mulgoa Road can vary between 97 m^3/s and 127 m^3/s depending on the temporal pattern that is used.

A temporal pattern that provided a peak discharge roughly in the middle of the range was adopted as part of the 'base' design flood simulations. However, it was considered important to gain an understanding of how variations in the rainfall temporal pattern may impact on the results generated by the model. Therefore, additional simulations were completed with temporal patterns that generated peak discharges at the "upper" and "lower" end of the discharge range.

A review of the temporal patterns was completed to identify the temporal pattern that most commonly produced the highest and lowest peak 1% AEP and 5% AEP discharges. This review yielded the selection of the following temporal patterns:

- 5% AEP Flood:
  - Highest discharge temporal patterns: 4572 (60 min) & 4591 (360min)
  - Lowest discharge temporal patterns: 4563 (60 min) & 4726 (360min)
- 1% AEP Flood:
  - Highest discharge temporal patterns: 4535 (45 min), 4499 (120min) & 4719 (360min)
  - Lowest discharge temporal patterns: 4526 (45 min), 4571 (120min) & 4596 (360min)

Appendix M summarises the peak discharges at each subcatchment that are generated using the temporal patterns listed above. The 'base' peak discharges are also listed for comparison.

The comparison presented in **Appendix M** shows that the temporal patterns that produce lower peak discharges will generate peak 5% AEP and 1% AEP discharges that are 25% and 13% lower (on average) respectively relative to the base peak discharges.

Adopting a temporal pattern that produces high peak discharges is predicted to increase peak 5% AEP discharges by 18% and peak 1% AEP discharges by 10%, on average. This indicates that the 'base' discharge estimates are slightly biased towards the upper end of the discharge range (i.e., slightly closer to the "higher" discharges than the "lower" discharges).

The revised discharge hydrographs were then applied to the TUFLOW model and the TUFLOW model was used to re-simulate the 5% AEP and 1% AEP flood with the modified temporal pattern. Flood level difference mapping was prepared to display the impacts of the temporal pattern modifications and are presented in **Plates O9** and **O12** in **Appendix O**.

The difference mapping was statistically analysed to determine the magnitude of changes in peak 5% AEP and 1% AEP water levels across areas of significant inundation depth (i.e., greater than 0.15 metres). The outcomes of this statistical assessment are shown in **Table 23** and **Table 24**.

Peak 5% AEP and 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented **Table 25** and **Table 26**.

The results of the sensitivity analysis show that the TUFLOW model is sensitive to changes in the temporal pattern. More specifically, **Table 24** shows that although the median difference is relatively minor (± 0.02m), 95th percentile differences are predicted to reach (± 0.1m). Some localised increases/decreases are predicted to exceed ±0.2 m.

It was also noted that the "higher" and "lower" temporal patterns did not generate global increases and decreases across all sections of the catchment. For example, the "lower" temporal pattern actually produced higher water levels across part sections of the lower catchment.

Accordingly, the peak design discharges and water level results are sensitive to the adopted temporal pattern. However, it should be acknowledged that the chance of the 1% AEP rainfall occurring in conjunction with the "worst case" temporal pattern is likely be rarer than 1% AEP. Accordingly, it is considered that the adopted temporal pattern better maintains AEP neutrality.

8.2.4 Manning's "n"

Manning's "n" roughness coefficients are used to describe the resistance to flow afforded by different land uses and 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 20% increase and a 20% decrease in the adopted design Manning's "n" values and additional 5% AEP and 1% AEP simulations were completed with the modified "n" values (no changes to hydrology were completed as part of this assessment). Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping, which is presented in **Plates O13** to **O16** in **Appendix O**. 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., greater than 0.15 metres). The outcomes of this statistical assessment are shown in **Table 23**. The outcomes of this statistical assessment are shown in **Table 23** and **Table 24**.

Peak 5% AEP and 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented **Table 25** and **Table 26**.

In general, the changes in 1% AEP flood levels are predicted to be less than 0.1 metres, but localised increases/decreases of 0.2 metres are predicted. It is noted that reducing "n" values will typically reduce flood levels. However, there are some areas where the more rapid response of rainfall is predicted to generate localised increases in flood levels.

Although localised changes in flood level of more than 0.1 metres are predicted at isolated locations, the median change in flood level is predicted to be no greater than 0.01 metres. As a result, it is considered that the overall model is relatively insensitive to changes in Manning's 'n' values.

8.2.5 Hydraulic Structure Blockage

As discussed in Section 6.2.3, blockage factors were applied to all bridges, culverts and stormwater inlets as part of the design flood simulations. However, as it is not known which structures will be subject to what percentage of blockage during any particular flood, additional TUFLOW simulations were completed to determine the impact that alternate blockage scenarios would have on flood behaviour. Specifically, additional simulations were undertaken with no blockage as well as complete blockage of all stormwater inlets, bridges and culverts. As part of this assessment, no changes to blockage were made for any of the M4 Motorway culverts (i.e., no blockage was applied to these culverts to maximise the potential impacts across the study area proper).

Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping, which is presented in **Plates O17** to **O20** in **Appendix O**. The difference mapping was statistically analysed to determine the magnitude of changes in peak 1% AEP water levels. The outcomes of this statistical assessment are shown in **Table 23** and **Table 24**.

Peak 5% AEP and 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented **Table 25** and **Table 26**.

Plates O19 and **O20** shows that complete blockage will cause some significant changes to 5% AEP and 1% AEP flood levels. Design flood levels are predicted to increase by over 2 metres at some locations and are driven by the significantly elevated embankments at some locations. There are predicted to be some commensurate decreases in water level downstream of these significant embankment structures which are associated with the "damming" effect provided by the embankment (this is most significant downstream of the Surveyors Creek crossing of Mulgoa Road).

In general, changes to stormwater inlet blockage are only predicted to have a relatively small impact on 5% AEP and 1% AEP water levels across the majority of the study area. This is likely associated with the stormwater system only having sufficient capacity to carry a relatively small proportion of the overall flow during a large storm event (such as the 1% AEP flood).

Consequently, changes to stormwater inlet blockage generally do not result in a large change in the amount of water travelling overland. The only exception to this occurs in the vicinity of the "trunk" pipe systems.

The results of the blockage sensitivity analysis do show that the model results are sensitive to variations in blockage in the immediate vicinity of major hydraulic structures, particularly if complete blockage of structures occurs. This outcome emphasises the need to ensure key drainage infrastructure and bridges and culverts are well maintained (i.e., debris is removed on a regular basis).

8.2.6 Nepean River Level

The Peach Tree and Lower Surveyors Creeks catchment drains into the Nepean River, which forms the downstream boundary of the catchment. The "base" simulations assumed that a 5% AEP flood (peak Nepean River water level at Peach Tree Creek confluence = 22.73 mAHD) was occurring along the Nepean River at the same time as a 1% AEP flood within the local catchment. However, if the prevailing water level within Nepean River at the time of a local catchment flood was different, it has the potential to impact on results across the downstream sections of the Peach Tree and Lower Surveyors Creeks catchment.

Therefore, additional 1% AEP sensitivity simulations were completed to assess the sensitivity of the model results to variations in the adopted Nepean River water level. The simulations included:

- 1% AEP water level in the Nepean River (Peak water level = 25.4 mAHD at the Peach Tree Creek confluence);
- 0.2% AEP water level in the Nepean River (Peak water level = 27.0 mAHD at the Peach Tree Creek confluence); and,
- PMF water level in the Nepean River (Peak water level = 30.3 mAHD at the Peach Tree Creek confluence)

In addition, a 5% AEP local catchment flood simulation was completed with a 1% AEP Nepean River water level. This is equivalent to the "scenario 4" assessed as part of the *'Peach Tree Creek Flood Study'* (1994) and has formed the basis for defining 1% AEP flood behaviour across much of the lower catchment areas over the past ~30 years.

To provide a reliable description of the propagation of floodwaters across the study area in the model, it was necessary for the Nepean River water level to be defined as a time varying water level along the full length of the river. The variation in water level with respect to time along the river was defined using stage hydrographs extracted from the results of flood modelling completed for the 'Nepean River Flood Study: Exhibition Draft Report' (Advisian, 2017).

Peak floodwater depths were extracted from the results of the Nepean River sensitivity modelling and are provided in **Figures 57** to **60**.

Peak flood levels were extracted from the results of the Nepean River modelling and were used to prepare flood level difference mapping, which is presented in **Plates O21** to **O24** in **Appendix O** for the 1% AEP local catchment flood and **Plate O25** for the 5% AEP local catchment flood. 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., greater than 0.15 metres). The outcomes of this statistical assessment are shown in **Table 23** and **Table 24**.

Peak 5% AEP and 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented **Table 25** and **Table 26**.

Peak floodwater levels were also extracted along the centreline of Peach Tree and Surveyors Creek and were used to prepare a floodwater surface profile for each of the Nepean River sensitivity simulations. The water surface profiles are provided in **Plate 19** and **Plate 20**.

The difference mapping shows that the adopted Nepean River water level can have a significant impact on flood extents and levels across the downstream sections of the study area. More specifically:

- For the 1% AEP Nepean River scenario, peak flood levels are predicted to increase by about 1 metre across the lower catchment area with some area exposed to increases of over 2 metres. The differences are primarily contained to the area west of Mulgoa Road.
- For the 0.2% AEP Nepean River scenario, peak flood levels are predicted to increase by over 3 metres across the lower catchment area (increases of over 4 metres are predicted at some locations). The differences are primarily contained to the area west of Mulgoa Road. However, some areas immediately east of Mulgoa Road are also predicted to be impacted.
- For the PMF Nepean River scenario, peak flood levels are predicted to increase by more than 5 metres across the lower catchment area with some areas predicted to be exposed to increases of over 7 metres. The differences extend across a large area extending from the banks of the Nepean River as far east as Jamison Park.

For the 1% AEP Nepean River flood with 5% AEP local catchment event, flood levels east of Mulgoa Road are predicted to be lower as flooding across this section of the catchment is dominated by local catchment runoff. The difference between the 5% AEP and 1% AEP levels in these areas is typically 0.1 and 0.15 metres. In areas to the west of Mulgoa Road, Nepean River 1% AEP flood levels dominate. Peak 1% AEP Nepean River levels are predicted to be more than 1.5 metre higher than 5% AEP levels, with some areas being exposed to flood level differences that exceed 2 metres. The water surface profile provided in **Plate 19** also indicates that the "transition point" between local catchment dominated 1% AEP flood level and local catchment dominated 1% AEP flood levels is located just downstream of the confluence between Peach Tree Creek and Surveyors Creek.

Overall, it can be concluded that flood levels across the downstream sections of the catchment are sensitive to changes in the adopted Nepean River level. However, flood level impacts across the upstream sections of the catchment are predicted to be negligible.

