College, Orth and Werrington Creeks Catchment Overland Flow Flood Study

Final Report

Volume 1 of 2: Report Text & Appendices

June 2017

PENRITH CITY COUNCIL

Catchment Simulation Solutions

College, Orth and Werrington Creeks Catchment Overland Flow Flood Study

Final Report

Client	Client Representative
Penrith City Council	Dr Elias Ishak

REVISION / REVIEW HISTORY

Revision #	Description	Prepared by	Reviewed by
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Note: The Study has been adopted by Council in its Ordinary Meeting on 22nd May, 2017.

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EXECUTIVE SUMMARY

The College, Orth and Werrington Creeks Catchment Overland Flow Flood Study was prepared for Penrith City Council to identify main stream and overland flood behaviour across the catchment. The catchment covers an area of 12 km^2 and includes the suburbs of Werrington, Werrington County, Cambridge Park, Kingswood, Caddens and parts of Orchard Hills.

The flood study was overseen by Penrith City Council's Floodplain Risk Management Committee and technical and financial support was provided by the State Government under the Floodplain Management Program. The study will serve to guide future development across the catchment in a way that is cognisant of the flood risk. The study will also serve as the basis for identifying options that may be implemented to reduce the existing flood risk as part of the subsequent floodplain risk management study and plan.

A consultation program was implemented as part of the study to obtain information from the community regarding their past flooding experiences. The primary goals of the community consultation were to identify flooding "hot spots" and to collate historic flood information that could be used to assist in the validation of the computer flood model that was developed as part of the study. This was achieved through the development of a flood study website and the distribution of a community information sheet and questionnaire to approximately 8,000 households and businesses.

The community responses to the questionnaire indicate that flooding has been experienced on a number of occasions across the catchment. Most notably, a flood that occurred in February 2012 caused widespread traffic disruption, damage to private and public property (e.g., fences) as well as above floor inundation of several properties. Smaller floods were also reported in February 2010 and November 2011.

A computer flood model of the College, Orth & Werrington Creeks catchment was developed using the TUFLOW software as part of the study. The model was developed to include a representation of all features that will influence the movement of floodwaters across the catchment. This includes all stormwater pits and pipes, bridges, culverts, detention basins, buildings and fences. The topography across the catchment was defined in the model based upon a digital elevation model derived from aerial survey collected in 2011. Areas modified between 2011 and 2016 or that were under development at the time the study was prepared (e.g., Caddens and French Street subdivisions) were included based upon design plans or work as executed survey.

The computer model was validated against historic flood information that was extracted from the community consultation responses. This included twenty-five flood marks for the 2012 flood, seven flood marks for the 2010 flood and four flood marks for the 2011 flood. The outcomes of the validation process showed that the developed computer model was providing a reliable representation of flood behaviour across the catchment.

The validated flood model was then used to simulate a range of design floods across the catchment. This included the 1 in 2-year ARI, the 20%, 10% 5%, 2%, 1%, 0.5% and 0.2% AEP floods and the Probable Maximum Flood.

The results of the design flood simulations are presented in a series of maps that are contained in Volume 2 of the flood study. These maps contain information on floodwater depths, levels, velocities, hazard, hydraulic categories, as well as emergency response precinct classifications for each of the design floods.

The outcomes of the design flood simulations indicate that areas subject to the greatest inundation depths are typically aligned with natural waterways, roadways and detention basins. Nevertheless, approximately 25% of properties located within the catchment will be at least partly inundated at the peak of the 1% AEP flood. This is predicted to increase to over 60% during the PMF. Accordingly, major flooding has the potential to impact a significant number of properties within the catchment. Therefore, flood planning level and flood planning area mapping has also been prepared to assist Council in defining "flood control lots" (i.e., properties subject to a flood-related development control) which will assist Council in ensuring that future development and redevelopment is undertaken in a way that is compatible with the flood risk.

The study has identified several flooding "hot spots". This includes:

- Jamison Road to Bringelly Road, Kingswood;
- Somerset Street to Bringelly Road, Kingswood;
- Cox Avenue, Kingswood;
- Chapman Gardens to the main western railway line, Kingswood;
- Railway Street, Landers Street and Walker Street, Kingswood;

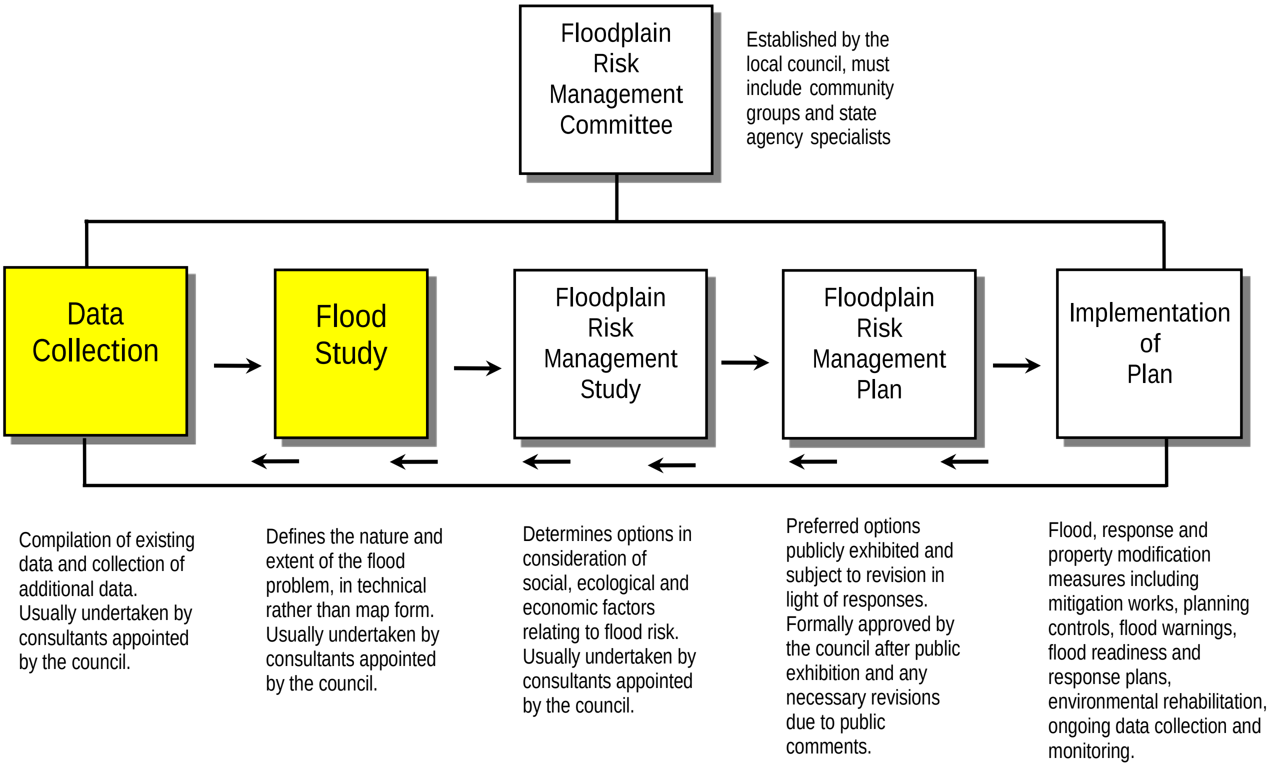
A list of preliminary potential mitigation measures has been prepared as part of the study for each of the flooding "hot spots" which may assist in reducing the existing flood risk. The effectiveness and feasibility of each of these options will be investigated in detail part of the subsequent floodplain risk management study for the catchment.

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 College, Orth and Werrington Creeks Catchment Overland Flow Flood Flood Study represents the first of the four 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.

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1 INTRODUCTION

1.1 Catchment Description

The College, Orth and Werrington Creeks catchment is located in the Penrith City Council Local Government Area (LGA) and occupies a total area of approximately 1,200 hectares. The extent of the catchment is shown in **Figure 1**. As shown in **Figure 1**, the catchment covers the suburbs of Werrington, Werrington County, Cambridge Park, Kingswood, Caddens and parts of Orchard Hills.

The urbanised sections of the catchment are typically drained by a stormwater system that conveys runoff into College and Orth Creeks which, in turn, drain into Werrington Creek. Werrington Creek generally drains in a north-easterly direction before discharging into South Creek downstream of Dunheved Road (refer **Figure 1**).

1.2 Purpose of Study

During periods of heavy rainfall across the College, Orth and Werrington Creeks catchment, there is potential for the capacity of the stormwater system to be exceeded. In such circumstances, the excess water travels overland, potentially leading to inundation of roadways and properties. There is also potential for water to overtop the banks of the creek network and inundate the adjoining floodplain.

Penrith City Council commissioned an overland flow flood 'Overview Study' (Cardno Lawson Treloar, 2006) to help gain a better understanding of the overland flood risk across the LGA. The Overview Study utilised modern 2-dimensional hydrodynamic modelling tools to assist Council in defining the location of major overland flow paths and identifying properties at risk of overland flooding. This information was used to define the variation in flood hazard and potential for flood damage and ultimately rank each subcatchment within the LGA based on the severity of the flood risk. This ranking is being used to prioritise each subcatchment within the LGA for detailed overland flood studies.

The Overview Study identified the College, Orth and Werrington Creeks catchment within the highest 10% of flood affected subcatchments across the LGA. Accordingly, Council resolved to undertake a detailed overland flow flood study for the catchment to improve their understanding of the overland flow risk and provide a suitable foundation for the preparation of a floodplain risk management study for the catchment.

This report forms the Overland Flow Flood Study for the College, Orth and Werrington 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 and appendices
- Volume 2: contains all figures and maps

2 METHODOLOGY

2.1 Objectives

Penrith City Council outlined a range of objectives for the College, Orth and Werrington Creeks catchments Overland Flow 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 stormwater drainage system, creeks and hydraulic structures
- to develop a computer flood model to simulate the transformation of rainfall into runoff and determine how that runoff would be distributed across the catchment;
- to calibrate and validate the computer model to reproduce past floods;
- to use the calibrated and validated computer model to define 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 2 year ARI 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 assess the trunk drainage system capacity for pipes greater than 600mm in diameter
- to produce maps showing flood hazard and hydraulic categories based on definitions provided in the *'Floodplain Development Manual'* (NSW Government, 2005) for the 5%, 1% AEP flood and 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 assess the potential impact of uncertainty on the results produced by the model;
- to provide 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 an integrated computer based hydrologic and hydraulic model to simulate the transformation of rainfall into runoff and simulate the movement of floodwaters across the catchment (Chapter 4);

- calibration and validation of the computer model to reproduce historic floods (Chapter 5);
- use of the computer models to determine peak discharges, water levels, depths, flow velocities and flood extents for the full range of design events up to and including the PMF for existing topographic and development conditions (Chapter 6);
- use of the computer model results to generate flood hazard, hydraulic category and flood risk precinct mapping (Chapter 7);
- use of the computer model results to undertake flood emergency response classification of communities for the full range of flood events (SES intelligence report) (Chapter 7);
- testing the sensitivity of the results generated by the computer model to variations in model input parameters as well as climate change (Chapter 8);
- use of computer model outputs and sensitivity analysis results to prepare flood planning area mapping (Chapter 9); and,
- summarising the impact of flooding on key facilities and transportation links.

3 DATA COLLECTION AND REVIEW

3.1 Overview

A range of data were made available to assist with the preparation of College, Orth and Werrington Creeks Catchment Overland Flow 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 Report on Hydrology and Hydraulics Study for Werrington Creek (1990)

The *'Report on Hydrology and Hydraulics Study for Werrington Creek'* was prepared by Lyall & Macoun Consulting Engineers for Penrith City Council. The study was undertaken to define hydrologic and hydraulic conditions in the upper reaches of the Werrington Creek catchment, south of Victoria Road, Kingswood, in order to facilitate drainage and development design in the area.

An XP-RAFTS hydrologic model was developed to define the hydrology across the upper Werrington Creek catchment under existing (i.e. 1990) conditions. The study determined a critical duration of 2 hours for the catchment, and the model was used to simulate a range of design events and define peak flows for the 1%, 2%, 5%, 10% and 20% AEP events.

The peak discharges were then used to estimate peak flood levels for the range of design events at 8 key locations within the catchment. The methodology was based on the U.S. Bureau of Public Roads procedure and was implemented in a computer program developed by the University of Wollongong. The locations where flood levels were calculated corresponded to major culvert crossings of upper Werrington Creek at:

- Victoria Street (upstream);
- Park Avenue (upstream and downstream);
- Main Western Railway east branch (upstream and downstream);
- Main Western Railway west branch (upstream);
- Great Western Highway (upstream); and,
- Second Avenue (upstream).

These investigations predicted that the culvert at Victoria Street would not be overtopped in events up to and including the 1% AEP event. However, flow would not be contained with the creek channel upstream of Victoria Street.

The Park Avenue culvert was predicted to experience overtopping for all design events considered. Second Avenue is also predicted to overtop during higher magnitude flood events.

While the Main Western Railway Line culvert crossing was not predicted to overtop in any of the design events considered, it was predicted to experience considerable backwater effects as floodwaters "drown out" the culvert and build up behind the embankment. As a result, the study recommended increasing the capacity of the culvert crossing by "jacking" additional pipes through the embankment.

The study also highlighted that the culverts within Chapman Gardens, upstream of the Great Western Highway, only had sufficient capacity to handle peak flows up to and including approximately a 1 in 30 year ARI (~3% AEP) event. Once the culvert is running full, floodwaters are predicted to spill over onto the Great Western Highway and run along the roadway. The study highlights previous plans to upgrade a section of this culvert to address the issue and recommends that this work be considered further.

It should be noted that trunk drainage upgrades to the Chapman Gardens system were undertaken in 2009. These included upgrades to inlet structures, pipe upgrades and the construction of a levee bank in the north-eastern corner of Chapman Gardens, adjacent to the Great Western Highway and Cosgrove Crescent.

3.2.2 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. The study was completed to define the nature and extent of overland flood behaviour across the LGA and generate sufficient information to 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 to generate overland flow estimations.

A coarse and fine grid combination was used for the modelling. 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 the 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 Werrington Creek subcatchment was modelled using a 3 metre grid.

The existing sub-surface stormwater drainage infrastructure was not included in the model, and 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 considered to be conservative.

Only significant 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. Therefore, flood behaviour around culverts and bridges may not be reliably represented.

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.

Hydraulic modelling was carried out for the 5% and 1% AEP events and the PMF to define flows, flood levels and velocity across the LGA. The overland inundation extents generated by the model are included on **Figure 2**. The model results were also used to define the provisional flood hazard.

The LGA was divided into subcatchments approximately 100 hectares in size. These subcatchments were then refined using the results of the detailed modelling of overland flow. A total of 249 subcatchments were delineated.

The flood risk in all subcatchments within the Penrith LGA was assessed 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 from the Annual Average Damages (AAD) for each subcatchment.

The 249 sub-catchments that were assessed were split into 10 percentile bands, with the 10% band representing the highest 10% of the flood affected sub-catchments. The Overview Study identified the College, Orth and Werrington Creeks catchment within the highest 10% of flood affected subcatchments across the LGA.

3.2.3 WELL Precinct Hydrology and Catchment Management Study (2006)

The *"WELL Precinct Hydrology and Catchment Management Study"* was undertaken by Cardno Willing for Penrith City Council. The study was undertaken at a strategic level to identify stormwater quantity and quality management issues and to develop management principles for input into the planning of the Werrington Enterprise Living and Learning (WELL) Precinct.

The WELL precinct covers an approximate area of 670 hectares. It contains land owned by the University of Western Sydney (now Western Sydney University), TAFE, Sydney Water, Landcom and a small number of other institutional and private land holdings. It includes the

Caddens Release Area and the Werrington Mixed Use site ("Signals" site). Both Werrington Creek and Claremont Creek drain through the precinct.

An XP-RAFTS hydrologic model was established for the study area to derive design flow estimates for a range of catchment conditions for both the Werrington Creek and Claremont Creek catchments. This included a 1,215 hectare section of the Werrington Creek catchment.

The XP-RAFTS model was used to simulate the 1%, 2%, 5%, 20% and 100% AEP and Probable Maximum Precipitation (PMP) events under existing catchment conditions. The 2 hour storm duration was determined to be the critical duration for the Werrington Creek catchment.

The study included the development of separate MIKE-11 hydraulic models for Werrington and Claremont Creeks. In addition, a coarse 2-dimensional TUFLOW model was also established to examine potential 2-dimensional flow effects and interaction between the two creeks. The MIKE-11 model for Werrington Creek extends from immediately downstream of Kingswood Road and Castle Road intersection, to about 200 metres downstream of John Oxley Avenue. It consists of over 40 cross-sections that were extracted from a DTM generated from 2002 ALS data supplied by Penrith City Council. The model includes the crossings of Werrington Creek at Caddens Road, O'Connell Street, Cosgrove Crescent, Great Western Highway, Main Western Railway Line, Victoria Street, William Street, Burton Street, and John Oxley Avenue.

Tailwater conditions for Werrington Creek were defined by South Creek flood levels, which were extracted from the results of MIKE-11 modelling undertaken for the *"Flood Study Report, South Creek"* (DWR, 1990). For this study, a conservative assumption was made that local flooding in Werrington Creek coincides with South Creek flooding. Design runs were completed for the 1%, 2% and 5% AEP and PMF events. Provisional hazard and hydraulic categories were also determined and mapped for the site.

The model results were validated through a comparison of predicted peak 1% AEP flood levels with previous HEC-2 flood level estimates from the *"Flood Study Report, South Creek"* (DWR, 1990). MIKE-11 flood level estimates were found to be lower than the previously predicted flood levels by the HEC-2 model. This may be due to the fact that the MIKE-11 model developed for this study predicts a large attenuation of flow in Werrington Creek. This flow attenuation is partly due to the relatively flat floodplain in the upper sections of the creek as well as the large number of roadway embankments which serve as small detention basins.

The proposed water management strategy developed by the study sets a target for no increase in peak 1%, 5% and 20% AEP runoff rate at key locations due to development within the precinct. In order to achieve this, a number of detention basins are proposed within the catchment. For Werrington Creek, these include:

- Upgrading the existing pond at the University of Western Sydney to provide flood detention;
- Eight (8) off-line basins serving subcatchments draining to Werrington Creek;
- On-site detention at the Precinct Centre site (subject to catchment–wide assessment);

- 2 new off-line basins serving Werrington Creek tributary subcatchments upstream of the Main Western Railway at 'Signal' land; and,
- One (1) existing pond modified to provide flood detention for flows from Werrington Creek tributary at 'Signals' land.

The study also recommended that urban development be excluded from the area affected by the 1% AEP flood along the watercourses.

3.2.4 Caddens Release Area – Catchment Management, Hydrology and Water Quality Report (2007)

The *"Caddens Release Area – Catchment Management, Hydrology and Water Quality Report"* was prepared by Hughes Trueman for Landcom. The study builds on the *"WELL Precinct Hydrology and Catchment Management Study"* (Cardno Willing, 2006) and addresses stormwater management, water quality, floodplain management and riparian management issues for the Caddens development area which forms part of the WELL Precinct.

The site is bisected by Werrington Creek, which flows from south to north through the site. A tributary of Werrington Creek also drains through the western portion of the site. Werrington Creek has a catchment area of approximately 77 hectares upstream of the site.

A hydrologic model of the site and adjoining catchment was developed using the XP-RAFTS software to estimate existing and post-development discharges and evaluate existing stormwater infrastructure adjacent to the site. The model was used to simulate the 1%, 10% and 20% AEP events for pre-development conditions and the 1%, 10% and 20% AEP and PMF events for post-development conditions. A 2 hour storm was determined to be the critical duration for the majority of the subcatchments within the model, however some smaller subcatchments were found to have a 25 minute critical duration.

Peak 1% AEP flows generated by the XP-RAFTS model developed for this study were validated against flows estimated as part of the *"WELL Precinct Hydrology and Catchment Management Study"* (Cardno Willing, 2006). The comparison found that this study generated higher peak flows for smaller sub-catchments and this was contributed to by a finer subcatchment breakup with higher average slopes. Along Werrington Creek, the flows generated by both studies were within 10% of each other.

The results of the modelling found that the trunk drainage system within Claremont Meadows had sufficient capacity to accept runoff from the Caddens Release area. The model was also used to estimate the sizes of detention basins required to attenuate peak flows for the post-development scenario to pre-development levels.

The study also assessed the capacity of existing road crossings. For this purpose, a HEC-RAS steady-state hydraulic model was developed along Werrington Creek, extending from downstream of Second Avenue to upstream of Caddens Road and along its tributary within the development site. Cross-sections included within the model were extracted from ground survey collected for the development site and were situated between 4 and 93 metres apart, with an average spacing of less than 40 metres.

HEC-RAS models were developed to represent pre-development and post-development conditions. The post-development scenario included modifications to the cross-sections within the model to reflect re-grading of the site, the removal of a dam within the site, and the addition of a new culvert crossing of Werrington Creek. Simulations were undertaken for the 1% and 10% AEP and PMF events for pre-development conditions, and for the 1% and 20% AEP and PMF events for post-development conditions.

The pre-development HEC-RAS model was used to assess the capacity of two (2) Werrington Creek culvert crossings of Caddens Road and at the University of Western Sydney entrance. HEC14 was also used to assess the capacity of five (5) culverts located on rural roads adjacent to the site, including Caddens Road (3 culverts) and O'Connell Street (2 culverts). It was determined that these road culverts do not meet Council's design standards of conveying flows in events up to and including the 5% AEP event.

The post-development HEC-RAS model was also used to design a new culvert at a road crossing of Werrington Creek within the site, as well as upgrades to an existing culvert under O'Connell Street to enable integration of the culvert into the proposed piped stormwater system for the site.

The results of the modelling were also used to assess the impact of flooding on the proposed development, as well as the impact of the development on existing flood conditions across the Werrington Creek floodplain. The study determined that all proposed roads within the site would be elevated above the peak 1% AEP flood level. The proposed development was also considered to have negligible impact on existing peak flood levels and flows outside of the site.

At the time the current flood study was being prepared, development of the Caddens area was primarily complete. Nevertheless, some development was on going.

3.2.5 Flood Study for Land at Werrington Creek, Kingswood (2006)

The *"Flood Study for Land at Werrington Creek, Kingswood"* was prepared by Cardno Willing. The study was commissioned by Penrith City Council to investigate flooding at the site, referred to as Lot 101 in DP 876202 at Great Western Highway, Kingswood (located between the Great Western Highway and Cosgrove Crescent). In particular, the study aimed to assess any potential impact of the proposed development of the WELL Precinct on flood risk within this site and determine flood mitigation options. The site straddles the alignment of Werrington Creek and contains an underground stormwater conduit, grassed floodway and a large surcharge pit.

The hydrologic and hydraulic models developed as part of the *"WELL Precinct Hydrology and Catchment Management Study"* (Cardno Willing, 2006) were used as the basis of the modelling undertaken for this study. However, the MIKE-11 model created as part of the *"WELL Precinct Hydrology and Catchment Management Study"* (Cardno Willing, 2006) was updated in the vicinity of Cosgrove Crescent to include additional stormwater drainage details extracted from plans provided by Council. The updated model was then used to simulate the 1%, 2% and 5% AEP events and the PMF.

ALS data provided by Council was used to establish proposed ground surface elevations within the developed site and assess these levels against predicted peak flood levels and extents. It was concluded that the filled portion of the site is predicted to remain above the peak 1% AEP flood level. Therefore, there are no significant restrictions on development of the land at or above this level.

The study also highlights that the overland flow path along the southern section of the property is an integral part of the flow regime within this section of Werrington Creek and should be retained. It suggests that access such as footpaths or driveways could be constructed across the flowpath but should be designed so as not to obstruct overland flow.

The study also compared predicted peak flood levels with surveyed floor levels of buildings in Cosgrove Crescent to the south of the site. It was found that all of the buildings have floor levels that are above the peak 1% AEP flood level.

3.2.6 Werrington Subdivision, Corner of French Street & Great Western Highway Kingswood – Civil, Flooding and Stormwater Management Report (2011)

The *"Werrington Subdivision, Corner of French Street & Great Western Highway, Kingswood –Civil, Flooding and Stormwater Management Report"* was prepared by Cardno ITC for Middle East Pty Ltd. The study was undertaken to define the flood risk and document a stormwater management strategy and road network design for the proposed subdivision of the site located at the corner of French Street and the Great Western Highway, Kingswood.

The site is traversed by a tributary of Werrington Creek which drains in a northerly direction through the site from the Great Western Highway at the southern site boundary to the Main Western Railway at the northern site boundary. A flood study was undertaken to determine the impacts of flooding on the site, and included hydrologic and hydraulic modelling.

A DRAINS model was developed to estimate the peak flood discharges through the site for pre-development and post-development conditions. This model was based on four (4) subcatchments draining to or through the site. Simulations were completed for the 1%, 5% and 20% AEP events.

A one-dimensional (1D), steady-state hydraulic model of the tributary of Werrington Creek was developed using the HEC-RAS software. The model extended along the watercourse from immediately upstream of the Great Western Highway to immediately upstream of the culvert crossing of the Main Western Railway.

Cross-sections were extracted from a DTM created from detailed survey of the site. Peak discharges extracted from the results of the DRAINS modelling were used to define the peak flows within the watercourse. Tailwater conditions were determined through the application of a Normal Depth calculation for the creek channel at the downstream boundary. The model was used to simulate three (3) scenarios – pre-development conditions with no culvert blockage; pre-development conditions with 50% culvert blockage and post-development conditions with 50% culvert blockage. Cross-sections within the model for post-development conditions were modified to reflect proposed changes to the topography within the site. Proposed flow diversions resulting from the proposed subdivision were also incorporated into the hydraulic model layout for the post-development scenario. Simulations were completed for the 1%, 5% and 20% AEP events.