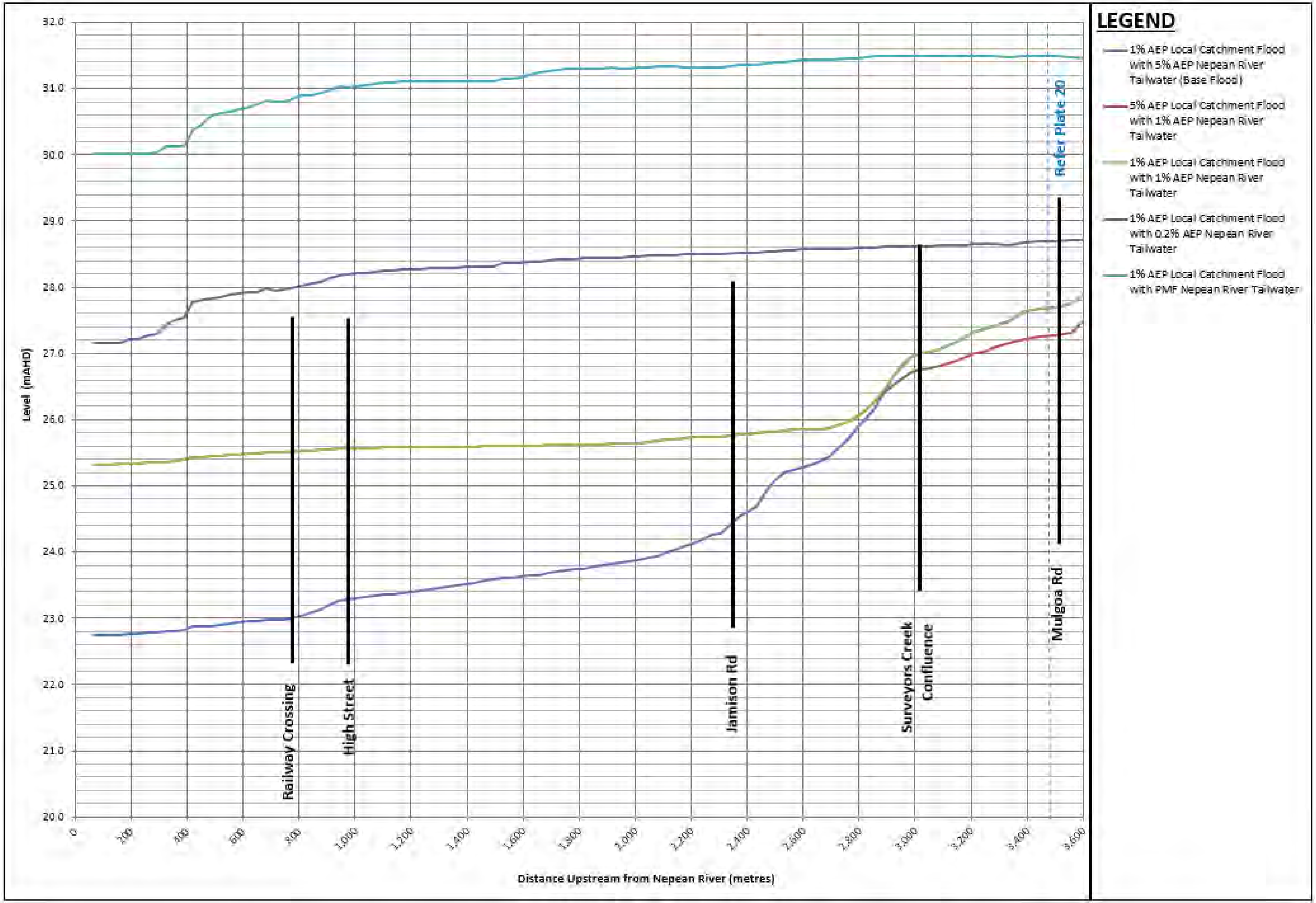


Plate 19 Floodwater Surface Profiles for Nepean River Sensitivity Simulations (lower catchment)

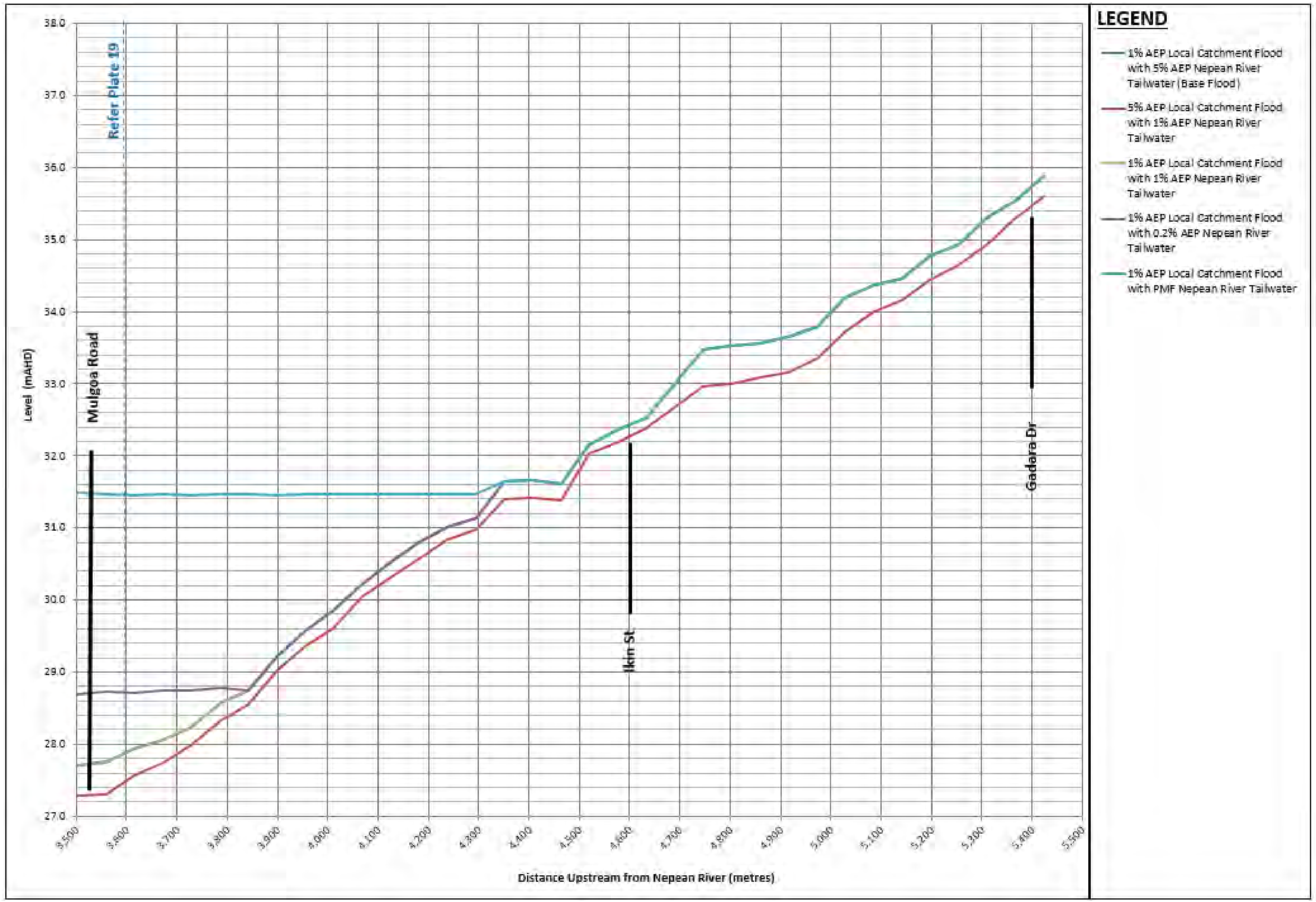


Plate 20 Floodwater Surface Profiles for Nepean River Sensitivity Simulations (upper catchment)

8.3 Australian Rainfall & Runoff 1987

Flood studies across the Penrith LGA over the last three decades have been prepared in accordance with *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987) (ARR1987). In late 2016, a revised version of *'Australian Rainfall and Runoff'* was released (referred to herein as ARR2016). The current study has been prepared in accordance with ARR2016, which reflects application of the more thorough hydrologic procedures as well as an additional 30 years of rainfall information. Nevertheless, it was considered important to understand how results produced based upon ARR2016 may differ from those generated using ARR1987. Therefore, an additional sensitivity assessment was completed to confirm the impact that the revised hydrologic procedures may have on design flood behaviour across the study area. The outcomes of this assessment is contained within **Appendix N**.

The hydrologic information presented in **Appendix N** shows that ARR1987 produces higher peak design discharge estimates relative to ARR2016. The ARR1987 discharges are typically 15 to 20% higher that the ARR2016 discharges. This is considered to be primarily associated with the higher burst losses being used as part of ARR2016.

The TUFLOW results presented in **Appendix N** also shows the ARR1987 is predicted to produce higher flood levels relative to ARR2016. Along major watercourse, the ARR1987 levels are typically a minimum of 0.1 metres higher than ARR2016 levels. Localised increases of more than 0.3 metres are predicted in the vicinity of major hydraulic controls (e.g., roadway embankments/culverts).

Accordingly, there are some notable difference between flood behaviour defined under ARR1987 versus ARR2016. However, ARR2016 takes advantage of a greater amount of historic rainfall information and employs that latest available research in deriving the design flood estimates. Therefore, it is considered that the flood estimates defined under ARR2016 are reasonable and improve upon the flood estimates provided by ARR1987.

8.4 Future Catchment Development

The Peach Tree and Surveyors Creeks catchment is already significantly developed, and a representation of this existing development was included as part of the 'base' flood results. In addition, the 'base' flood information provided in Section 6 included a representation of developments that are likely to be implemented in the immediate future (e.g., The Northern Road upgrade).

Nevertheless, there are some sections of the catchment where there is potential for further, new development to occur (e.g., Glenmore Park). In addition, there is potential for re-development/intensification to occur in areas that are already developed (e.g., construction of granny flats). This potential development may alter the hydrologic and hydraulic results presented in this report. Accordingly, an additional simulation was completed to quantify the potential impacts that future development may have on the results of the modelling.

Firstly, those areas that are currently undeveloped but are likely to be developed in the future were identified. This was completed by reviewing land use zoning information relative to contemporary aerial imagery. This review identified two major areas were new development is likely to occur in the future:

- Glenmore Park (refer **Plate 21**): Zoned residential
- Penrith Industrial Area (refer **Plate 22**): Zoned industrial

As the future characteristics of these areas is not known, assumptions were made regarding the likely land use composition. This information, in turn, was used to calculate weighted average impervious and pervious "n" values for each land use (refer **Table 27**).

Table 27 Adopted land use information for future development assessment

Future Land Use	Composition	Impervious	Pervious "n"
Residential	50% building, 25% grass, 20% concrete, 5% trees	70%	0.029
Industrial	70% building, 15% concrete, 10% car park, 5% grass	95%	0.023

For the balance of the catchment (i.e., those areas that are already developed but where intensification of development may occur in the future), it was assumed that the current impervious proportion would increase by 15% up to a maximum of 100%. For example, an existing residential allotment that is currently 60% impervious would be increased to 75% impervious while an existing industrial allotment that is currently 95% impervious would be "capped" at 100% impervious. The updated total impervious areas for each subcatchment were subsequently modified to an effective impervious area using the same 0.85 adjustment factor that was adopted for the base design simulations.

The updated impervious and pervious "n" values were applied to an updated "ultimate catchment development" version of the XP-RAFTS model. The updated model was used to re-simulate the 5% AEP and 1% AEP storms as well as the PMF under potential future catchment development conditions. Peak discharges extracted from the results of the revised hydrologic assessment are presented in **Appendix M**. Peak 1% AEP and 5% AEP discharges for current catchment development conditions are also included in **Appendix M** for comparison.

The discharge comparison indicates that the adopted catchment modifications are predicted to generate small increases in peak 5% AEP and 1% AEP discharges. Peak 5% AEP discharges are predicted to increase by around 4% at ley locations across the study area while peak 1% AEP discharges are predicted to increase by just under 3%, on average, at key locations.

The future catchment development discharge hydrographs were also applied to the TUFLOW model to confirm that nature and extent of potential changes in flood behaviour associated with future development. Flood level difference mapping was prepared for the 1% AEP event and is presented in **Plate O15** in **Appendix O**.