The results of the modelling for existing conditions with no culvert blockage found that the Main Western Railway culvert was at full capacity at the peak of the 1% AEP flood and as a result, the railway line would be overtopped by floodwaters. Moreover, assuming 50% culvert blockage, the railway line could be potentially overtopped at the peak of the 5% AEP event under existing conditions.

The proposed subdivision was not predicted to result in any increases to flood levels along the main watercourse. The proposed road levels within the site are at least 160mm above the peak 100 year flood level and the finished elevations of proposed residential lots would provide a minimum of 500mm freeboard above the peak 1% AEP flood level.

At the time the current flood study was being prepared, the French Street subdivision was partly constructed.

3.2.7 Caddens Knoll Stormwater Management Report (2013)

The *"Caddens Knoll Stormwater Management Report"* was prepared by J. Wyndham Prince for Landcom. The study was undertaken to investigate stormwater management issues affecting the proposed rezoning and development of Lot 2107 DP 2631589 Caddens Road, Orchard Hills and prepare a proposed drainage strategy to ensure no adverse flood impacts occured within the catchment.

Detailed hydrologic and hydraulic modelling was completed to determine whether the existing stormwater infrastructure downstream of the site had sufficient capacity to cater for discharges from the Caddens Knoll development site under both current zoning and proposed rezoning configurations.

An additional basin was proposed to manage the stormwater runoff from the part of the site draining south to Caddens Road. Hydrologic modelling for the proposed detention basin was carried out using the XP-RAFTS software package in order to estimate the detention volume required. The model was simulated for existing and post-development conditions for the 1% and 20% AEP events. Results of the modelling determined that the proposed detention basin adjacent to Caddens Road, with a 400m^3 volume, would attenuate flows from the south of the site to restrict peak post-development flows to pre-development levels. This basin has since been constructed.

Hydrologic and hydraulic modelling of the existing minor and major stormwater network, including the existing detention basins, was also undertaken using the DRAINS software package. The DRAINS model was used to simulate existing and post-development conditions for the 1% and 20% AEP events.

The results of the DRAINS modelling found that under its current zoning (i.e. part residential and part rural residential), the majority of the existing stormwater infrastructure has sufficient capacity to convey discharges from the Caddens Knoll site and surrounding residential developments. The modelling also showed that there are a number of pits in the area where flows surcharge in the 20% AEP event.

For post-development conditions (i.e., incorporating 45 residential lots, park and associated infrastructure), the results of the modelling predicted only a minor impact on the existing stormwater network. Moreover, only a minor increase in water levels in the basins was predicted as a result of the increased discharge from the Caddens Knoll site.

3.2.8 Updated South Creek Flood Study (2015)

The *"Updated South Creek Flood Study"* was prepared by WorleyParsons Services Pty Ltd on behalf of Penrith City Council, acting in association with Liverpool, Blacktown and Fairfield City Councils. The objective of the study was to update the existing hydrologic and hydraulic models that were previously developed for the catchment as part of the *"Flood Study Report, South Creek"* (DWR, 1990) and provide contemporary tools for the assessment of flood conditions across the South Creek catchment. The results of the study define the flood behaviour within the South Creek catchment for a range of design floods and provide more reliable estimates of planning flood levels for each local government area.

The flood study covers the South Creek catchment extending from Bringelly Road in the south to the Blacktown Road-Richmond Road Bridge crossing in the north. The total study area is about 240 km^2 and lies within the Hawkesbury, Penrith, Blacktown, Liverpool and Fairfield Local Government Areas. Werrington Creek is a minor tributary of South Creek and, as such, the Werrington Creek catchment is included within the extents of the study area.

The report documents two historic flood marks for the Werrington Creek catchment for the 1986 and 1988 floods. The location and elevation of each flood mark is shown in **Figure 2**.

The XP-RAFTS model of the South Creek catchment developed for the 1990 Flood Study was updated from the 1991 version of the software (Version 2.56) to a later version of XP-RAFTS (Version 6.52). Subcatchment delineation and parameters were reviewed and refined based on contemporary topographic and catchment conditions, and in order to improve the relationship between the hydrologic and hydraulic models for this study. The XP-RAFTS model represented the Werrington Creek catchment using two (2) subcatchments and the critical duration of the Werrington Creek catchment was determined to be 2 hours.

Validation of the updated XP-RAFTS model was based on a comparison between the peak discharges and hydrograph shape produced by the XP-RAFTS model developed for the 1990 Flood Study and the results of the latest XP-RAFTS model for the 1% AEP event. Flows produced by the updated model were within approximately 10% of flows from the 1990 Flood Study. Differences between the modelling results were thought to be due to updated subcatchment delineations and impervious percentages that were reviewed as part of the flood study.

A 2D hydraulic model of the South Creek system was developed using the RMA-2 software package to replace the previous 1D MIKE-11 and HEC-2 hydraulic models that were developed as part of the 1990 Flood Study. The model is based on a Digital Terrain Model (DTM) developed from ALS data that was gathered for the entire South Creek floodplain between

2002 and 2006. The RMA-2 model only includes the Werrington Creek floodplain from the William Street Footbridge, downstream to the confluence with South Creek. As such, it did not include hydraulic modelling of the entire Werrington Creek catchment that forms the study area for this project.

The results of the RMA-2 model were validated against the results of the MIKE-11 and HEC-2 models produced for the 1990 study for the 1% AEP flood. Overall, there was found to be reasonable correlation between the peak 1% AEP flood levels produced by the MIKE-11 and HEC-2 models and those predicted by the RMA-2 model.

The XP-RAFTS and RMA-2 models were used to simulate a range of design floods, including the 0.2%, 0.5%, 1%, 2% and 5% AEP events and the Probable Maximum Flood. The report documents the findings from the modelling investigations, including details on flows, flood levels, flood depths, flow velocities, and provisional hydraulic and hazard categories for current catchment and floodplain conditions. RMA-2 model outputs were provided as part of the current study in waterRIDE outputs. Accordingly, a range of spatial and temporal flood information could be extracted for each design event.

The results of the study indicate that the suburb of Werrington can be inundated as a result of a number of scenarios associated with overtopping of either the Werrington Road Levee to the east, the Werrington Earthen Levee to the west, or even failure of the flood gate on the earthen levee.

The study assessed the performance of the Werrington Road and Werrington Earthen Levees and indicated that the Werrington Road Levee and Werrington Earthen Levee would not be overtopped by floodwaters from South Creek during events up to and including the 0.5% AEP flood. Inundation of Werrington is predicted at the peak of the 0.2% AEP event as flood levels along South Creek to the east become high enough to overtop the Werrington Road Levee. However, the Werrington Road Levee acts to reduce the depth of inundation across Werrington during events up to the PMF.

3.3 Hydrologic Data

3.3.1 Historic Rainfall Data

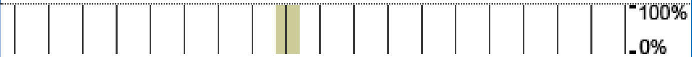
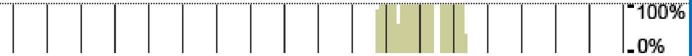
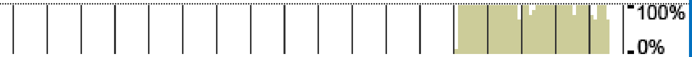
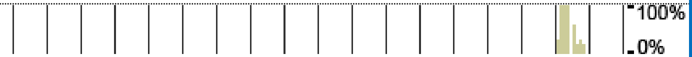
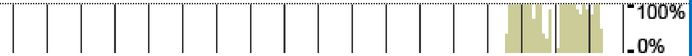
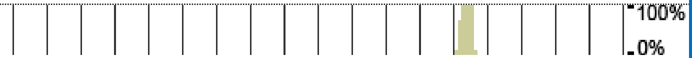
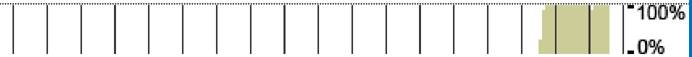
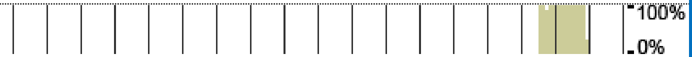
A number of daily read and continuous (i.e., pluviometer) rainfall gauges are located near the catchment. The location of each gauge is shown in **Figure 3**. Key information for each gauge is summarised in **Table 1**.

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

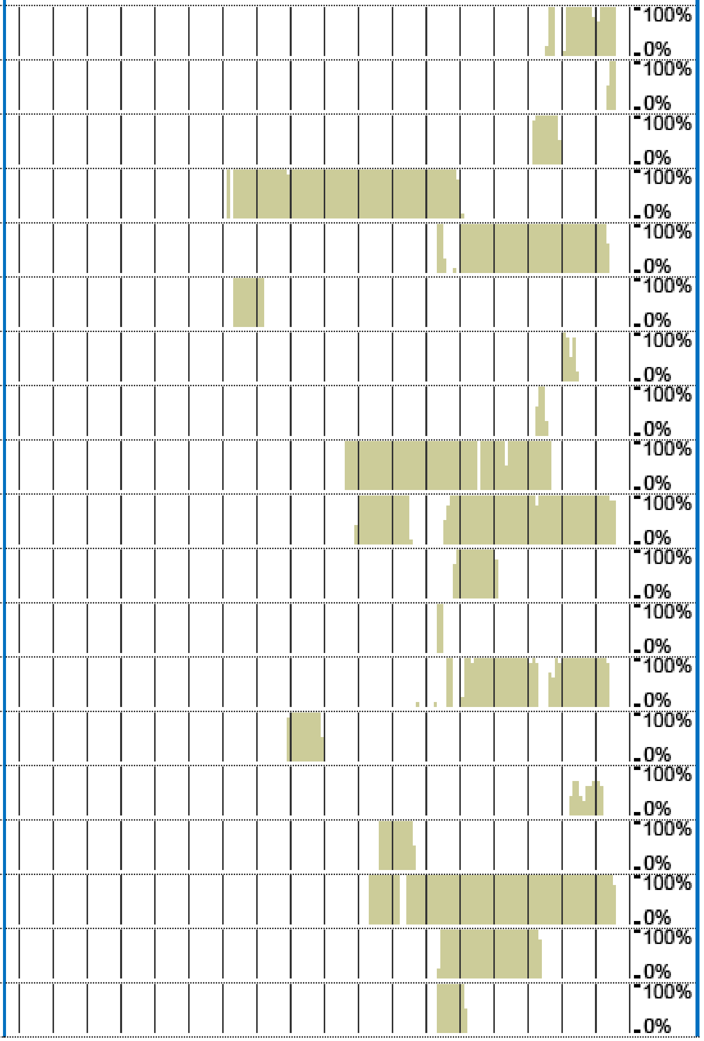
3.3.2 Historic Stream Gauge Data

There are no stream gauges located within the College, Orth and Werrington Creeks catchment. Accordingly, no stream flow information could be uncovered for the study area.

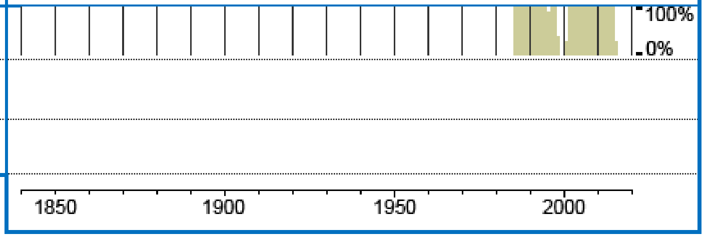
Table 1 Available rain gauges in the vicinity of the College Orth & Werrington Creeks catchment

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Catchment (km)	Temporal Availability and Percentage of Annual Record Complete
567158	Orchard Hills (Kingswood Road Reservoir)	Continuous	SW	1991 Aug	2015 May	2.3	
67096	Penrith (Glenroy)	Daily	BOM	1917 Jan	1923 Dec	3.1	
67024	St Marys Bowling Club	Daily	BOM	1897 Jul	1984 Dec	3.2	
567156	Orchard Hills (Flinders AV)	Continuous	SW	1991 Aug	2015 May	3.9	
67025	St Marys Mwsdb	Daily	BOM	1947 Feb	1973 Apr	4.5	
567087	St Marys STP	Continuous	SW	1990 Jan	2015 May	4.6	
567159	Mount Pleasant (Cranebrook Reservoir)	Continuous	SW	1991 Aug	2015 May	4.7	
567082	Orchard Hills (Orchard Hills WTW)	Continuous	SW	1991 Aug	2015 May	5.1	
67084	Orchard Hills Treatment Works	Daily	BOM	1970 Dec	2015 Aug	5.1	
67018	Penrith Ladbury Avenue	Daily	BOM	1890 Jan	1995 Oct	5.1	
67003	Colyton (Carpenter St)	Daily	BOM	2000 Oct	2008 Feb	5.4	
567163	Regentville Rural Fire Service	Continuous	SW	1992 Sep	2015 May	5.8	
67102	St Clair (Juba Close)	Daily	BOM	1985 Sep	2013 Jul	6.2	
67004	Emu Plains	Daily	BOM	1880 Jan	1973 Jun	6.3	
67083	Mount Druitt Francis St	Daily	BOM	1970 Dec	1976 Jan	6.3	
67113	Penrith Lakes Aws	Daily	BOM	1995 Sep	2015 Nov	6.9	
		Continuous	BOM	1996 Jan	2015 Nov		
67115	Glenmore Park (Cartwright Cl)	Daily	BOM	1995 Jan	2009 Apr	7.0	
67067	Emu Plains	Daily	BOM	1911 Jan	1996 Dec	7.1	

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Catchment (km)	Temporal Availability and Percentage of Annual Record Complete
67116	Willmot (Resolution Ave)	Daily	BOM	1995 Oct	2015 Nov	7.4	
67066	Erskine Park Reservoir	Daily	BOM	2013 Jul	2015 Nov	7.6	
67118	Oakhurst (Lawton Place)	Daily	BOM	1991 Mar	1999 May	9.8	
67016	Minchinbury	Daily	BOM	1901 Feb	1970 Aug	10.2	
63185	Glenbrook Bowling Club	Daily	BOM	1963 Jan	2013 Jul	10.5	
63206	Wascoe	Daily	BOM	1903 Jan	1911 Dec	11.0	
67000	Eastern Creek (Wonderland)	Daily	BOM	2000 Feb	2004 Feb	11.6	
67106	Berkshire Park First Rd	Daily	BOM	1992 Jul	1995 Mar	11.7	
67068	Badgerys Creek McMasters F.STN	Daily	BOM	1936 Jan	1996 Dec	11.8	
67002	Castlereagh (Castlereagh Rd)	Daily	BOM	1939 Sep	2015 Nov	12.1	
63230	Blaxland Western Highway	Daily	BOM	1968 May	1980 Sep	12.4	
67039	Ajana	Daily	BOM	1963 Jan	1964 Dec	12.8	
67076	Quakers Hill Treatment Works	Daily	BOM	1957 May	2013 Nov	13.5	
67050	Badgerys Creek School	Daily	BOM	1919 Jan	1929 May	13.6	
63183	Valley Heights (Sun Valley Rd)	Daily	BOM	2002 Sep	2011 Oct	13.7	
63078	Springwood (Journeys End)	Daily	BOM	1946 Jan	1956 Jun	14.1	
67029	Wallacia Post Office	Daily	BOM	1943 Feb	2015 Sep	14.2	
67059	Blacktown	Daily	BOM	1963 Nov	1993 Sep	14.2	
67092	Quakers Hill Douglas Rd	Daily	BOM	1963 Feb	1971 May	14.3	



Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Catchment (km)	Temporal Availability and Percentage of Annual Record Complete
63286	Winmalee (Pentlands Drive)	Daily	BOM	1985 Jan	2015 Mar	14.5	
67119	Horsley Park Equestrian Centre AWS	Continuous	BOM	1997 Sep	2015 Nov	14.8	
67108	Badgerys Creek AWS	Continuous	BOM	1996 Jan	2015 Nov	14.9	



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

3.4 Topographic and Survey Information

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

- 2011 Light Detection and Ranging (LiDAR) survey
- 2002 Aerial Laser Survey (ALS)
- Caddens design and work-as-executed Digital Terrain Models (DTM)
- French Street Subdivision design DTM

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 full extent of the College, Orth and Werrington Creeks catchment. 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.

As the LiDAR was collected relatively recently, it is considered to provide a reliable representation of contemporary topographic conditions across the majority of the catchment. Nevertheless, some areas have been developed since 2011. This includes:

- Caddens and Caddens Knoll subdivisions; and
- French Street subdivisions.

As a result, the topography across these new development areas will not be reliably defined by the 2011 LiDAR. Further information on how the topography across these new development areas was defined is provided in **Sections 3.4.3** and **3.4.4**.

3.4.2 2002 ALS

ALS was collected across the Penrith City Council LGA in 2002. This include the full extent of the College, Orth and Werrington Creeks catchment. 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. In order to identify areas where the 2002 ALS may not provide a reliable representation of contemporary topographic conditions, a terrain 'difference map' was prepared by subtracting 2002 ground surface elevations from 2011 ground surface elevations. The difference map is shown in **Plate 1** and shows locations where changes in elevation of more than 0.2 metres are prevalent. Decreases in ground surface elevation that have occurred since 2002 are shown in blue and increases in ground surface elevation are shown in red.

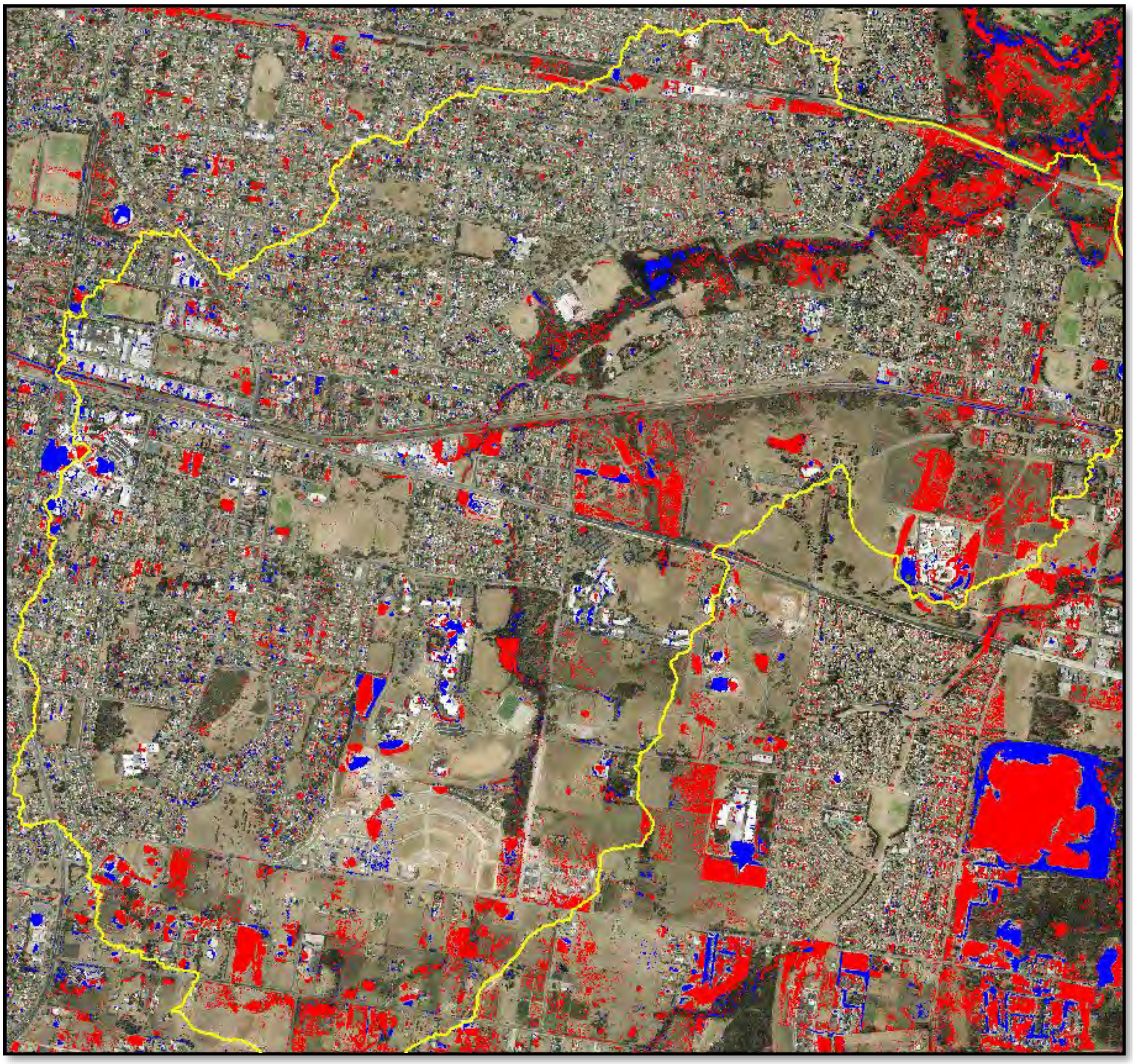


Plate 1 Differences in terrain elevation of more than 0.2 m between 2002 and 2011 datasets. Red indicates increases in elevation since 2002 and blue indicates decreases in elevation since 2002

The terrain difference map indicates that there are some notable changes in ground surface elevations across some sections of the catchment. In most cases, these topographic changes appear to be a result of re-development of residential lots that have occurred since 2002. In addition, some notable differences are evident across the Caddens and French Street subdivisions. Further information on the Caddens and French Street developments is provided in the following sections.

The ALS generally provides a good representation of the variation in ground surface elevations across the catchment. 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. The difference map provided in **Plate 1** shows some notable differences between the 2002 and 2011 datasets in areas of significant vegetation. Therefore, additional checks were performed to verify the reliability of each dataset in areas of dense vegetation.

Plate 2 provides an example of the 2002 ALS point density in the vicinity of Victoria Street, Kingswood. **Plate 2** shows a high ALS point density across grassed and paved areas but reduced ground points in the vicinity of the dense tree and vegetation coverage. **Plate 2** also shows no ground points across buildings. Therefore, it appears that non ground points have correctly been removed from the 2002 dataset. Overall, the 2002 dataset is considered to provide a more reliable description of the variation in terrain in the vicinity of vegetation. Therefore, the 2002 ALS was used in preference to the 2011 LiDAR across vegetated areas (refer to "polygons" in **Figure 4** to determine where 2002 ALS has been used in preference to 2011 LiDAR).

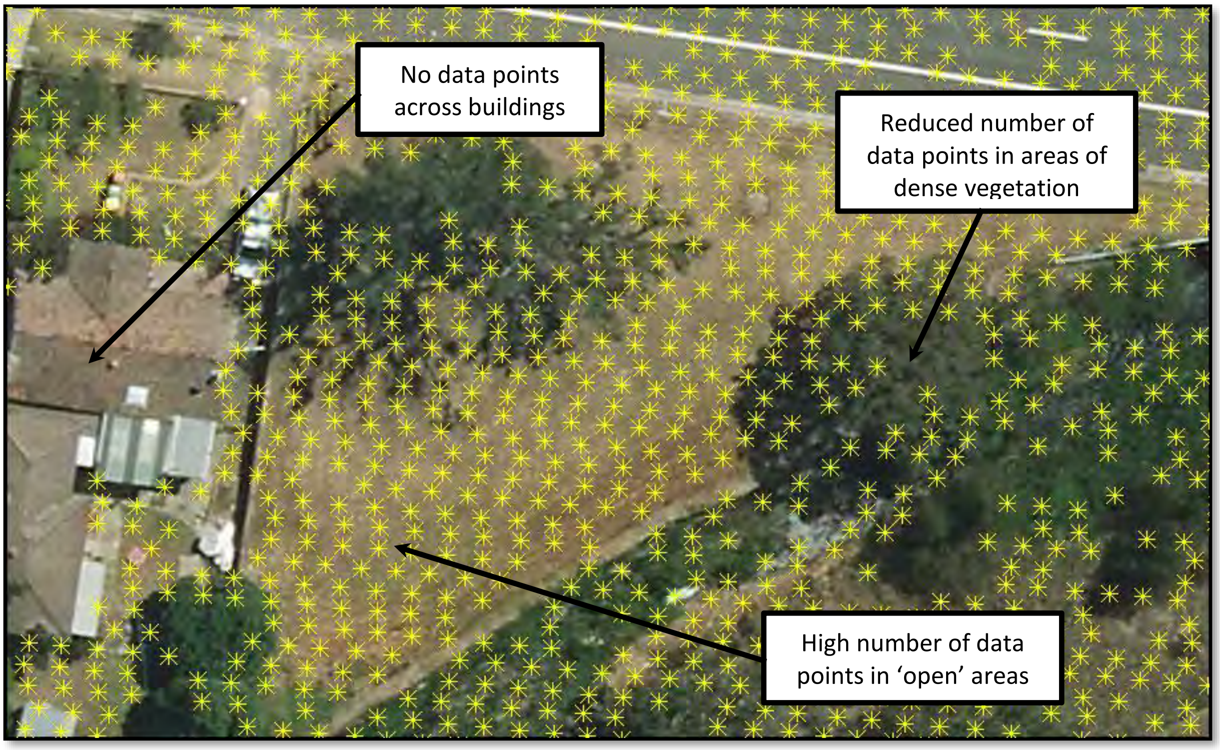


Plate 2 ALS data points (yellow crosses) in the vicinity of Werrington Creek crossing of Victoria Street

Nevertheless, the reduced point density shown in **Plate 2** means that there will be a less detailed representation of the variation in terrain in areas of dense vegetation. Therefore, it was considered necessary to supplement the 2002 ALS data with detailed ground survey in areas of high vegetation density (e.g., creeks) to ensure a reliable representation of the terrain in major conveyance areas was provided. In this regard, creek cross-sections were surveyed as part of the project. Further details on the cross-section survey is provided in **Section 3.8**.