Plate 21 Area of Glenmore Park that was assumed to be developed as part of future catchment development assessment

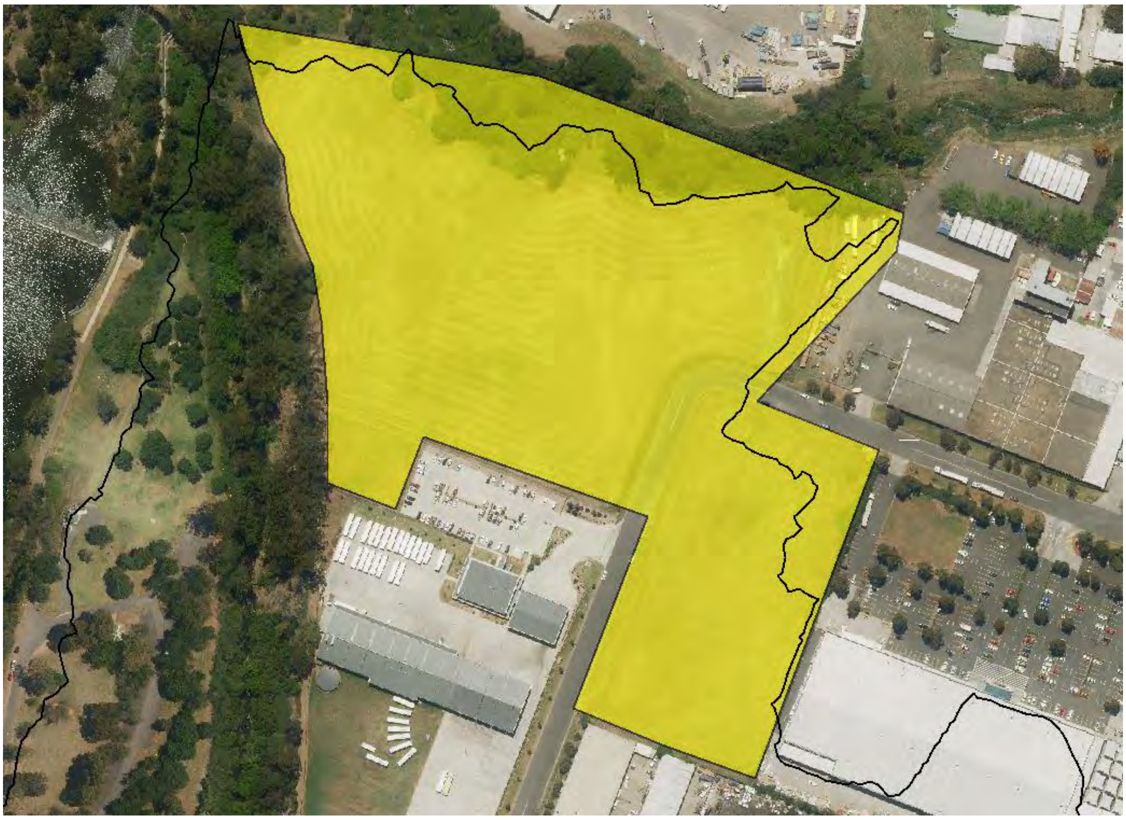


Plate 22 Penrith industrial area that was assumed to be developed as part of future catchment development assessment

The difference map shows that future development is predicted to generate small increases in 1% AEP flood levels. In general, the increases in flood levels are predicted to be less than 0.05 metres, although some localised increases of up to 0.1 metres are anticipated.

The results of the revised hydrologic and hydraulic modelling indicate that the existing detention basins located throughout the catchment are likely serving to minimise the potential impacts of increases in runoff associated with future development. Accordingly, future development across the catchment is only predicted to have a relatively small adverse impact on existing flood behaviour.

8.5 Climate Change Analysis

Climate change refers to a significant and lasting change in weather patterns arising from both natural and human induced processes. The Office of Environment and Heritage's *'Practical Consideration of Climate Change'* states that climate change is expected to have adverse impacts on sea levels and rainfall intensities in the future.

Although there is considerable uncertainty associated with the impact that climate change may have on rainfall, it was considered important to provide an assessment of the potential impact that climate change may have on the current flood risk across the study area. The interim climate change factors published in Australian Rainfall and Runoff (Geoscience Australia, 2016) indicates that a 9.1% increase in rainfall is the best estimate of likely rainfall intensities increases by 2090 under Representative Concentration Pathway scenario 4.5 (RCP4.5) (i.e., greenhouse gas emissions are reduced in the future). Under RCP 8.5 conditions (i.e., current greenhouse gas emissions increase in the future), rainfall intensities would likely increase by 18.6% by 2090 (refer **Plate 23**).

The rainfall intensity increases were applied to the current 1% AEP rainfall depths and the revised rainfall was routed through the XP-RAFTS model. The revised 1% AEP discharges were extracted from the results of the modelling and are included in **Appendix M**. Peak 1% AEP discharges for the "base" conditions are also included in **Appendix M** for comparison.

Interim Climate Change Factors

Values are of the format temperature increase in degrees Celcius (% increase in rainfall)

	RCP 4.5	RCP6	RCP 8.5
2030	0.892 (4.5%)	0.775 (3.9%)	0.979 (4.9%)
2040	1.121 (5.6%)	1.002 (5.0%)	1.351 (6.8%)
2050	1.334 (6.7%)	1.28 (6.4%)	1.765 (8.8%)
2060	1.522 (7.6%)	1.527 (7.6%)	2.23 (11.2%)
2070	1.659 (8.3%)	1.745 (8.7%)	2.741 (13.7%)
2080	1.78 (8.9%)	1.999 (10.0%)	3.249 (16.2%)
2090	1.825 (9.1%)	2.271 (11.4%)	3.727 (18.6%)

Plate 23 Adopted rainfall intensity increase for climate change simulation (Geoscience Australia, 2016)

The peak discharge comparison indicates that increases in rainfall will increase peak discharges throughout the catchment. The peak 1% AEP discharges are predicted to increase by 13%, on average, at key locations if rainfall intensities were to increase by 9.1%. If rainfall intensities were to increase by 18.6%, peak 1% AEP discharge could be expected to increase by 26%, on average, at key locations.

The revised discharge hydrographs were then applied to the TUFLOW model and the TUFLOW model was used to re-simulate the 1% AEP flood with the rainfall intensity increases. 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. The difference mapping is presented in **Plate 24** and **Plate 25**.

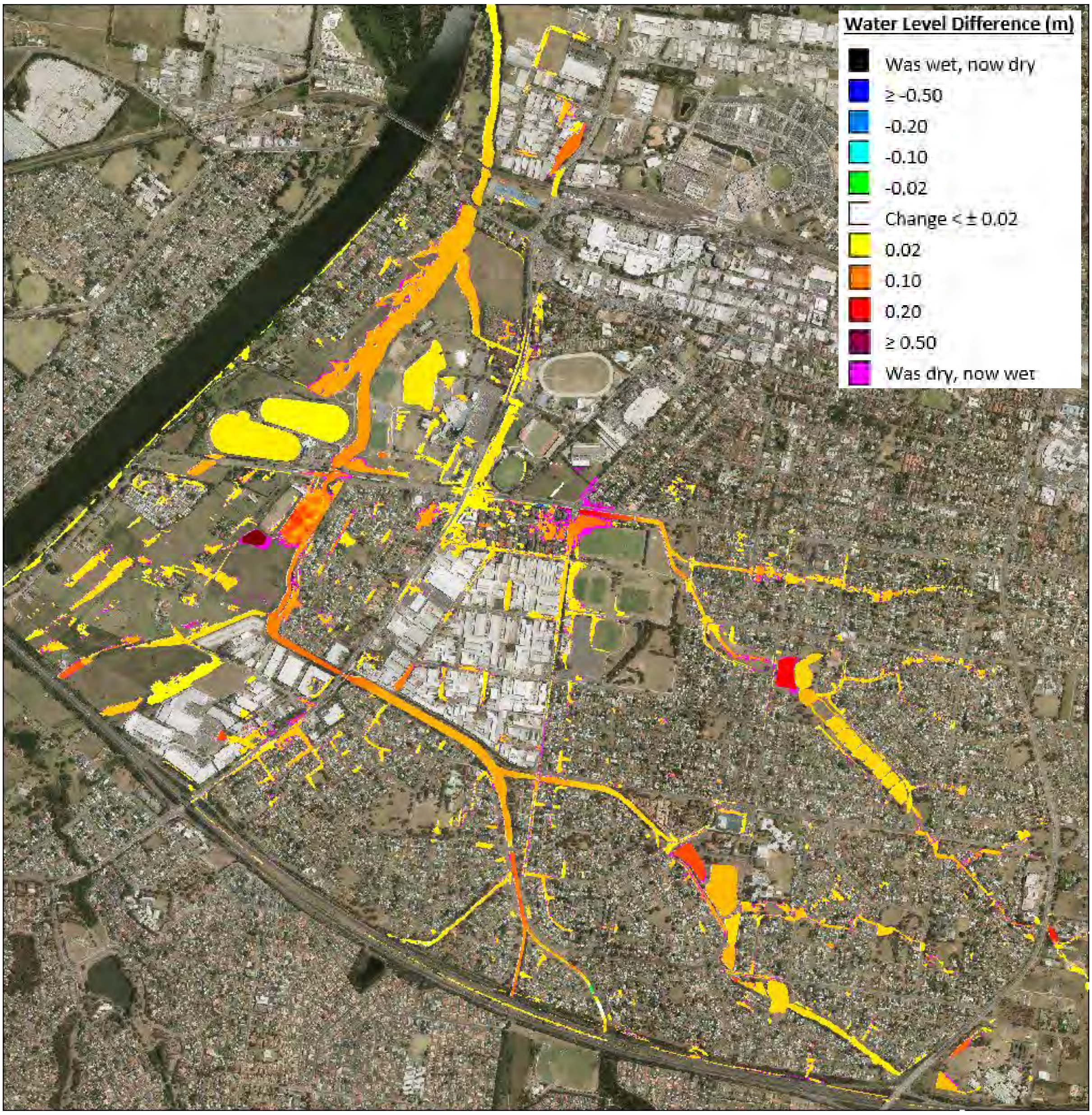


Plate 24 1% AEP Flood level difference map with 9.1% Increase in rainfall

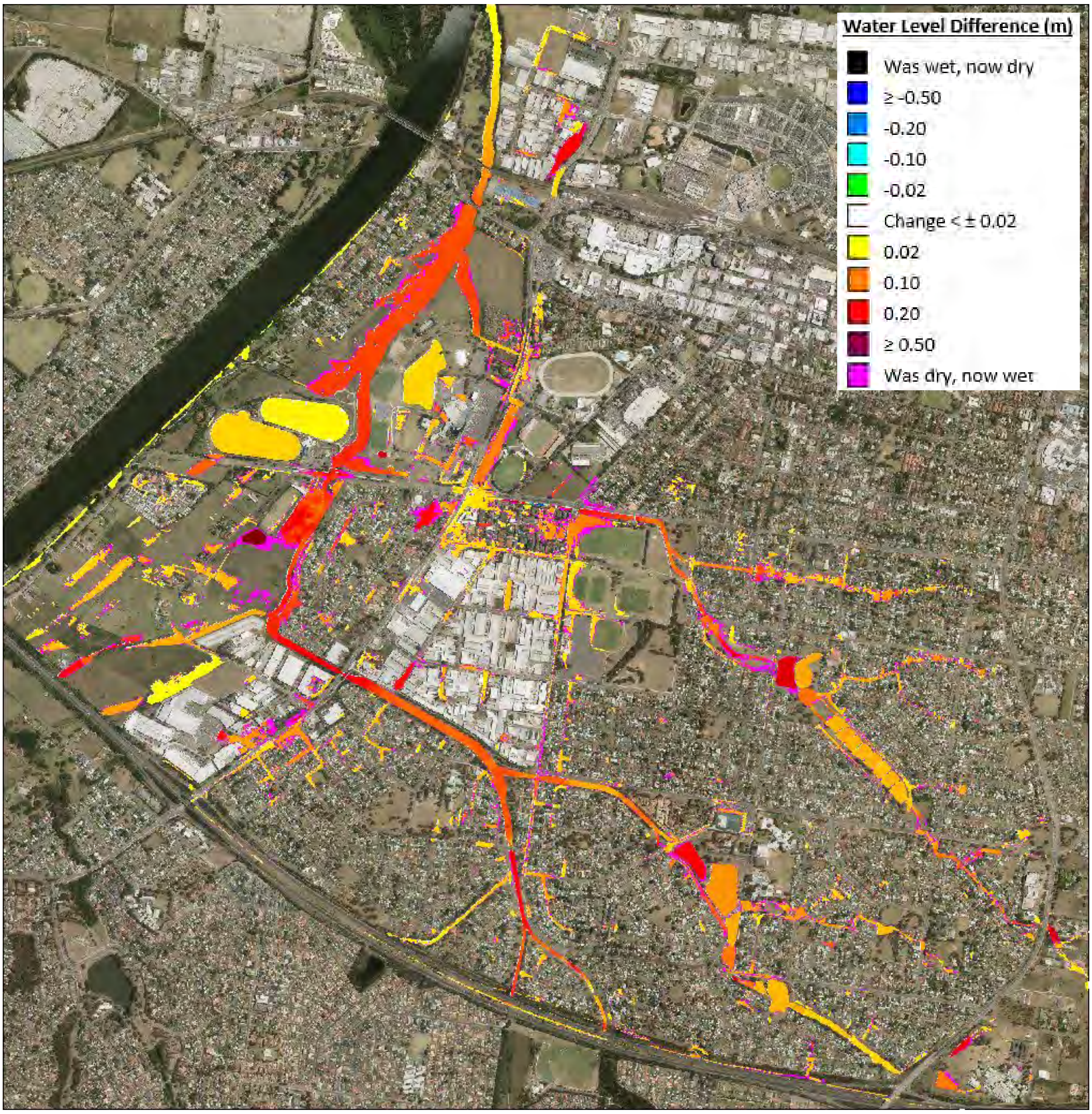


Plate 25 1% AEP Flood level difference map with 18.6% Increase in rainfall

Plate 24 and **Plate 25** show that rainfall increases will likely increase current 1% AEP flood level estimates across the study area. The 9.1% increase in rainfall scenario is predicted to increase 1% AEP flood levels by at least 0.1 metres along the major watercourses with localised increases approaching 0.2 metres in some areas. The 18.6% increase in rainfall scenario is predicted to commonly increase existing 1% AEP flood levels by 0.2 metres.