It was also recognised that the ALS data will not pick up the details of drainage features that are obscured from aerial survey techniques, such as bridge and culvert dimensions and subsurface stormwater pits and pipes. Therefore, survey of hydraulic structures and the stormwater system was also completed to ensure a reliable representation of these drainage structures was provided. Further details of the hydraulic structure and stormwater survey is provided in **Section 3.8**.

3.4.3 Caddens Digital Terrain Models

As discussed, the Caddens area has been developed since the 2002 ALS and 2011 LiDAR information was collected. Therefore, it was necessary to supplement the ALS and LiDAR with design and work-as-executed survey information to ensure a reliable representation of contemporary topographic conditions was provided across the Caddens subdivision. Accordingly, the following datasets were provided by Council to assist in this regard:

- Caddens Knoll Design Digital Terrain Model (DTM);
- Caddens Road West Work-as-Executed DTM;
- Caddens Stages 1 & 2 Design DTM
- Caddens Stage 3 to 6 Design DTM
- Caddens Pond Design DTM (pond located near the corner of Caddens Road and O'Connell Lane)

The extent of the area covered by the Caddens datasets is shown in **Figure 4**. In all cases, the Caddens datasets were used in preference to the 2011 and 2002 datasets across these areas to develop the DEM shown in **Figure 4**.

3.4.4 French Street Subdivision Terrain Models

At the time this study was being prepared, the French Street subdivision was also being constructed. As this flood study will serve as the basis for defining flood behaviour for a number of years into the future it was considered important to include a fully constructed description of the French Street subdivision in the terrain representation. Accordingly, design terrain information for the subdivision was provided by Council and was used to define the topography across the French Street subdivision. The extent of the area where the design terrain information was used is shown in **Figure 4**.

3.5 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 2002, 2008 and 2014. Non ortho-rectified imagery was also provided for 1981, 1985, 1988 and 1992
- Cadastre – provides property boundary polygons
- Contours: provides ground surface elevation contours at 0.5 metre intervals derived from the 2002 ALS and 2011 LiDAR
- Drainage Infrastructure – shows the location of key components of the drainage system including open channels, stormwater pipes and pits, headwalls and culverts. The datasets were collated from a variety of data sources including old paper maps and the accuracy or completeness of the data is unknown.
- Easements – Shows the locations of drainage, sewer and water easements
- Roadways – provides polygons and centrelines for all roadways in the catchment
- Suburbs – provides suburb polygons
- Watercourses – shows the alignment of major watercourses, including College, Orth and Werrington Creeks as well as several unnamed tributaries

The extent of the cadastre, stormwater, suburbs and watercourse GIS layers is provided on **Figure 2**.

In general, the GIS layers provide a suitable basis for preparing report figures as well as informing the computer flood model development. However, a review of the stormwater pits and pipes GIS layers revealed that only limited information was included describing the stormwater system. This included pipe size and length information but did not include other key attributes including grate and lintel sizes. In addition, the spatial accuracy of the stormwater dataset across some sections of the catchment was poor. The stormwater system can convey a significant proportion of flood flows during most storms. Therefore, it was considered important to include a reliable representation of the stormwater system in the computer model. In this regard, it was considered necessary to undertake a survey of the stormwater system to ensure a full and reliable description of the stormwater system could be provided in the computer model. Further information on the stormwater system survey is provided in **Section 3.8**.

3.6 Remote Sensing

In addition to providing ground point elevations, the 2011 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 model.

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
- Impervious (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.

As discussed, the Caddens and French Street subdivisions were being developed at the time this study was prepared. As a result, the remote sensing outputs will not reflect current or future land use across these areas. Therefore, a separate 'area currently under construction' land use classification was introduced.

The final remote sensing output is shown in **Figure 5**.

3.7 Engineering Plans

A range of engineering plans were also provided by Council. The plans provided design details and work-as-executed survey for a range of drainage infrastructure (primarily stormwater pits and pipes) across the Caddens and French Street subdivisions. This included:

- "Caddens Release – Werrington Creek Pond" (Cardno, 2009);
- "Caddens Release Stage 1" (Cardno, 2010);
- "Caddens Release Civil Works, Construction Certificate, Stage 2" (Cardno, 2012);
- "Caddens Collector Roads, Kingswood – O'Connell Lane Detail Design" (Cardno Young, 2012);
- "Caddens Stage 3, Council Ref: DA 11/0139, Proposed Lot, Road and Drainage Works: Construction Certificate" (J. Wyndham Prince, 2013);
- "Caddens Stage 4, Council Ref: DA 11/0139, Proposed Lot, Road and Drainage Works: Construction Certificate" (J. Wyndham Prince, 2013);
- "Caddens Stage 5, Council Ref: DA 11/0139, Proposed Lot, Road and Drainage Works: Construction Certificate" (J. Wyndham Prince, 2014);
- "Caddens Stage 6, Council Ref: DA 11/0139, Proposed Lot, Road and Drainage Works: Construction Certificate" (J. Wyndham Prince, 2014);
- "Caddens Knoll Construction Certificate, Proposed Subdivision of Lot 21 DP 1151724 – Lot, Road and Drainage Works: Council Ref: 14/0186" (J. Wyndham Prince, 2015);
- "Werrington Subdivision, Cnr French Street & Great Western Highway, Werrington NSW 2747 – Construction Certificate, Civil Package – Stage 2 (DA 11/0546.02" (SGC, 2015);

The stormwater drainage information (i.e., stormwater pits and pipes) contained in each set of plans was extracted and incorporated in a stormwater GIS database for each development area. The location of where stormwater pit and pipe information was extracted from the plans is shown in **Figure 6**. Details for 594 pits and 595 pipes were extracted from the plans.

The details of proposed culverts were also extracted from the plans. The location of culverts extracted from the plans is also shown in **Figure 6**.

3.8 Survey

3.8.1 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, stormwater pits and pipes, culverts and bridges. Consulting surveyors, Lawrence Group, collected the additional survey information (refer **Plate 3**).

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

Plate 3 Survey of stormwater infrastructure completed as part of the study

3.8.2 Stormwater System

Survey of all stormwater pits and pipes that were contained within the catchment but outside of new development areas where drainage plans were available, was completed. This involved the survey of 1,929 pipes and 1,879 pits. The location of stormwater pits and pipes that were surveyed as part of the study is shown in **Figure 6**.

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 invert elevations and lintel and grate sizes as well as pipe sizes and invert elevations.

The surveyed ground surface elevations at the centre of each stormwater pit were also compared against ground surface elevations defined in the 2002 ALS and 2011 LiDAR data to assist in confirming the vertical accuracy of these broad-scale topographic datasets. This involved subtracting the surveyed ground surface elevations from the 2002 ALS and 2011 LiDAR ground surface elevations at each pit location. The elevation differences were then statistically analysed and the outcomes of this analysis are presented in **Table 2**.

Table 2 Percentile Difference Between ALS/LiDAR and Surveyed Ground Elevations

Dataset	Average Difference (metres)	Standard Dev. of Difference (metres)	5th	10th	25th	50th	75%	90th	95th
2002 ALS	0.09	0.16	-0.17	-0.10	0.03	0.12	0.18	0.24	0.29
2011 LiDAR	0.13	0.12	-0.06	-0.03	0.07	0.15	0.20	0.25	0.28

The information presented in **Table 2** indicates that the 2002 dataset provides a smaller average difference (0.09 metres), but a higher variance (as indicated by the larger standard deviation and the wider range of percentile differences). The 95th percentile difference for the 2011 LiDAR is 0.28 metres, which agrees well with the 95% confidence interval reported in the LiDAR metadata (i.e., 0.3 metres). In general, the 2011 LiDAR is considered to provide a better representation of the topography relative to the 2002 ALS.

The 2002 and 2011 datasets generally provide higher elevation estimates relative to the surveyed elevations. This may be associated with the ALS and LiDAR only providing 1-2 elevation points per square metre, meaning that the 2002 and 2011 datasets won't always provide a reliable representation of the sudden changes in topography around kerbs and gutters (i.e., the ALS and LiDAR may be picking up the top of the kerb and nature strip rather than the gutter invert where the pit elevations were surveyed). Most of the elevation differences are contained within 0.15 metres (i.e., an average kerb height). In general, this is considered to be an acceptable level of accuracy for undertaking an overland flood study across an urban catchment.

3.8.3 Creek Cross-Sections

As discussed in Section 3.4.2, LiDAR and ALS can provide a less reliable description of the variation in terrain in areas of dense vegetation, including the major creeks within the catchment. Therefore, cross-sections were surveyed along each of the major creeks 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 200 metre intervals along each creek. This resulted in the survey of sixty-six (66) 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.

3.8.4 Bridges

The details of six (6) bridges located within the catchment were also collected as part of the survey. The location of each bridge that was surveyed is shown on **Figure 6**.

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

3.8.5 Culverts

Thirty-five (35) culverts were also surveyed at various locations across the catchment. The location of each culvert that was surveyed is shown on **Figure 6**.

Key characteristics of each culvert were 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.

3.9 Community Consultation

3.9.1 General

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

Although some historic flood information could be sourced from the previous investigations, additional information on past flooding was sought from the community to assist with the 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.

3.9.2 Flood Study Website

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

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

During the course of the study (up to August 2016), the website was visited 1,801 times by 1,390 unique visitors.

3.9.3 Community Information Brochure and Questionnaire

A community information brochure and questionnaire was prepared and distributed to all residential and business properties in the catchment. This resulted in brochures and questionnaires being distributed to approximately 8,000 addresses. A copy of the brochure and questionnaire is included in **Appendix A**.

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 421 questionnaire responses were received. A summary of all questionnaire responses is provided in **Appendix A**. The spatial distribution of questionnaire respondents is shown in **Figure A1**, which is also enclosed in **Appendix A**.

The responses to the questionnaire indicate that:

- The majority of respondents have lived in or around the catchment for about 25 years. Accordingly, most respondents would have been living in the area during the 2012 flood (discussed in more detail below) but not necessarily the 1986 or 1988 events.
- 28% 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** and **Plate 5**):
  -> 62 respondents have experienced traffic disruptions;
  -> 82 respondents have had their front or back yard inundated;
  -> 37 respondents have had their garage inundated; and,
  -> 11 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 A1** in **Appendix A** (refer red dots).
- Flooding problems were reported in the following streets and areas in multiple questionnaire responses:
  -> Victoria Street
  -> John Oxley Avenue
  -> Chapman Gardens
  -> George Street
  -> Stafford Street
  -> Cosgrove Crescent
  -> Stapley Street
  -> Cox Avenue
- 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 a 2012 flood. A selection of these photographs are provided in **Appendix B**. Unfortunately, the flood occurred at night making it difficult to identify specific water depths in many of the photos. Nevertheless, several "post-flood" photos provide debris marks which indicates the height that water reached at the peak of the event.

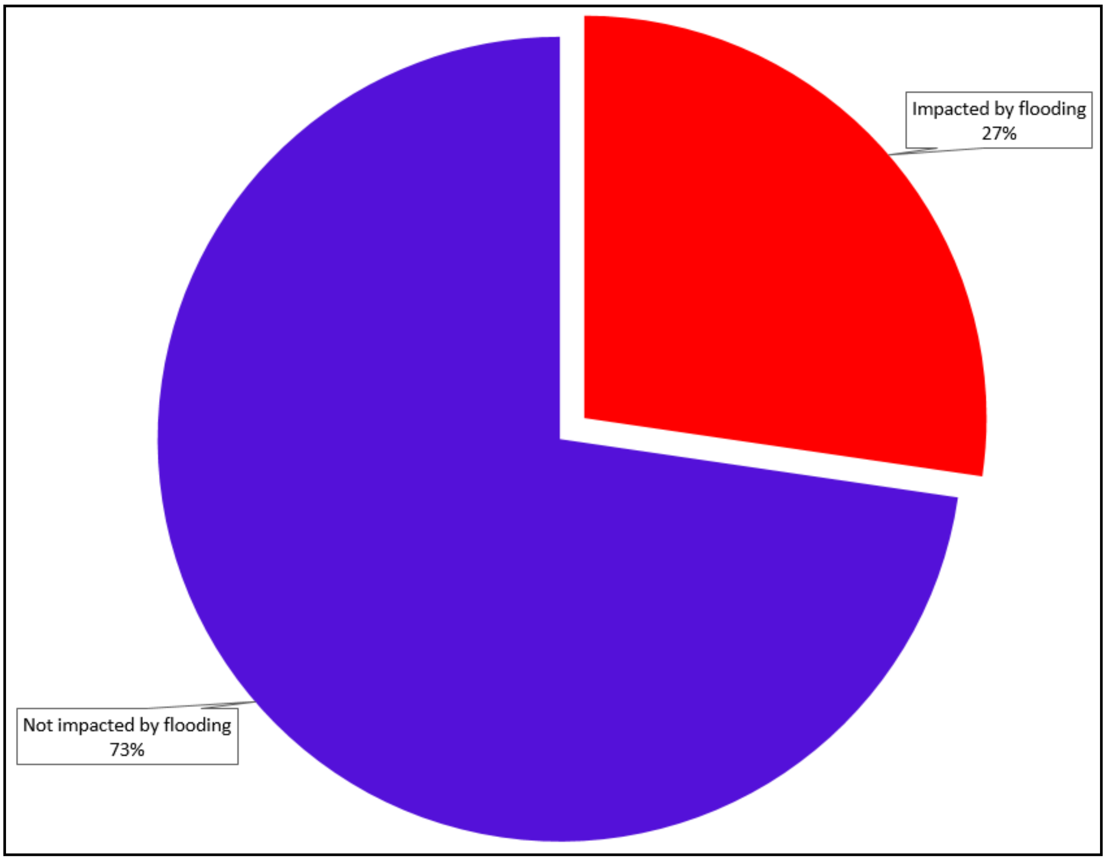


Plate 4 Number of Questionnaire Respondents Impacted by Past Floods

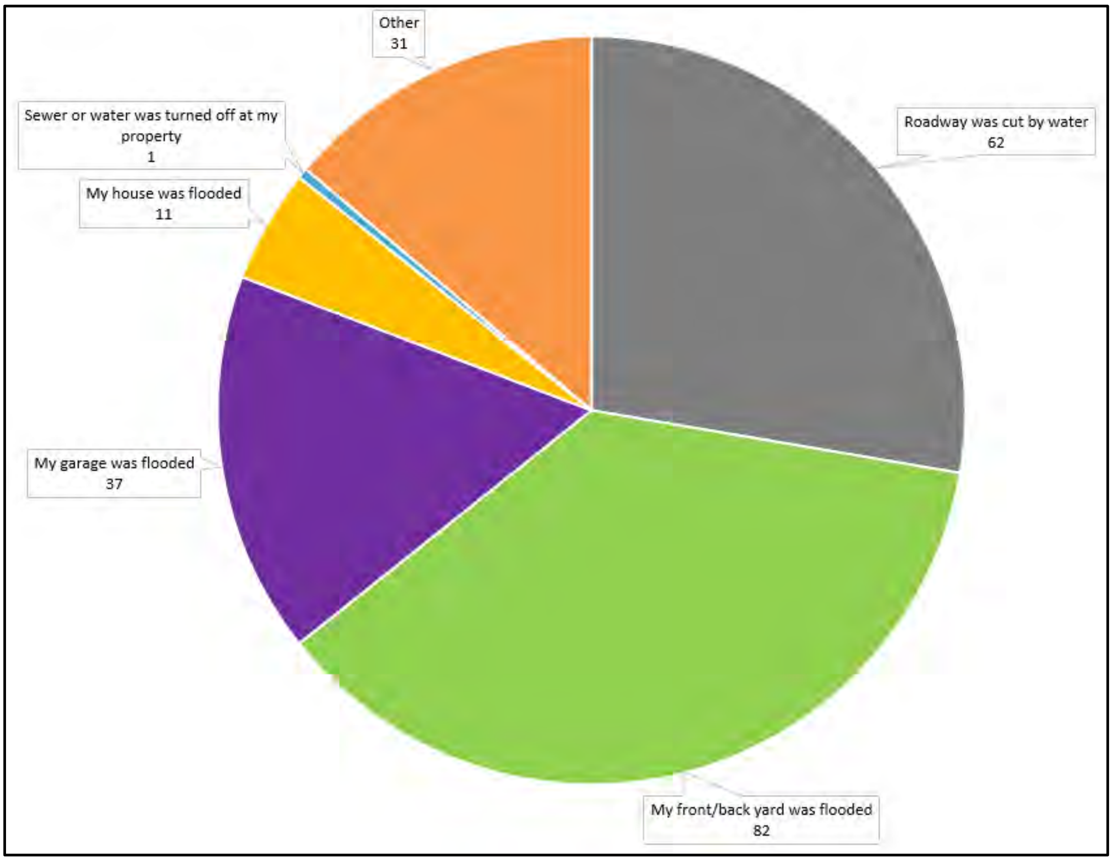


Plate 5 Type of Flood Impact Reported by Questionnaire Respondents

The photos show the 2012 event inundated a number of properties above floor level. This included residential and industrial properties as well as the Western Sydney University, Kingswood Campus. The photos also indicate that the depth and speed of floodwater damaged many fences.

4 COMPUTER FLOOD MODEL

4.1 General

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

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

The following sections describe the computer model development process.

4.2 Model Development

4.2.1 2D Model Extent and Grid Size

A 2-dimensional computer model of the College, Orth & Werrington Creeks catchment was developed using the TUFLOW software (version 2013-12-AE). The extent of the model area is shown in **Figure 7**. As shown in **Figure 7**, the TUFLOW model extends across the full extent of the catchment draining to the confluence of Werrington Creek and South Creek.

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 2002 ALS and 2011 LiDAR data as well as the design and work-as-executed Digital Elevation Models for the Caddens and French Street subdivisions (refer **Figure 4**).

4.2.2 Material Types

The TUFLOW software uses land use information to define the hydrologic (i.e., rainfall losses) and hydraulic (i.e., Manning's 'n') properties for each grid cell in the model. As discussed in Section 3.6, a remote sensing approach was employed to provide a detailed spatial description of the variation in land use types across the catchment (refer **Figure 5**).

This land use information was used to inform the specification of rainfall losses and Manning's "n" roughness coefficients, which is described in more detail below.

Rainfall Losses

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

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

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

The catchment includes extensive urban areas that are relatively impervious as well as areas of "open" space that are pervious. The impervious and pervious sections of the catchment respond differently from a hydrologic perspective, i.e.:

- rapid rainfall response and low rainfall losses across impervious areas; and,
- slower rainfall response and higher rainfall losses across pervious areas.

In recognition of the differing characteristics of the two hydrologic systems, the rainfall losses were varied spatially based on the different material types or land uses shown in **Figure 5**. The initial and continuing losses were applied to each material type based on design values documented in 'Australian Rainfall and Runoff – A Guide to Flood Estimation' (Engineers Australia, 1987) and are summarised in **Table 3**. As shown in **Table 3**, pervious areas were assigned an initial loss of 10 mm and a continuing loss rate of 2.5 mm/hr and impervious areas were assigned an initial loss of 1 mm and a continuing loss rate of 0 mm/hr. No losses were assumed across water as any rain falling on water will directly contribute runoff to that water body (i.e., no potential for interception or infiltration).

As shown in **Figure 5** and **Table 3**, the catchment includes some areas that are currently under construction. These areas will ultimately comprise a mix of pervious and impervious surfaces. However, as the future composition of these areas is not known, an assumption was made regarding the potential proportion of pervious and impervious surfaces. In this regard, it was assumed that each of these areas would comprise 25% pervious surfaces and 75% impervious surfaces. A weighted initial loss and continuing loss rate was subsequently calculated based upon this assumption and is included in **Table 3**.

Table 3 Rainfall Loss Values

Material Description	Initial Rainfall Loss *(mm)*	Continuing Rainfall Loss Rate *(mm/hr)*
Building	1.0	0.0
Water	0.0	0.0
Trees	10.0	2.5
Grass	10.0	2.5
Concrete / roadways	1.0	0.0
Areas currently under construction (assumed 100% developed)*	3.3	0.6

NOTE: *Weighted rainfall losses for "areas currently under construction" were estimated assuming that these areas would ultimately comprise 75% impervious and 25% pervious surfaces

Manning's "n" Roughness Coefficients

Manning's "n" is an empirically derived coefficient that is used to define the resistance to flow (i.e., roughness) afforded by different material types 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).

When using a "direct rainfall" computer model, the depth of flow across much of the catchment will be shallow (often referred to as "sheet flow"). In such instances, the depth of flow can be equal to or less than the height of the vegetation and the vegetation is not necessarily rigid (e.g., grass can bend under the force of flowing water). 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 C** 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	$Depth_3$ *(metres)*	n_3	$Depth_4$ *(metres)*	n_4
Building*	<0.01	0.025	>0.01	10.00	-	-	-	-
Water	0.035 for all depths	-	-	-	-	-	-	-
Trees	<0.30	0.133	0.50	0.078	>2.00	0.098	-	-
Grass	<0.03	0.107	0.05	0.077	0.07	0.052	>0.10	0.031
Concrete / roadways	<0.005	0.034	>0.005	0.015	-	-	-	-
Areas currently under construction	<0.005	0.054	0.03	0.039	0.05	0.033	0.07	0.028
Areas currently under construction *(continued)*	0.10	0.024	0.50	0.021	>2.00	0.022	-	-

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 water bodies as the initial water level in most of these water bodies would be well above the height of any vegetation (i.e., the "shallow depth" problem will not be as significant in water bodies).

For "areas currently under construction", weighted depth varying Manning's "n" values were calculated assuming these areas would be fully developed and each lot comprised 75% concrete or roadway, 20% grass and 5% trees.

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. However, a reduced "n" value was applied to shallow depths of inundation to reflect the relatively rapid runoff of water from the roof areas during the early stages of a rainfall event. Further information on the representation of buildings in the model is provided in Section 4.2.7.

4.2.3 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 geometry of each channel segment was defined using the surveyed cross-sections.

The location of channels that were included within the 1D domain is shown in **Figure 7**. The location of surveyed cross-sections is also provided on **Figure 7**.

4.2.4 Culverts and Bridges

Culverts and bridges can have a significant influence on flood behaviour. Therefore, all bridges and culverts were also represented within the 1D domain of the TUFLOW model. The location of culverts and bridges that were included within the TUFLOW model is shown in **Figure 7**.

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. An entrance loss coefficient of 0.5 and an exit loss coefficient 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 D**.

Culvert and Bridge Blockage

During a typical flood, sediment, vegetation and urban debris (e.g., litter, shopping trolleys, fences) from the catchment can become mobilised leading to blockage of downstream culverts and bridges (refer **Plate 6**). Consequently, bridges and culverts will typically not operate at full efficiency during most floods. This can increase the severity of flooding across areas located adjacent to these structures.

In recognition of this, blockage factors varying between 0% and 100% were applied to all bridges and culverts. The blockage factors were applied based on blockage guidelines contained in the Australian Rainfall & Runoff document titled 'Blockage of Hydraulic Structures' (Engineers Australia, 2015). This guideline requires an assessment of potential debris type, debris availability, debris mobility and debris transportability at each structure location. This assessment was completed using the land use information shown in **Figure 5** as well the LiDAR information. The outcomes of the blockage assessment are summarised in **Appendix F** for each culvert or bridge located within the catchment.

4.2.5 Other Hydraulic Structures

A range of other hydraulic structures are also scattered across the catchment. This includes trash racks and gross pollutant traps (GPT) and energy dissipation structures. Most of these structures required tailored approaches to represent the unique hydraulic characteristics of each. Further details on how each of these structures was represented in the TUFLOW model is provided in **Appendix G**.

4.2.6 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.

Plate 6 View showing debris accumulation upstream of Cosgrove Crescent culvert following February 2012 flood

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.

Survey of all stormwater pits and pipes was completed as part of the study. This survey information provided a detailed description of the key attributes of all stormwater pits and pipes allowing these stormwater components to be directly included in the TUFLOW model. The extent of the stormwater system included within the TUFLOW model is shown in **Figure 7**.

Once all stormwater pit types were defined across the catchment, 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
- Pit exit loss

Stormwater Blockage

There is also potential for blockage of stormwater inlets or pits to occur during storms (refer **Plate 7**). Accordingly, blockage factors were assigned to all stormwater pits to reflect the reduced inflow capacity that would occur with partial pit blockage.

Plate 7 View showing blockage of a stormwater pit

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.

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%

4.2.7 Building Representation

The College, Orth and Werrington 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.

A number of options are available to represent buildings within computer models. In order to evaluate which building representation option would be best suited to the catchment, a range of options were evaluated. The outcomes of this evaluation are summarised in **Appendix I**.

As outlined in **Appendix I**, the most appropriate building representation approach involves representing the lower part of each building (i.e., the area between the ground surface and the floor level) as a complete flow obstruction. This is shown conceptually in **Plate 8**.

Once the water level exceeded 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 8**.