Accordingly, the outcomes of the climate change simulations show that increases in rainfall associated with climate change have the potential to increase the severity of flooding across the catchment.

9 FLOOD PLANNING INFORMATION

9.1 Overview

Appropriate land use planning is one of the most effective measures available to manage the flood risk (particularly to control future risk but also to reduce existing flood risks as redevelopment occurs). A full review of land use planning including appropriate zoning, policies and planning/building controls is typically undertaken as part of the floodplain risk management study.

Nevertheless, *'Australian Disaster Resilience Handbook 7 Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia'* (ADR Handbook 7) (AIDR, 2017) recommends using the best available information to manage the flood risk at all times. Therefore, if a flood study is available that contains relevant information (such as this one), there is no need to wait for the floodplain risk management study before this flood information is used to inform land-use planning. Accordingly, the following chapter outlines the process that was employed to develop flood planning category constraint mapping to assist in informing future land-use planning decisions.

In addition, the results of the flood study can be used to develop a refined understanding of the flood planning area (i.e., the area within which flood-related development controls apply). Improved definition of the flood planning area and the associated identification of flood control lots can help to ensure that areas with a higher flood exposure/risk are identified and, should new development or re-development occur, will help ensure appropriate controls are implemented such that the flood exposure/risk is not increased.

9.2 1% AEP Flood Selection

Flood Planning Levels (FPLs) are an important tool in the management of flood risk and are discussed in more detail in Section 9.3.1. Flood planning levels are derived by combining a "planning flood" with a "freeboard". Penrith City Council has defined the 100 year ARI (1% AEP) flood as the planning flood through its Local Environmental Plan 2010. Accordingly, the selection of a reliable 1% AEP flood level across all sections of the catchment is a critical component of defining a suitable FPL and ensuring the flood risk is appropriately managed.

As discussed in Section 6.3.1, the design 1% AEP local catchment flood was defined as part of the study based upon a peak 5% AEP water level within the Nepean River. This is considered to provide a reasonable description of 1% AEP flood levels associated with local catchment runoff. However, as discussed in Section 8.2.6, the downstream sections of the catchment also have the potential to be inundated during large Nepean River floods. Therefore, from a flood planning perspective, it was considered important to not only define 1% AEP flood levels from local catchment flooding but also include 1% AEP flood levels as a result of Nepean River flooding.

As outlined in Section 6.3.1, it was not considered appropriate to assume a 1% AEP flood was occurring in the Nepean River at the same time as a 1% AEP local catchment flood due to the significant differing characteristics of the contributing catchments. Past studies (e.g., *'Peach Tree Creek Flood Study'* as well as the *'Panthers Precinct Master Plan – Flood Assessment Report'*) have overcome this by creating a design flood "envelope" comprising the following events:

- 1% AEP local catchment flood with 5% AEP Nepean River tailwater (reflecting 1% AEP local catchment flooding); and,
- 5% AEP local catchment flood with 1% AEP Nepean River tailwater (reflecting 1% AEP Nepean River flooding).

It was considered appropriate to retain this approach for defining 1% AEP flood levels for the current study. Accordingly, revised 1% AEP depth, level and velocity maps were prepared based upon the combined local catchment and Nepean River results and are presented in **Figures 61**, **62** and **63** respectively.

Peak flood levels, depths and velocities were also extracted at twenty-four discrete locations across the study area and are provided in **Table 28**.

Table 28 Peak Design Flood Levels, Depths and Velocities at Various Locations across the Catchment for the 1% AEP Flood

Location (refer to Plate 12)	**Peak 1% AEP Level (mAHD)**	**Peak 1% AEP Depth (m)**	**Peak 1% AEP Velocity (m/s)**
1	45.17	0.72	0.23
2	38.03	0.28	0.28
3	32.84	0.53	0.66
4	49.84	0.74	0.52
5	44.73	1.11	0.62
6	39.46	1.32	0.29
7	34.88	0.85	0.67
8	28.85	1.86	1.05
9	49.90	0.69	0.20
10	50.22	1.20	0.39
11	41.18	1.59	0.79
12	37.63	1.33	0.16
13	32.65	1.27	0.93
14	35.43	2.21	2.55
15	34.73	1.77	2.93
16	36.44	0.45	1.16
17	29.73	1.29	2.69
18	27.18	0.89	0.66
19	26.72	1.43	0.68
20	26.90	2.71	1.49
21	25.66	5.96	1.15
22	26.05	2.04	1.25
23	25.64	5.70	1.63
24	25.55	1.76	0.09

Flood hazard and hydraulic category mapping was also prepared for the combined 1% AEP results set and are provided in **Figures 64** and **65** respectively.

9.3 Flood Planning Area

9.3.1 Flood Planning Level

As discussed, Flood Planning Levels (FPLs) are an important tool in the management of flood risk and are derived by adding a freeboard to the "planning" flood (i.e., 1% AEP flood, as discussed in Section 9.2). The FPLs can then be combined with topographic information to establish the Flood Planning Area (FPA). The FPL and FPA can then be used to assist in managing the existing and future flood risk by:

- Setting design levels for mitigation works (e.g., levees); and
- Identifying land where flood-related development controls apply to ensure that new development is undertaken in such a way as to minimise the potential for flood impacts on people and property.

The "freeboard" is a factor of safety that is used to account for uncertainties in deriving the planning flood levels. Penrith City Council currently specify a 0.5 metre freeboard in its Local Environmental Plan 2010. As part of the current study, Council wished to confirm the suitability of adopting a 0.5 metre freeboard across the Peach Tree and Lower Surveyors Creeks catchment.

As discussed, freeboard is used to account for uncertainties when deriving the 1% AEP flood levels. More specifically, freeboard is used to account for the following uncertainties:

- Model parameter uncertainty; and,
- "Local" factors that can't be explicitly represented in the computer modelling (e.g., wave action or small flow paths less than the model grid size).

A discussion on each of these components is presented below. A discussion on the most appropriate planning flood level is also provided.

Model Parameter Uncertainty

The potential impacts of model parameter uncertainty can be quantified by reviewing the results of the sensitivity simulations presented in Section 8.

The information presented in Section 8 shows that across the majority of the study area, the 99th percentile change in 1% AEP flood levels from each of the sensitivity simulations does not exceed 0.3 metres. That is, we are 99% confident that the "true" 1% AEP flood level is contained within ±0.3 metres of the "base" 1% AEP simulations documented in Section 6 across the majority of the catchment.

However, some localised areas are subject to greater uncertainty. This includes the Surveyors Creek crossing of Mulgoa Road where blockage has the potential to increase 1% AEP flood levels by more than 0.5 metres.

Local Factors

Unfortunately, the uncertainty associated with the remaining factors (i.e., wave action and local factors that cannot be represented in the model) cannot be as readily quantified.

However, across the catchment the wind fetch length is small, water depths are generally shallow and any boats or cars would typically be operating at low speeds. As shown in **Plate 26**, under these circumstances, the waves generated by cars are unlikely to exceed 0.15 metres and dissipate significantly in height by the time the wave reaches the edges of the road. Therefore, a wave action allowance of 0.15 metres is considerd to be sufficient.

Plate 26 Example of cars driving through flood waters and generating waves

Overall, it is considered that a freeboard that accounts for the following uncertainties would be approriate across the majoirty of the study area:

- Modelling and climate change uncertainty = 0.30 metres; and
- *'Other'* uncertainty (e.g., wave action) = 0.15 metres

Accordingly, a minimum freeboard of 0.30 metres + 0.15 metres = 0.45 metres is considered to be reasonable with the 1% AEP flood results documented in Section 9.2. Therefore, the adoption of a 0.5 metre freeboard appears to be suitable for the majority of the study area.

Consideration could be given to implementing a higher freeboard in areas of higher uncertainty (e.g., upstream of the Mulgoa Road crossing of Surveyors Creek). Conversely, consideration to a lower freeboard could be given in areas of shallow flow (e.g., inundation depths less than 0.3 metres). These recommendations will be considered for further detailed investigation through the subsequent floodplain risk management study process.

9.3.2 Flood Planning Area

The 0.5 metre freeboard was added to the 1% AEP water level results documented in Section 9.2 to produce a flood planning level grid. The flood planning level grid was combined with the digital elevation model to produce a flood planning area based upon the following approach:

- In areas where the 1% AEP inundation depths were greater than or equal to 0.3 metres, the flood planning level grid was projected laterally until the flood planning level encountered higher terrain;
- The flood planning level grid was also projected laterally until the flood planning level encountered higher terrain across all areas within the 1% AEP that were traversed by a stormwater pipe. This was completed to comply with the definition of "major drainage" within the *'Floodplain Development Manual'* (NSW Government, 2005) and is intended to account for the uncertainty associated with 1% AEP water levels in the vicinity of stormwater pits where blockage can significantly impact on flood behaviour; and,
- In areas where the 1% AEP inundation depths were less than 0.3 metres, the flood planning level grid was not projected laterally. This is intended to reflect the increased confidence in model results across areas of shallow inundation.

The resulting flood planning area is shown in **Figure 66**.

9.3.3 Flood Control Lots

A preliminary flood control lots layer was prepared by selecting all cadastral lots that were intersected by the flood planning area. That is, if the flood planning area extended across any part of a cadastral parcel it was selected as a flood control lot. This layer was subsequently provided to Penrith City Council in electronic format.

9.4 Flood Planning Constraint Categories

Flood planning category constraint mapping was prepared based on guidance provided in the *'Australian Disaster Resilience Guideline 7-5: Flood Information to Support Land-use Planning'* (AIDR 2017). This guideline delineates flood liable land into one of four major "constraint" categories (with several subcategories) based upon key flooding considerations such as flood hazard, flood function and emergency response. The resulting categories can serve to inform land use planning activities. The guideline notes that the categorisation is intended to support community/precinct scale decisions where flow paths and flood extents can be readily defined and was not developed to support change of land use or development at the lot/site scale.

The flood planning constraint categories (FPCC) are summarised in **Table 29**. **Table 29** also summarises how the categories are defined along with the associated planning implication/considerations. In general, a FPCC categorisation of "1" implies a more flood constrained section of land relative to FPCC category "2", and so on.

The categories use a "Defined Flood Event" (DFE), which is analogous to the "planning flood" (i.e., 1% AEP event). It also requires consideration of flood impacts in events rarer than the DFE. The 0.5% AEP event was selected for this purpose. In both cases, the local catchment floods results were used.

The information contained in **Table 29** was used with the flood modelling outputs (most notably the flood hazard, hydraulic category and emergency response mapping) to prepare the FPCC map shown in **Figure 67**. Also included on **Figure 67** are the current land use zones to gain an appreciation of how the current zoning aligns with the FPCC. The proportion of each land zone that falls within each FPCC was also extracted and is presented in **Table 30**.