4.2.8 Fences

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

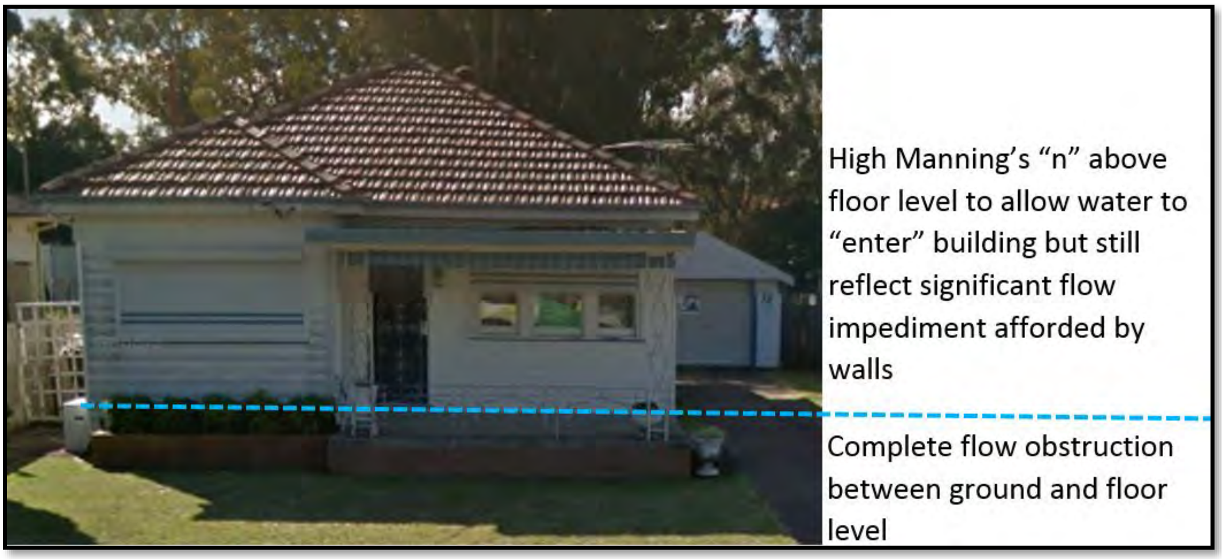


Plate 8 Conceptual representation of buildings in TUFLOW model

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

Fence Blockage

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

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

The large array of fence types and debris blockage potential means that there will be considerable variability in the overall blockage provided by different fence types. Although it can be difficult to quantify the variation in debris accumulation potential across the catchment, the types of fences are more readily identifiable. Therefore, fence types along major overland flow paths were identified as part of the study through field inspections and Google Street View. Specifically, a preliminary 1% AEP simulation was completed and all fences exposed to a water depth of greater than 0.15 metres were classified. This resulted in fences along major overland flow paths being classified according to one of six fence types. The extent of the different fence types delineated using this approach is shown in **Plate 9**.

Plate 9 shows that although there are a variety of fence types located along major flow paths, the most common fence type is Colorbond™.

Those fences located outside of major overland flow paths were delineated as "non-defined" fences. These fences are shown in yellow in **Plate 9** and were not explicitly categorised as part of the study.

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

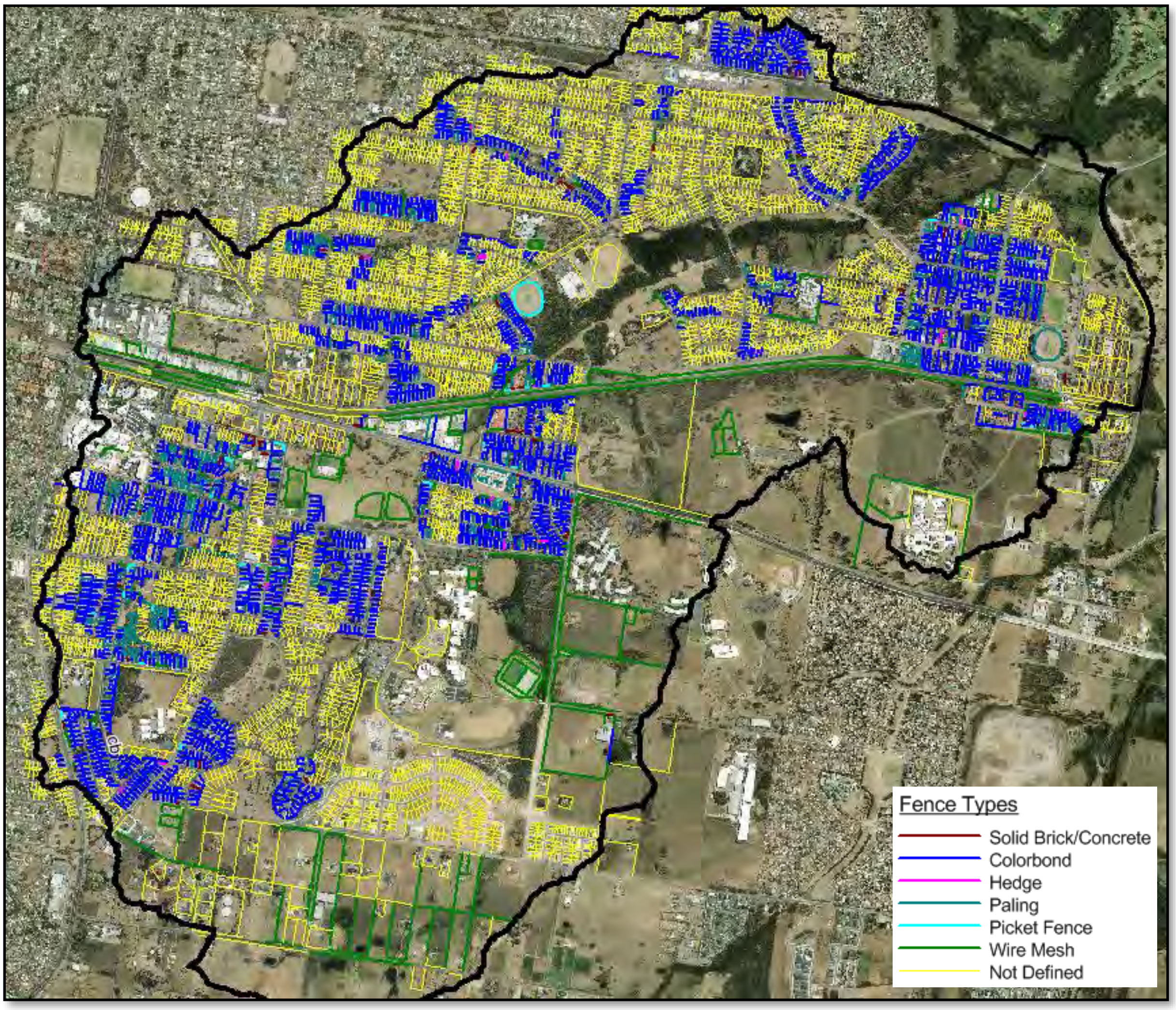


Plate 9 Fence alignments and types included in the TUFLOW model

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

Therefore, professional judgement was used as the basis for assigning representative blockage factors to each fence type. The adopted blockage factors are summarised in **Table 6**. These blockage factors are intended to account for the impediment to flow afforded by the fence itself as well as additional blockage associated with debris accumulation on the fence. As shown in **Table 6**, the adopted blockage factors fall within the 50% to 100% range suggested in the Project 11 document.

Although the height of each fence across the catchment varies, it was considered that most fence types will fail once the water depth and associated debris height exceeds 0.5 metres (refer **Plate 5**). Therefore, the blockage factors summarised in **Table 6** were applied for fences subject to water depths up to 0.5 metres. Any water exceeding a water depth of 0.5 metres was assumed to "overtop" the fence and no blockage was applied. The 0.5 metre fence height was considered to provide a reasonable estimate of the degree of blockage that can be provided by an average fence without failing.

Table 6 Adopted Fence Blockage Factors

Fence Type	Blockage Factor
Colorbond	95%
Solid Brick/Concrete	100%
Paling	80%
Wire Mesh	50%
Hedge	85%
Picket Fence	70%
Non-defined	50%

Plate 10 Collapsed fence adjoining Second Avenue, Kingswood showing debris mark approximately 0.5 m high (February 2012 flood)

4.2.9 Detention Basins and Water Storages

The College, Orth & Werrington Creeks catchment incorporates a number of detention basins, farm dams and water features (e.g., Lake Werrington) that may attenuate downstream flows during rainfall events. Therefore, a representation of each basin and dam 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 (e.g., Chapman Gardens).

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 and vegetation density 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 computer model. Simulated flows and flood levels are extracted from the model results at locations where recorded data are available. Calibration is completed by iteratively adjusting the model parameters within reasonable bounds to achieve the best possible match between simulated and recorded flood flows and flood levels.

Unfortunately, there are no stream gauges located within the study area. Moreover, there are no rainfall gauges located within the catchment. Therefore, it is not possible to complete a full calibration of the computer 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 model and comparing simulated floodwater depths against floodwater depths and flood photographs provided by the community.

A large number of anecdotal reports of flooding were provided by the community for the February 2012 flood. Accordingly, the validation of the computer model focussed on the February 2012 event. However, anecdotal reports of flooding were also provided for several other recent floods. This included the February 2010 and November 2011 events. Therefore, further validation of the computer model was completed using reported information on the 2010 and 2011 floods. Further information on outcomes of the computer model validation is provided below.

5.2 February 2012 Flood

5.2.1 Rainfall

The 2012 flood was produced by an intense downpour that occurred between 5:30pm and 8:30pm on the 9th February. During this period, around 50 mm of rain fell. This was proceeded by 15 mm of rain that fell between 3:00pm and 4:30pm. The significant amount of proceeding rainfall likely means that the catchment would have been saturated prior to the main downpour.

Accumulated daily rainfall totals for each rainfall gauge that was operational during the 2012 event were used to develop a rainfall isohyet map for the event, which is shown in **Figure 8**. The isohyet map shows that around 70 mm of rain fall across the catchment within a 24-hour period. The isohyet map was used as the basis for describing the spatial variation in rainfall in the TUFLOW model for the 2012 event

The temporal (i.e, time-varying) distribution of rainfall was applied based on the closest, active, continuous rainfall gauge. The closest continuous gauge was determined to be the Regentville Rural Fire Service gauge (Gauge #567163), which is located approximately 5 kilometres west of the College, Orth and Werrington Creeks Catchment. The location of the gauge is shown in **Figure 8** and the pluviograph for the gauge is presented in **Appendix J**.

The continuous rainfall information for Gauge #567163 was also analysed relative to design rainfall-intensity-duration information for the catchment. This information is presented in **Appendix J** as **Figure J4** and indicates that, based on the available rainfall records, the 2012 event was slightly more severe than a 5% AEP flood event.

5.2.2 Downstream Boundary Conditions

Hydraulic computer models also require the adoption of a suitable downstream boundary condition in order to reliably define flood behaviour throughout the area of interest. The downstream boundary 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 Werrington Creek and South Creek. Accordingly, the water level across the downstream reaches of the catchment will be driven by the prevailing water level along South Creek at the time of the flood.

Unfortunately, there are no stream gauges located in the vicinity of the Werrington Creek and South Creek confluence. Furthermore, none of the anecdotal flooding reports provide any information describing water levels along South Creek or the downstream reaches of Werrington Creek. Accordingly, no information is available to define a specific water level within South Creek at the time of 2012 flood.

Therefore, a “normal depth” (i.e., Manning’s) boundary condition was applied at the downstream boundary of the model. The normal depth boundary automatically calculates a water level based upon the amount of water travelling across the downstream model boundary and the characteristics of the channel at that location (i.e., channel geometry, slope and roughness).

As outlined above, no anecdotal reports of flooding are provided for the 2012 event along the lower reaches of Werrington Creek. Therefore, any uncertainties associated with the downstream boundary definition will not impact on the validation results.

5.2.3 Modifications to Represent Historic Conditions

Although the February 2012 flood occurred relatively recently, there have been some notable changes across the catchment since this flood occurred (e.g., Caddens subdivision).

Therefore, the TUFLOW model that was developed to represent "contemporary" catchment conditions was modified in an attempt to reflect catchment conditions at the time of the 2012 flood.

Google Earth™ was used to assist in identifying the extent of changes that have occurred across the catchment since 2012. More specifically, Google Earth™ was used to extract aerial imagery from 2012 as well as 2016, which is presented in **Plate 11**. As shown in **Plate 11**, only Stages 1 and 2 of the Caddens subdivision had commenced in 2012. The review of aerial imagery also showed that construction across the French Street subdivision had not commenced in 2012.

Therefore, the following updates were completed to the TUFLOW model to represent 2012 catchment conditions:

- Topography across French Street and Caddens was defined based upon the 2011 LiDAR. The design TIN for Stages 1 and 2 of Caddens was superimposed.
- Materials (and the associated Manning's "n" and rainfall losses) was defined based upon the remote sensing outputs from the 2011 LiDAR. Stages 1 and 2 of Caddens were implemented as "areas currently under construction".
- The "online" dam that was located along College Creek in 2012 was reinstated.
- Fences were removed across all of the Caddens area.
- The stormwater system was removed from the Caddens and French Street subdivisions with the exception of Stages 1 and 2 of Caddens

5.2.4 Antecedent Catchment Conditions

As discussed in Section 5.2.1, the main downpour for the 2012 event was preceded by 15 mm of rainfall. As a result, the catchment would likely have been "wet" before the main rainfall event. Therefore, no initial losses were applied across the catchment for the 2012 event to reflect the significant proceeding rainfall.