Table 29 Flood Planning Constraint Categories (AIDR, 2017)

FPCC	Sub-Category	Constraint	Implications	Consideration
1	A	Flow conveyance and storage areas in the DFE	Development or changes to topography within flow conveyance areas and flood storages areas affect flood behaviour, which will alter flow depth or velocity in other areas of the floodplain. Changes can negatively affect the existing community and other property	The majority of developments and uses have adverse impacts on flood behaviour. Consider limiting uses and development to those compatible with maintaining flood function
	B	H6 hazard in the DFE	Hazardous conditions considered unsafe for vehicles and people. All building types are considered vulnerable to structural failure	The majority of developments and uses are vulnerable to failure in this flood hazard category. Consider limiting developments and uses to those that are compatible with flood hazard H6
2	A	Flow conveyance area in events larger than the DFE	Flow conveyance areas may develop during an event larger than the DFE. People and buildings in these areas may be affected by flowing and dangerous floodwaters	Consider compatibility of developments and users with rare flood flows in this area
	B	H5 hazard in the DFE	Hazardous conditions are considered unsafe for vehicles and people, and all buildings are vulnerable to structural damage	Many uses and developments will be vulnerable to flood hazard. Consider limiting new uses to those compatible with flood hazard H5. Consider treatments such as filling (where this will not affect flood behaviour) to reduce the hazard to a level that allows standard development conditions to be applied. Alternatively, consider a requirement for special development conditions
	C	Isolated and submerged areas (low flood island or low trapped perimeter in 1%AEP event)	Area becomes isolated by floodwater or impassable terrain, with loss of evacuation route to the community evacuation location. The area will become fully submerged with no flood-free land in an extreme event, with ramifications for those who have not evacuated and are unable to be rescued	Consequences of isolation and inundation can be severe. Consider the consequences of: • evacuation difficulty or inundation of the area on the development and its users, which may include limitations on land use, or on land use that has occupants who are more vulnerable to disruption and loss • the development on emergency management planning for the existing community, including the need for additional treatments • the development on community flood recovery • disruption or loss of the development on the users and wider community
	D	Isolated but not submerged areas (high flood island or high trapped perimeter in 1%AEP event)	Area becomes isolated by floodwater or impassable terrain, with loss of an evacuation route to a community evacuation location. The area has some land elevated above the extreme flood level. Those not evacuated may be isolated with limited or no services, and will need rescue or resupply until floods recede and roads are passable	Some developments and their users may be vulnerable to disruption or loss. Consider: • the consequences of disruption or loss of the development on the users and the wider community • limiting land use, or land use that has occupants who are more vulnerable to disruption and loss • additional emergency management treatment requirements • issues associated with the level of support required during a flood, particularly for long-duration flood events
	E	H6 hazard in events rarer than the DFE	Hazardous conditions may develop in an event rarer than the DFE, which may have implications for the development and its occupants	Consider the need for additional development conditions to reduce the effect of flooding on the development and its occupants

FPCC	Sub-Category	Constraint	Implications	Consideration
3	-	Outside FPCC 2 but generally below the DFE plus freeboard	Hazardous conditions may exist creating issues for vehicles and people. Structural damage to buildings that meet building standards unlikely because of flooding	Standard land-use and development controls aimed at reducing damage and the exposure of the development to flooding in the DFE are likely to be suitable. Consider the need for additional conditions for emergency response facilities, key community infrastructure and vulnerable users
4	-	Outside of FPCC 3 but within the PMF extent	Emergency response may rely on key community facilities such as emergency hospitals, emergency management headquarters and evacuation centres operating during an event. Recovery may rely on key utility services being able to be readily re-established after an event	Consider the need for conditions for emergency response facilities, key community infrastructure and land uses with vulnerable users

Table 30 Land use zones falling within each FPCC

Zone	FPCC									
	1		2					3	4	Not Impacted
	A	B	A	B	C	D	E			
B1	1%	0%	0%	0%	0%	0%	0%	1%	36%	63%
B2	12%	0%	0%	0%	0%	0%	0%	3%	54%	31%
B3	0%	0%	0%	0%	0%	0%	0%	0%	17%	83%
B4	0%	0%	0%	0%	0%	0%	0%	2%	68%	31%
B5	4%	0%	0%	0%	0%	34%	0%	1%	59%	1%
B6	2%	0%	0%	0%	0%	21%	0%	0%	1%	75%
IN1	5%	0%	0%	0%	1%	31%	0%	1%	59%	2%
R2	3%	0%	0%	0%	1%	4%	0%	1%	23%	68%
R3	4%	0%	0%	0%	3%	6%	0%	1%	47%	39%
R4	18%	0%	2%	0%	0%	7%	0%	6%	65%	1%
R5	12%	0%	0%	0%	0%	0%	0%	2%	71%	16%
RE1	28%	0%	1%	0%	0%	30%	0%	1%	16%	24%
RE2	45%	0%	1%	0%	0%	30%	0%	1%	22%	1%

The FPCC categories presented in **Figure 67** show that current land use zones are broadly compatible with the level of flood exposure. More specifically, the more highly constrained land (i.e., FPCC 1) typically coincides with areas of open space (i.e., zones RE1 and RE2), which is considered to be a compatible land use. The only notable exceptions are the major overland flow paths that extends from near The Northern Road towards Jamison Park (refer **Figure 67.3**). FCC 1A extends across multiple residential properties in this area. This categorisation indicates that any flow obstructions have the potential to impact on flood behaviour. Therefore, care will need to be exercised if any new/redevelopment occurs in this area to ensure existing flow paths and storage areas are retained. For example, any filling or buildings that would serve to obstruct flow or remove storage volume should be avoided and the potential to install “open” fencing could be explored.

FPCC 4 is the most dominant category impacting on the study area. In general, this should not present significant issues for most development types. However, care will need to be exercised for particularly sensitive or important land uses (e.g., hospitals, aged care facilities, preschools).

10 Hot Spots Investigation

10.1 General

As part of the study a detailed analysis of flood behaviour was completed across a number of high flood hazard “hot spots”. The outcomes of this detailed analysis are summarised below and includes the following areas:

- South of Jamison Road (between Fragar Road and Racecourse Road), South Penrith;
- South of Smith Street (between Mazepa and Aston Avenues), South Penrith;
- Huron Place, Peter Ct and Glenbrook St, Jamisontown;
- South of Jamison Road (between York Road and Mulgoa Road), South Penrith;
- South of Jamison Road (between Mulgoa Road and Anakai Drive), South Penrith;

A list of potential flood and drainage mitigation measures are presented for each of the flooding “hot spots”. The goal of the assessment was to provide a list of potential measures that could be implemented to reduce the existing flood risk across these high hazard “hot spots”. Those mitigation measures could then be shortlisted for a more comprehensive analysis as part of the subsequent floodplain risk management study.

10.2 Flooding “Hot Spots” and Potential Mitigation Measures

10.2.1 South of Jamison Road (between Fragar Road and Racecourse Road), South Penrith

As shown in **Figure 20.3**, a major overland path extends through a number of residential properties south of Jamison Road (between Fragar Road and Racecourse Road) at South Penrith. The flow path also extends through a part section of the Penrith South Primary School before discharging in a westerly direction along Kennedy Drive and across Racecourse Road. At the peak of the 1% AEP flood, floodwater depths are predicted to exceed 0.5 metre at multiple locations.

Figure 39.3 also shows that peak flow velocities are predicted to exceed 2 m/s at a number of locations (most commonly where water is “squeezed” between buildings). Velocities of over 1 m/s are also predicted down Kennedy Drive. Fortunately, the most significant depths and highest velocities tend to be concentrated across areas of open space. However, significant depths and velocities are also predicted in areas where overland flow is concentrated between buildings. The high velocities along Kennedy Drive would also likely mobilise any vehicles parked along this street during the 1% AEP flood.

As shown in **Figure 43.3,** the stormwater system across the upper sections of this flow path have a capacity of no greater than the 50% AEP event. Accordingly, during significant rainfall events, a sufficient amount of flow is predicted to be directed overland through properties.

The lack of stormwater capacity is further illustrated in **Plate 27** and **Plate 28**, which shows water depths 30 minutes after the initial onset of rainfall during the 1% AEP, 45-minute simulation. It shows significant ponding depths at the “sag” points in Penrose Circuit and Taloma Street (i.e., the downstream pipe system does not have sufficient capacity even with notable upstream water depths to “drive” water through the pipe system). Notable depths of inundation are also evident in the villa complex at 115 Evan Street as well as the adjoining residential properties at 14-16 Victory Street. Some shallow inundation is also evident along Kennedy Drive, particularly near its intersection with Racecourse Road.

Potential Mitigation Options

As discussed, the inundation problems across this area are primarily associated with the limited capacity of the existing drainage system. Two primary options are available to rectify this limitation:

- Increase the capacity of the existing stormwater system (e.g., lay additional stormwater pipes and/or upgrade existing pipes and pits so that a greater proportion of the flow can be conveyed below ground); and
- Reduce the amount of water travelling through the stormwater system and overland (e.g., construct detention basins to temporarily store excess runoff).

The stormwater capacity map was reviewed to determine locations where stormwater upgrades would be most beneficial. These are shown on **Figure 68.1**.

Areas considered to be suitable for consideration as detention basins are shown on **Figure 68.1** and include:

- Butler Park;
- 75A Penrose Cres
- 24 Taloma Street
- South-western corner of Penrith South Public School.

It should be noted that it is rarely economically feasible to provide a stormwater system that can carry flow during all events up to and including the PMF. Therefore, provision will still need to be made for areas to convey overland flows or the stormwater upgrades will need to be completed in conjunction with the detention basins described above to reduce overland flows to tolerable levels. It may also be possible to develop a formal drainage easement through the rear of the impacted properties and create a dedicated overland flow path to carry overland flows during larger events. Although a drainage easement does extend through this area (refer **Figure 5.3**), this does not appear to be formalised with a significant number of overland flow obstructions evident. The easement alignment also appears to extend across the footprint of several buildings. Formalisation of this drainage easement would likely include realigning some sections of the easement to avoid buildings, removing existing flow obstructions (e.g., fences) and design of a swale system (potentially in conjunction with a new or upgraded underlying pipe system, as discussed above) to more efficiently and safely carry overland flow. If acquisition of a drainage easement is not found to be feasible, the potential to install “open” fencing could also be investigated.

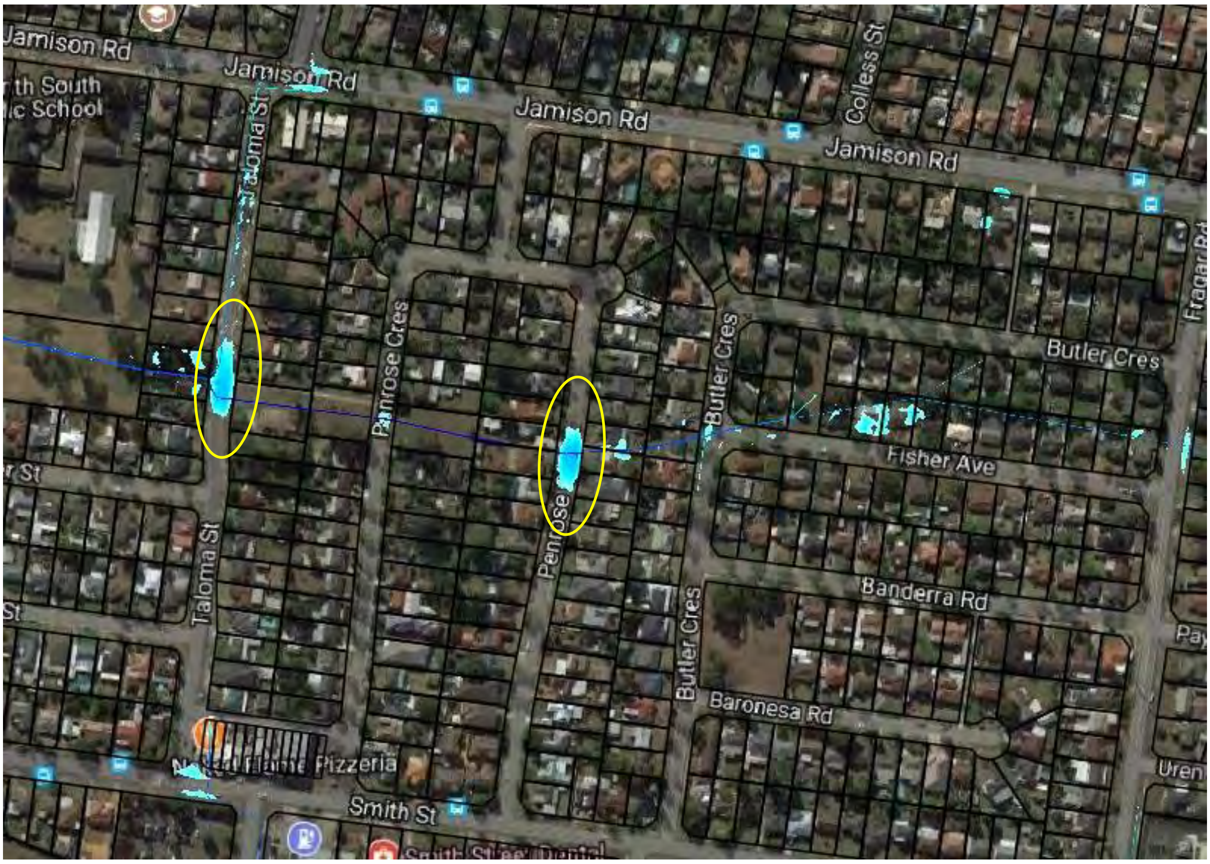


Plate 27 1% AEP Depths south of Jamison Road (upstream section) after 30mins of rainfall.

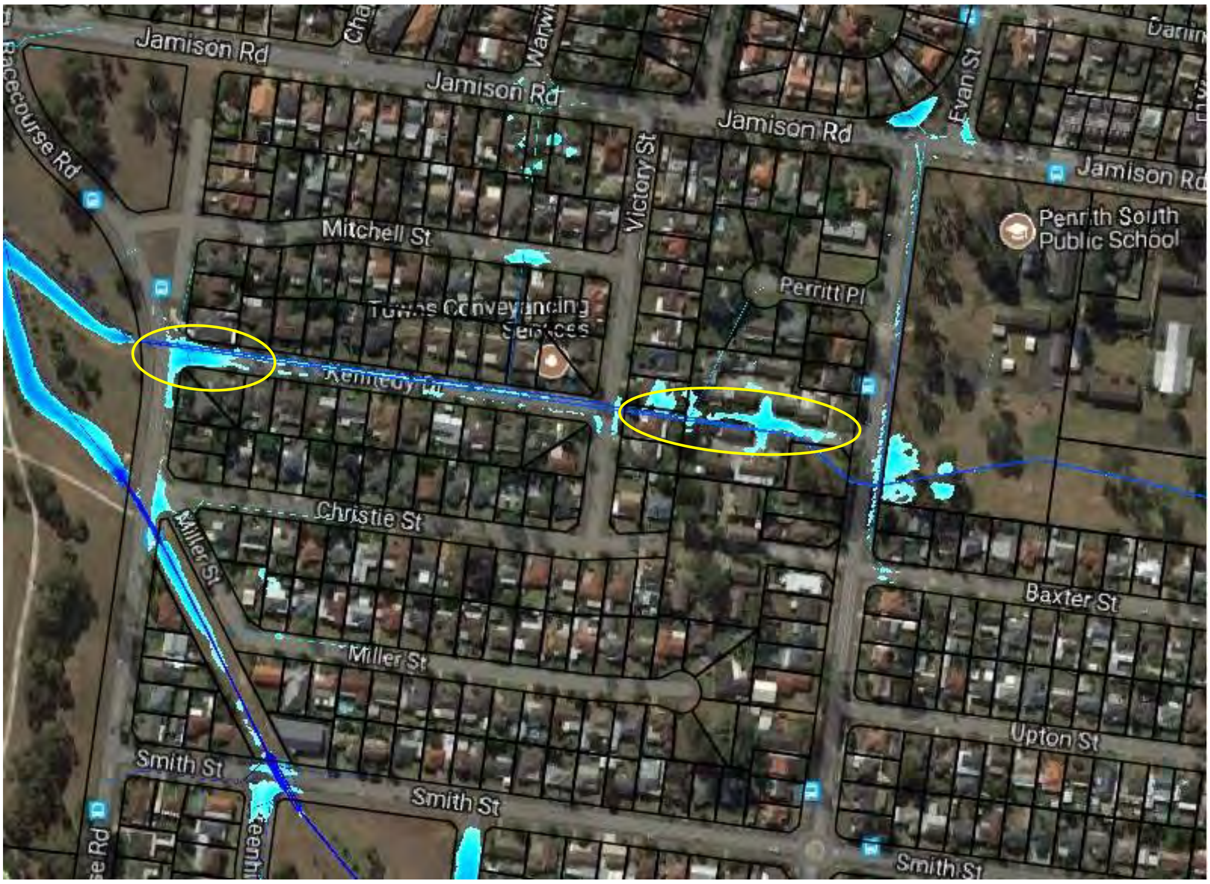


Plate 28 1% AEP Depths south of Jamison Road (downstream section) after 30mins of rainfall.

Opportunities to undertake regrading near the intersection of Kennedy Drive and Racecourse Road could also be explored in an effort to reduce the extent of "ponding" of water at this location and allow overland flows to more readily escape into the Racecourse channel.

10.2.2 South of Smith Street, South Penrith

As shown in **Figure 20.3**, a major overland path also extends through a number of residential properties immediately south of Smith Street (between Mazepa and Aston Avenues) at South Penrith. At the peak of the 1% AEP flood, floodwater depths are predicted to exceed 0.5 metre at some locations with peak depths of more than 0.3 metres being common. **Figure 39.3** also shows that peak flow velocities are predicted to exceed 1 m/s at a number of locations.

Figure 20.3 also shows a smaller, secondary flow path extending between Fragar Road and Wentworth Drive (north of Blue Gum Avenue). Peak 1% AEP floodwater depths along this flow path are predicted to exceed 0.4 metres in Fragar Road and exceed 0.3 metres across the rear yards of some residential properties. Flow velocities along this secondary flow path typically do not exceed 0.5 m/s.

Figure 43.3 shows that the stormwater system along both flow path alignments provides no greater than a 50% AEP capacity. Several pits are also predicted to surcharge and/or exceed a ponding depth of 0.2 metres during larger rainfall events (i.e., 5% AEP or rarer). Accordingly, during significant rainfall events, a sufficient amount of flow is predicted to be directed overland through properties.

The lack of stormwater capacity is further illustrated in **Plate 29**, which shows water depths 35 minutes after the initial onset of rainfall during the 1% AEP, 45-minute simulation. It shows significant ponding depths forming across the rear of properties located at 19-27 Treetops Avenue and 152 to 156 Smith Street. The ponding depths appear to be primarily associated with the lack of stormwater capacity in this area, but the flooding problem is also being exacerbated by the significant overland flow obstructions including fences, buildings and garages/sheds.

Plate 29 also shows water depths appearing along the secondary flow path at the low point in Fragar Road. The lack of any significant overland flow obstructions indicates that it is a lack of stormwater capacity that is the initial reason for the inundation problems along this flow path. Notwithstanding, as soon as water spills from Fragar Road, the downstream fences and buildings are likely contributing to the inundation problem.

Potential Mitigation Options

As discussed, the inundation problems across this area are primarily associated with the limited capacity of the existing drainage system. Two primary options are available to rectify this limitation:

- Increase the capacity of the existing stormwater system (e.g., lay additional stormwater pipes and/or upgrade existing pipes and pits so that a greater proportion of the flow can be conveyed below ground); and
- Reduce the amount of water travelling through the stormwater system and overland (e.g., construct detention basins to temporarily store excess runoff).

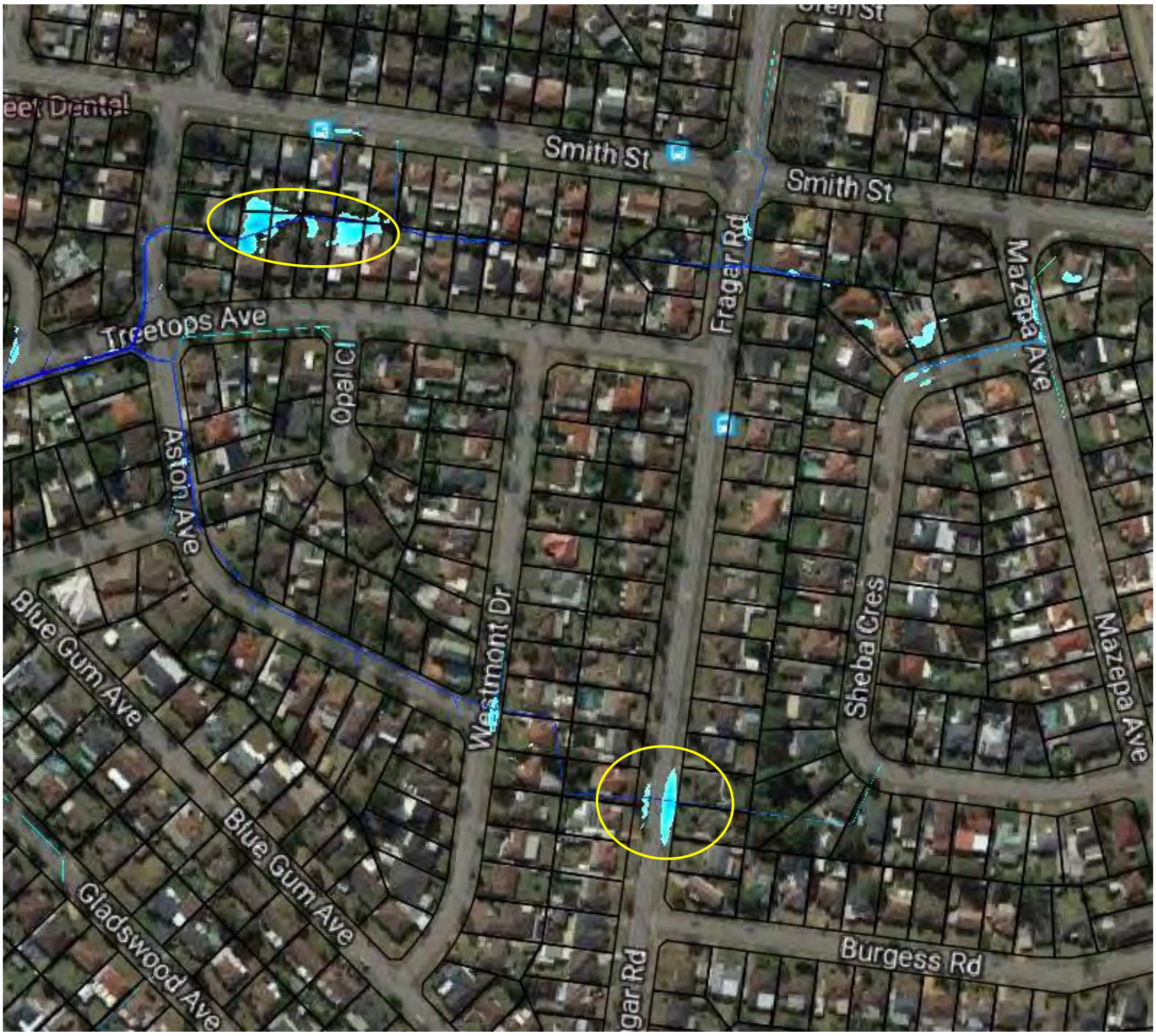


Plate 29 1% AEP Depths south of Smith Street after 35mins of rainfall.

Unfortunately, the lack of open space makes it difficult to implement a cost-effective detention basin. More specifically, if a detention basin option was pursued, some existing residential properties would need to be purchased to accommodate the basin, which is likely to be prohibitively expensive. Alternatively, underground storage tanks could be explored for installation within the existing road reserve. The potential location of underground storage tanks is shown in **Figure 68.2**.

Figure 68.2 also shows the locations where stormwater upgrades could be explored. The pipe upgrades could also be extended downstream along Treetops Ave where the current 1% AEP depths and velocities would be sufficient to mobilise cars at some locations.

As for the Jamison Road overland flow path, opportunities to formalise an overland flow path along the alignment of existing drainage easements could also be explored in an effort to reduce the overland flow impediments. Similarly, opportunities to create a formalised overland flow easement / remove overland flow impediments could be investigated along the secondary/southern flowpath.

10.2.3 Huron Place, Peter Ct and Glenbrook St, Jamisontown

As shown in **Figure 20.2**, significant ponding of water is predicted along Huron Pl, Peter Ct and onto Glenbrook St, Jamisontown (i.e., immediately east of Mulgoa Road). At the peak of the 1% AEP flood, floodwater depths are predicted to approach 1 metre deep along the roadways and water is also predicted to extend into some adjoining properties (although depths across properties are approximately 0.3 metres).

Figure 39.2 shows that peak flow velocities across this area do not exceed 0.5 m/s. nevertheless, some areas are exposed to velocities that approach 1 m/s. Although the floodwaters are not particularly fast moving, the significant depths mean that some areas would be exposed to a H3 hazard category (dangerous for children and the elderly).

As shown in **Figure 43.2,** the stormwater system in the area has limited capacity (generally no greater than the 50% AEP). When this lack of stormwater capacity is coupled with the elevated embankment formed by Mulgoa Road, significant ponding is likely to occur during most significant rainfall events.

Potential Mitigation Options

The major cause of flooding in this area is a result of the lack of stormwater capacity. Therefore, upgrades to the stormwater pit and pipe system should be explored. The location of existing stormwater pits and pipe that would benefit from an upgrade are shown in **Figure 68.3**.

Opportunities to reduce the amount of flow draining to the area could also be explored via detention areas. An existing park located near the corner of Glenbrook Street and Warragamba Crescent could provide an opportunity to create a flood detention basin (refer **Figure 68.3**). This would likely afford benefits to properties adjoining Peter Ct and Glenbrook St but is unlikely to afford benefits to Huron Pl. it is noted that the park contains several large trees and care will need to be exercised in any basin design to help ensure this existing vegetation is not adversely impacted.

Potential for localised regrading of Mulgoa Road could also be explored. This could be potentially explored as part of future upgrade work for Mulgoa Road. This would typically involve lowering the elevation of Mulgoa Road to reduce the magnitude of ponding in this area. The specific locations where roadway lowering/regrading could be explored is shown in **Figure 68.3**. However, care will need to be exercised to ensure properties to the west of Mulgoa Road are not adversely impacted as a result of additional flow passing across Mulgoa Road.

10.2.4 South of Jamison Road (between York Road and Mulgoa Road), South Penrith

As shown in **Figure 20.3**, multiple overland flow paths and areas of ponding are visible through a medium density residential area located south of Jamison Road (between York Road and Mulgoa Road) at South Penrith. The primary flow path enters the area from York Road (immediately west of Jamison Park) and moves in a west/north-westerly direction through multiple properties towards the Racecourse channel adjoining Jamison Road.

Notable inundation depths are also evident immediately south-east of the Jamison Road and Mulgoa Road intersection as well as areas adjoining Preston Street. At the peak of the 1% AEP flood, floodwater depths in part-sections of all flow paths/inundation areas are predicted to exceed 0.5 metres.

Underground carparking is also provided below several unit blocks within the area. It is typical for the driveway entry to these underground carparks to slope down away from the roadway (see images in **Appendix R**). During large flood events, water can spill from the roadway and run down the driveways and inundate the garage areas (refer **Plate 30**). However, it should be noted that the hydraulic model developed for the current study does not include a representation of the private draining system contained within these properties. Therefore, the extent of inundation across some areas may be exaggerated by the model (although the private drainage system is typically designed to carry more frequent rainfall events only, so some inundation would still be expected during significant rainfall events).

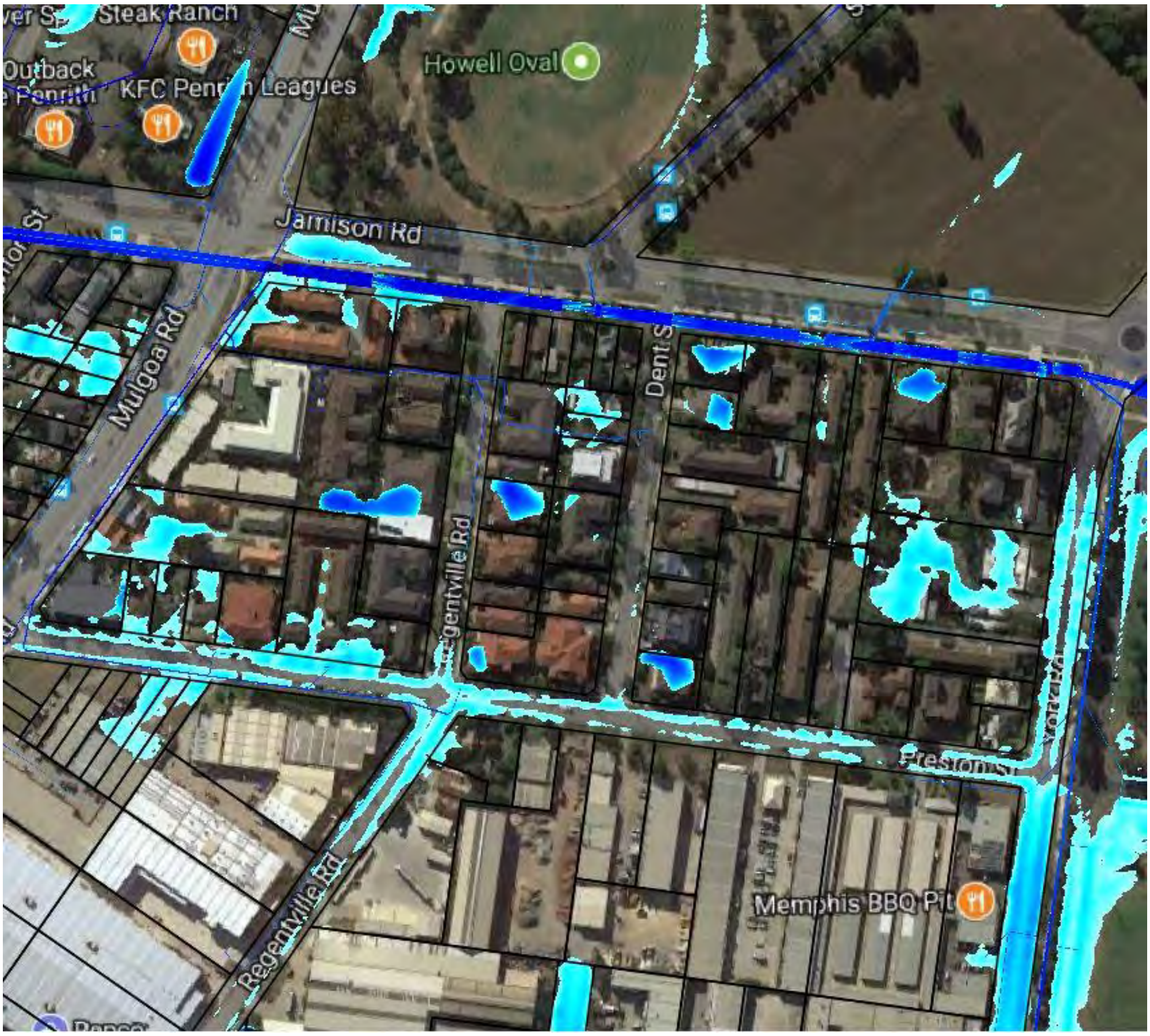


Plate 30 1% AEP Depths south of Jamison Road after 100mins of rainfall.

Figure 39.3 shows that peak flow velocities are generally predicted to remain below 0.5 m/s throughout the whole area. The relatively low flow velocities are associated with the flat terrain and significant flow impediments (e.g., buildings, retaining walls) in the area.

As shown in **Figure 43.3,** there is a relatively limited amount of stormwater infrastructure represented in the hydraulic model within the area of interest. Those stormwater pipes that are present typically have a capacity of no greater than the 50% AEP event. Accordingly, during significant rainfall events, a significant amount of overland flow and ponding could be expected. However, it is noted that the pipe system along York Road appears to have additional capacity that is not being utilised. This may indicate the need for additional stormwater pits to direct more flow into the pipe system.

Potential Mitigation Options

As discussed, the inundation problems across this area are associated with the limited capacity of the existing stormwater system, the flat terrain, the many flow obstructions and water spilling from the adjoining Jamison Park.

Options available to assist in reducing the inundation problem include providing additional stormwater pits and pipes along York Road. This would aim to take advtange of the spare capacity within the existing pipe system, thereby reducing the amount of flow spilling across York Road and into the downstream properties. Suggested locations for additional stormwater pits and pipes are shown in **Figure 68.4**.

Stormwater upgrades could also be investigated in Preston St as well as Dent St and Regentville Rd (e.g., lay additional stormwater pipes and/or upgrade existing pipes and pits so that a greater proportion of the flow can be conveyed below ground). The location of where pipe and pit upgrades may be beneficial is also shown in **Figure 68.4**.

The potential to provide a flood detention area upstream of York Road could also be investigated. This would aim to attenuate some flow from the upstream catchment thereby reducing the amount of flow spilling across York Road as well as reduce the building up of water near the Mulgoa/Jamison Road intersection. The most obvious location for this storage area is within Jamison Park. A suggested location is shown in **Figure 68.4**. The storage could take the form of an above ground storage. This would likely be the more affordable option and would involve lowering the elevation of the current sports field. However, if the functionality of the current sports fields is to be retained (i.e., the current elevation is to be largely maintained), an opportunity to install underground storage tanks could also be investigated.

The outcomes of the blockage sensitivity analysis confirmed that blockage of the culvert draining the Racecourse Channel beneath Mulgoa Road will increase the severity of flooding in the vicinity of the Mulgoa/Jamison Road intersection. Therefore, reducing or removing blockage generating debris from the upstream catchment would also be beneficial in ensuring that the existing culvert infrastructure is being fully utilised. It is suggested a debris control device could be installed within Jamison Park (refer **Figure 68.4**). This location was selected as it is located away from existing residential properties which will help ensure any blockage of the debris control device itself should not adversely impact on existing properties.

10.2.5 South of Jamison Road (Mulgoa Road to Anakai Drive), South Penrith

As shown in **Figure 20.2**, floodwaters are predicted to overtop the banks of Peach Tree Creek immediately downstream of its junction with Surveyors Creek and inundate Anakai Drive as well as adjoining residential properties. At the peak of the 1% AEP flood, floodwater depths are predicted to reach 0.9 metre at some locations within Anakai Drive. **Figure 39.2** also shows that peak flow velocities are predicted to exceed 1 m/s at some locations along Anakai Drive.

Figure 20.2 also shows significant inundation depths centred around McNauhgton Street (located west of Mulgoa Road). Peak 1% AEP floodwater depths at this location are predicted to exceed 0.7 metres. Flow velocities are typically less than 0.3 m/s indicating water is largely “ponded” at this location.

Plate 31 shows 1% AEP floodwater depths 110 minutes after the onset of rainfall (based upon the 120 minute storm duration). It shows significant ponding depths forming in McNauhgton Street. Notable inundation depths are also commencing across properties to the east of McNauhgton Street (i.e., adjoining Mulgoa Road). The storm capacity information provided in **Figure 43.2** confirms that the stormwater system draining Mulgoa Road and McNauhgton Street has a 50% AEP capacity indicating the inundation at these locations is primarily a result of the lack of stormwater capacity. The inundation depths are also exaggerated by the surrounding terrain which forms a large topographic “bowl” which prevents flows in excess of the capacity of the stormwater system to “escape” from the area.

Plate 31 also shows floodwater ponding near the intersection of Anakai Drive and Yanco Ave. **Figure 43.2** confirms that inundation at this location is first occurring as a result of lack of stormwater capacity. However, elevated water levels within the receiving Peach Tree Creek channel also appears to be reducing the efficiency of the drainage system at this location. **Plate 32** also shows that in addition to local stormwater inundation, Surveyors Creek is predicted to overtop its banks near its confluence with Peach Tree Creek and flow north along Anakai Road. Therefore, although inundation first occurs as a result of failure of the local drainage system, it is the floodwaters from Surveyors / Peach Tree Creeks that is the main contributor to the significant inundation depths and velocities along Anakai Drive.

Potential Mitigation Options

The inundation problems in the vicinity of McNauhgton Street and Mulgoa Road are mainly associated with the limited capacity of the existing drainage system. Unfortunately, the lack of any open space makes opportunities for an above ground flood detention basin difficult. However, underground storages could be explored within the road reserve of McNauhgton Street and Mulgoa Road to provide additional storage volume and reduce ponding depths. The potential locations for underground storage tanks are shown in **Figure 68.5**.

Stormwater pipe and pit upgrades could also be investigated along McNauhgton Street and Mulgoa Road to provide additional below ground conveyance capacity which may also assist in reducing ponding depths. The location of potential pipe and pit upgrades is shown in **Figure 68.5**.

Plate 31 1% AEP Depths near Anakai Drive and McNauhgton Street after 110mins of rainfall.

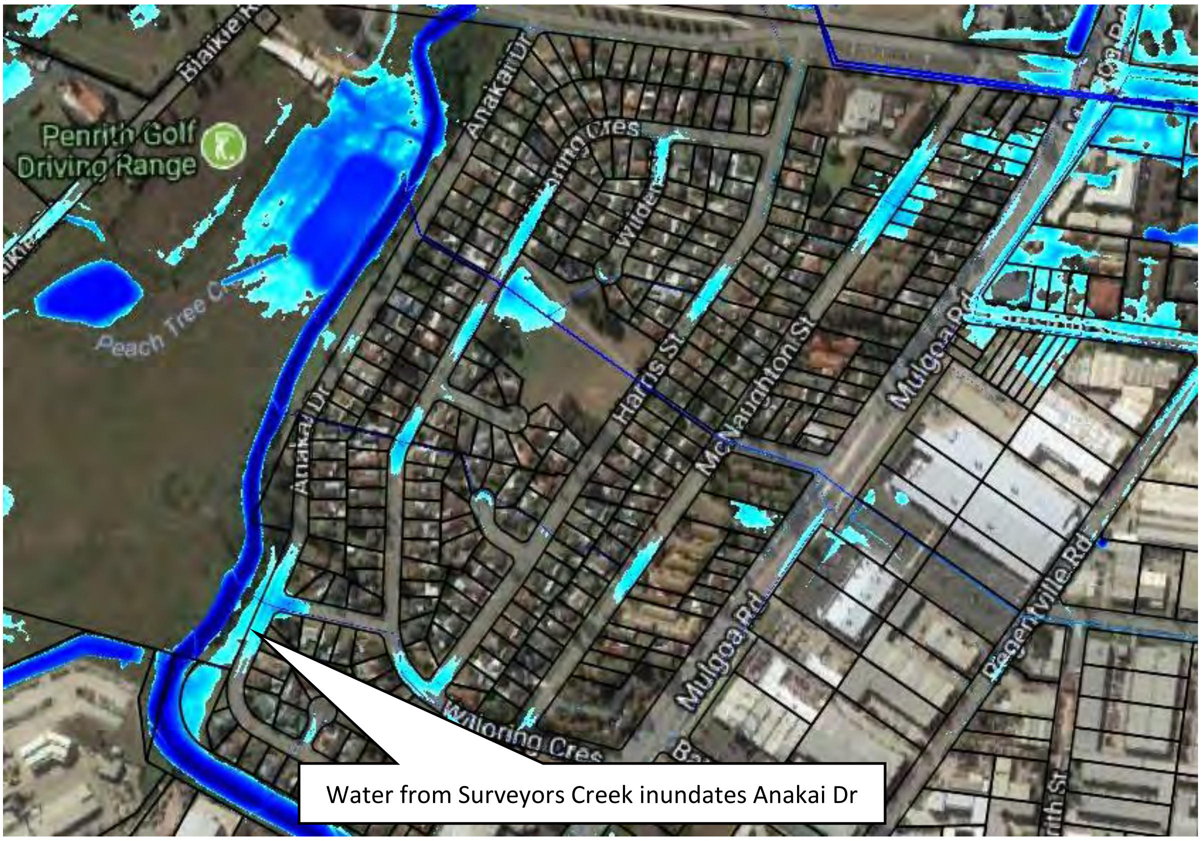


Plate 32 1% AEP Depths near Anakai Drive and McNauhgton Street after 115mins of rainfall.

Stormwater upgrades could also be explored in the vicinity of Anakai drive and Yanco Ave. However, as discussed above, flooding across this area is dominated by mainstream flooding from Surveyors and Peach Tree Creeks. Accordingly, any stormwater upgrades are unlikely to provide a significant benefit once floodwater begins to overtop the creek banks as the elevated water levels in the creek would prevent the local stormwater system from draining under gravity. However, the potential to install a levee between Surveyors / Peach Tree Creeks and Anakai Drive could be investigated to reduce peak floodwater depths and velocities along Anakai Drive. Although the levee will reduce the potential for inundation from Surveyors and Peach Tree Creeks it will also reduce the potential for runoff from Anakai Drive to drain into the creek system (as the levee will introduce an overland flow obstruction). Therefore, the levee would potentially need to be accompanied by a potential upgrade of the local drainage system to ensure the area behind the levee can still drain into Peach Tree Creek. The potential alignment of the levee is shown in **Figure 68.5**.

If existing flood levels along Surveyors and Peach Tree Creek could be reduced, it might also assist in reducing the frequency and severity of inundation along Anakai Road. In this regard, opportunities to increase the conveyance capacity of the existing creek system could be explored. This could be potentially achieved by reducing the density of vegetation along the creek banks (e.g., removing any non-native species of vegetation) and/or widening the channel. If creek widening was pursued, this would need to ensure the local flora and fauna is not adversely impacted while still preserving flood function (i.e., concrete lining of the channel would be undesirable). The potential extent of channel conveyance improvement is shown in **Figure 68.5**.

Finally, the potential to create an offline storage area on the western side of Peach Tree Creek could be examined. The offline storage would be provided with an invert lower than the elevation of Anakai Road allowing water to spill into this storage area and lowering water levels within the main creek. The storage would only become active once the capacity of the creek system is exceeded and would be designed to drain under gravity once water levels in the creek system recede. The potential location of the storage area is shown in **Figure 68.5**.

11 Conclusion

This report documents the outcomes of investigations completed to quantify overland and mainstream flood behaviour across the Peach Tree and Lower Surveyors Creeks 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 study area was defined using a hydrologic computer model of the entire Peach Tree and Surveyors Creeks catchment as well as a two-dimensional hydraulic model. The hydrologic computer model was developed using the XP-RAFTS software and the hydraulic model was developed using the TUFLOW software.

The XP-RAFTS and TUFLOW models were validated using historic rainfall and reported descriptions of flood behaviour that were provided by the community for floods that occurred in 2006, 2012 and 2016. The model was also verified against alternate modelling techniques as well as results presented in other flood-related reports.

The calibrated and verified model was used to simulate the design 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods based upon the 2016 version of Australian Rainfall and Runoff (Geoscience Australia). The Probable Maximum Flood (PMF) was also simulated. The following conclusions can be drawn from the results of the investigation:

- Flooding across the catchment can occur as a result of major watercourses overtopping their banks, overland flooding when the capacity of the stormwater system is exceeded as well as inundation from elevated water levels in the Nepean River. Flooding east of Mulgoa Road is typically dominated by local catchment runoff while flooding west of Mulgoa Road and south of the railway line is typically dominated by Nepean River inundation. A 1% AEP flood from the Nepean River has the potential to generate flood levels that are more than 2 metres higher than 1% AEP local catchment flood levels in areas west of Mulgoa Road.
- Flooding can occur from a variety of different storm and rainfall durations. The worst-case flooding across the urban sections of the catchment typically occurs as a result of rainfall bursts that are 2 or less hours in duration. Across the downstream sections of the catchment, rainfall over a period of 6 hours will typically produce the worst flooding. Accordingly, flooding across the catchment may be produced by relatively short, high intensity thunderstorms through to longer rainfall events that may be generated by east coast lows.
- Inundation of over 900 properties is predicted at the peak of the 1% AEP flood (out of a total of 6,505 properties located within the study area). The most notable flooding "hot spots" include:
    - South of Jamison Road (between Fragar Road and Racecourse Road), South Penrith;
    - South of Jamison Road (between York Road and Mulgoa Road), South Penrith;

    - South of Jamison Road (between Mulgoa Road and Anakai Drive), South Penrith;
    - South of Smith Street (between Mazepa and Aston Avenues), South Penrith;
    - Huron Place, Peter Ct and Glenbrook St, Jamisontown;
- The catchment incorporates a number of bridges and culverts. The results of a blockage sensitivity analysis show that the severity of flooding upstream of these structures can be significantly increased due to blockage. This highlights the importance of routine maintenance on this infrastructure, particularly immediately after a flood.
- A number of roadways are predicted to be overtopped during the 1% AEP flood. This would typically render the roadways impassable for at least 1 hour.

A preliminary list of flood risk mitigation measures has been compiled as part of the study for each of the identified flooding “hot spots”. It is recommended that these measures be investigated further as part of the floodplain risk management study.

12 References

- Advision (2006). *Nepean River Flood Study: Exhibition Draft Report*". Prepared for Penrith City Council
- Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors) (2016) Australian Rainfall and Runoff: A Guide to Flood Estimation, © Commonwealth of Australia (Geoscience Australia).
- BMT WBM. (2016). TUFLOW User Manual: GIS Based 1D/2D Hydrodynamic Modelling. Build 2016-03-AA
- Cardno Lawson Treloar (2006). *Penrith Overland Flow Flood "Overview Study"*. Prepared for Penrith City Council
- Cardno (2015). *Penrith CBD Detailed Overland Flow Flood Study*. Prepared for Penrith City Council
- Engineers Australia. (1987). Australian Rainfall and Runoff - A Guide to Flood Estimation. Edited by D. Pilgrim.
- GHD (1994). *Surveyor Creek, Glenmore Park – Concept Design Report*. Prepared for Penrith City Council
- J. Wyndam Prince (1999). *Glenmore Park Stages 8DEFGHJ: Eastern Branch of Surveyors Creek West Arm – Trunk Drainage Design Report*. Prepared for Penrith City Council
- J. Wyndam Prince (2016). *Panthers Precinct Master Plan – Flood Assessment Report*. Prepared for Panthers Group
- J. Wyndam Prince (2016). *ESQ1818 Development Panthers Site – Flood Impact Assessment and Water Quality Management Report*. Prepared for Panthers Group
- Lawson Treloar (1994). *Emu Plains Flood Study*. Prepared for Penrith City Council
- Lyall & Macoun, (1991). *Surveyors Creek / Peach Tree Creek Hydraulic Study*. Prepared for Penrith City Council
- Lyall & Macoun, (1994). *Peach Tree Creek / Showground Creek Flood Study*. Prepared for Penrith City Council
- Lyall & Macoun, (1995). *Panthers Redevelopment Project – Flood Study: Buildings Extensions and Carpark at Club Building*. Prepared for panthers
- NSW Government. (2005). *Floodplain Development Manual: the Management of Flood Liable Land*. ISBN: 0 7347 5476 0
- NSW Department of Land and Water Conservation (1997). *Nepean River at Penrith Flood Study'*. Prepared for Penrith City Council
- NSW Public Works (1994). *Peach Tree Creek Flood Study*. Prepared for Penrith City Council
- Ryan, C (2013). *Using LiDAR Survey for Land Use Classification*. Paper presented at the 2013 Floodplain Management Authorities Conference, Tweed Heads.
- XP Software (2009). *XP-RAFTS: Urban & Rural Runoff Routing Application*. User Manual.