5.2.5 Structure Blockage

As discussed, there is potential for blockage of hydraulic structures to occur during a flood. 'Base' blockage factors for each bridge and culvert in the catchment were determined based upon recommendations in 'Blockage of Hydraulic Structures (Engineers Australia, 2015)' (refer **Appendix F**). However, this document notes that there appears to be a correlation between the size of a particular flood and the degree of blockage. More specifically, there is a higher potential for blockage during larger events where there is more runoff to mobilise debris.

Therefore, the document recommends adjusting the 'base' blockage factors down when simulating smaller floods (i.e., >5% AEP event) and adjusting the blockage factors up when simulating larger floods (i.e., <0.5% AEP). The 'base' as well as the adjusted blockage factors for each structure are summarised in **Appendix F**.

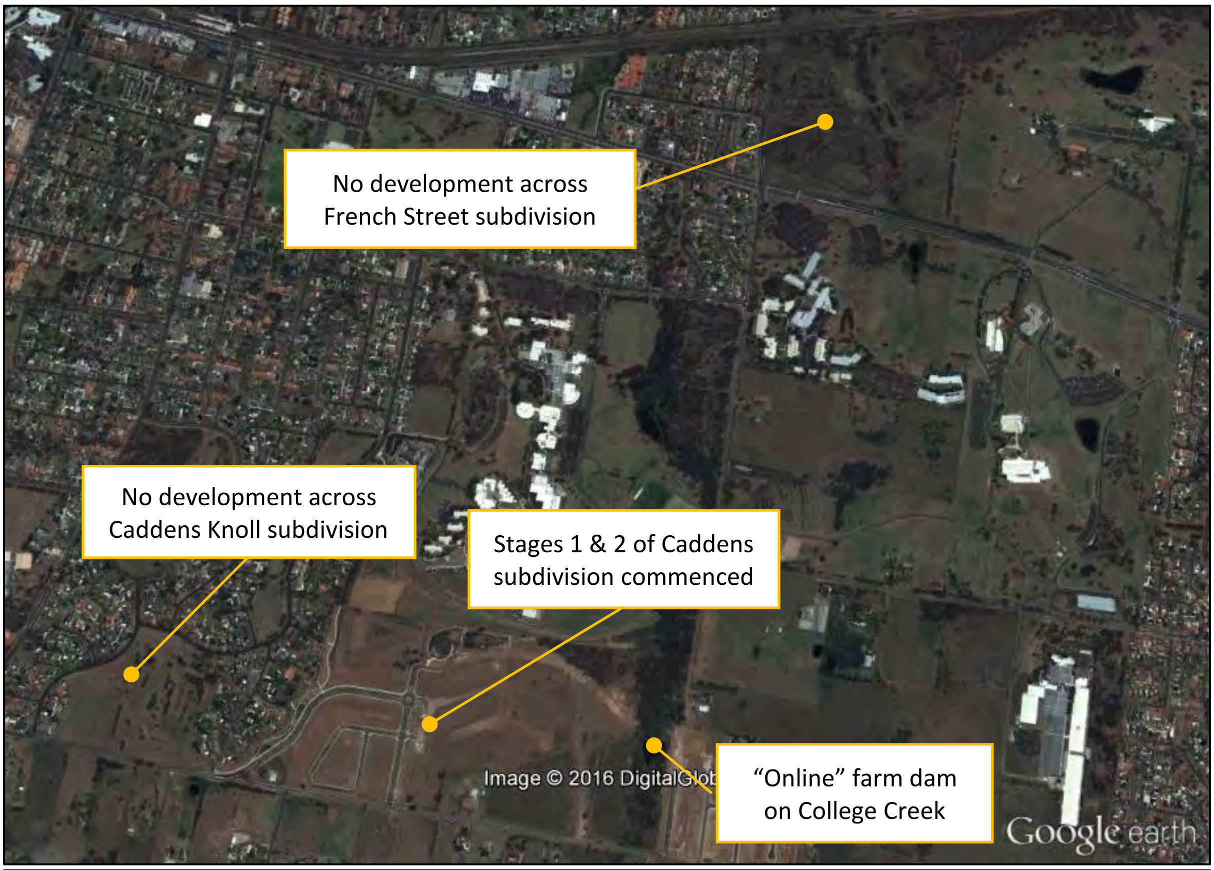


Plate 11 Comparison between 2012 (top image) and 2016 (bottom image) aerials showing extent of changes across Caddens (Google, 2016)

The rainfall during the 2012 event exceeded the 5% AEP design rainfall but was less severe than the design 0.5% AEP rainfall. Therefore, the '5% to 0.5% AEP' blockage factors were adopted. This equates to blockage factors of between 0% and 50%.

Blockage factors for stormwater pits were applied based upon Council's current blockage policy, which is summarised in **Table 5**.

5.2.6 Results

Validation of the TUFLOW computer model was attempted based upon twenty-five (25) anecdotal reports of flood behaviour for the 2012 event. In general, the anecdotal reports of flooding describe floodwater depths at discrete locations across the catchment.

The validation was undertaken by routing the historic rainfall described in Section 5.2.1 through the TUFLOW model and adjusting model parameter values until a reasonable agreement between simulated and reported and anecdotal floodwater depths was achieved.

Peak floodwater depths were extracted from the results of the 2012 flood simulation and are included on **Figure 9**.

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 9**. The flood depth comparison is also summarised in **Table 7**. The 'confidence level' that was reported by the community for each reported floodwater depth is also provided in **Table 7** 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.

As shown in **Table 7**, 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 7** shows that the TUFLOW model is generally reproducing the reported depths of inundation to within 100 mm. The only significant exception to this occurs for the flood mark reported by Response #20 where the difference exceeds 0.3 metres. However, it was noted that a specific flood depth location was not reported as part of the questionnaire responses. Therefore, there is some uncertainty associated with this flood mark.

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

Table 7 Comparison between simulated and observed floodwater depths for the 2012 flood

Response #	Description of Flood Behaviour	Reported Depth* (m)	Confidence Level#	Simulated Depth (m)	Difference (m)
3	Water entered through laundry door approximately 0.15m deep.	0.15	Low	0.13	-0.01
5	Water 0.3m deep across property	0.32	High	0.26	-0.05
13	Air conditioner damaged. Needed to evacuate to neighbouring property	0.18	High	0.17	0.10
20	A specific flood depth location was not reported. Assumed to be in sag point in roadway in front of property	1.00	Medium	0.51	-0.39
23	0.2m deep at fence line	0.20	Medium	0.18	-0.05
30	Water ankle deep around garage. Items in garage damaged	0.10	Medium	0.10	0.00
34	Water 0.3m deep under house	0.30	Medium	0.14	-0.11
78	3 inches of water under house	0.08	High	0.1	0.00
93	0.5m of water on fence and building walls	0.50	High	0.47	0.07
102	0.1m on brick fence	0.10	High	0.14	0.03
141	0.5m across frontage (water 4 steps from front door)	0.50	Low	0.46	-0.06
152	0.01m from the top of the front step (assume step height is 0.15 m)	0.14	High	0.11	-0.01
178	Up to 0.5m across part sections of property	0.50	Medium	0.41	0.02
179	Water overtopped Cosgrove Crescent driveway culverts by 0.25 metres.	0.25	High	0.20	-0.05
193	Driveway, fence and air conditioner flooded to depths of 0.15m. Car written off	0.15	-	0.15	-0.04
210	Up to the first verandah step. Assume 0.15m deep	0.15	-	0.11	-0.03
215	Water rose to 0.35m at back fence	0.35	High	0.24	0.02
241	Side and back fences damaged	1.50	Low	1.64	0.06
266	Numerous photos showing debris marks around 0.3 metres high across dealership	0.30	Medium	0.39	0.02
215	Water 0.44m deep at base of gutter in front of property	0.44	High	0.46	-0.02
414	0.3m in garage	0.30	-	0.23	-0.09
154	0.1m in front driveway	0.10	Medium	0.11	0.03
402	Water overtopped Cosgrove Crescent Pedestrian Bridge by 0.25m	0.25	High	0.24	-0.01

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

5.3 February 2010 Flood

5.3.1 Rainfall

The 2010 flood was generated by a short duration rainfall burst that started around 6:45pm on 5th February 2010. Over the next three quarters of an hour, over 45 mm of rain fell across the catchment.

Accumulated daily rainfall totals for each rainfall gauge that was operational during the 2010 event were used to develop a rainfall isohyet map for the event, which is shown in **Figure 10**. The isohyet map shows that between 54 and 70 mm of rain fell across the catchment within a 24-hour period. Due to the significant spatial variation in rainfall, the isohyet map shown in **Figure 10** was used as the basis for describing the spatial variation in rainfall in the TUFLOW model for the 2010 flood simulation

The temporal (i.e, time-varying) distribution of rainfall was applied based on the closest, active, continuous rainfall gauge. The closest continuous gauge was determined to be the Regentville Rural Fire Service gauge (Gauge #567163), which is located approximately 5 kilometres west of the College, Orth and Werrington Creeks Catchment. The location of the gauge is shown in **Figure 10** and the pluviograph for the gauge is presented in **Appendix J**.

The continuous rainfall information for Gauge #567163 was also analysed relative to design rainfall-intensity-duration information for the catchment. This information is presented in **Appendix J** as **Figure J4** and indicates that the 2010 rainfall intensity approached that of a 2% AEP design event.

5.3.2 Downstream Boundary Conditions

As with the 2012 flood simulation, no information describing peak water levels along the downstream reaches of Werrington Creek or South Creek was available for the 2010 simulations. Therefore, a "normal depth" boundary condition was applied to the downstream boundary of the TUFLOW model for the 2010 simulation.

5.3.3 Modifications to Represent Historic Conditions

Google Earth™ was used to assist in identifying the extent of changes that have occurred across the catchment since 2010. The outcomes of this assessment is presented in **Plate 12**.

As shown in **Plate 12**, development across the Caddens and French Street subdivision had not commenced in 2010. Therefore, the following updates were completed to represent 2010 catchment conditions:

- Topography across French Street and Caddens was defined based upon the 2011 LiDAR.
- Materials (and the associated Manning's "n" and rainfall losses) was defined based upon the remote sensing outputs from the 2011 LiDAR.
- The "online" dam that was located along College Creek was reinstated.
- Fences were removed across all of the Caddens area.
- The stormwater system was removed from the Caddens and French Street subdivisions.

Plate 12 Comparison between 2010 (top image) and 2016 (bottom image) aerials showing extent of changes across upper catchment areas (Google, 2016)

5.3.4 Antecedent Catchment Conditions

The rainfall hyetograph presented in **Figure J2** in **Appendix J** indicates that the main down pour during the 2010 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.

5.3.5 Structure Blockage

As noted in Section 5.3.1, the rainfall during the 2010 event is considered to be approximately equal to a 2% AEP event. Therefore, blockage factors for the '5% to 0.5% AEP' design range were adopted for the 2010 flood simulation based on the information contained in **Appendix F**. This equates to blockage factors of between 0% and 50%.

Blockage factors for stormwater pits were applied based upon Council's current blockage policy, which is summarised in **Table 5**.

5.3.6 Results

Validation of the TUFLOW computer model was attempted based upon seven (7) anecdotal reports of flood behaviour for the 2010 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 2010 flood simulation and are included on **Figure 11**. A comparison between the peak flood depths generated by the TUFLOW and the flood depths reported by the community for the 2010 flood is also provided in **Figure 11**. The flood depth comparison is also summarised in **Table 8**.

Table 8 Comparison between simulated and observed floodwater depths for the 2010 flood

Response #	Description of Flood Behaviour	Reported Depth* (m)	Confidence Level#	Simulated Depth (m)	Difference (m)
44	0.5m on Gascoigne Street near Nichols Place	0.50	Medium	0.38	0.12
45	0.3-0.4m on Burton Street (assume 0.35m deep)	0.35	Medium	0.32	0.03
49	No specific location information included. Preliminary model results show water "ponding" near garage	0.05	High	0.06	-0.01
93	0.5m of water on fence and building walls	0.50	High	0.49	0.01
186	Three inches of water near the corner of Daphne Close and Cosgrove Crescent	0.08	Medium	0.11	-0.03
198	Water extended 2m inside front fence line and washed away garden bed (assume at least 0.1m deep at front fence line)	0.10	Medium	0.11	-0.01
220	0.16m lapped at bottom of rear sliding glass door, water entered garage through rear door	0.16	High	0.15	0.01
296	30cm throughout the property and on Cox Avenue	0.30	High	0.32	0.02

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

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 0.12 metres of recorded depths and levels. The average difference between the simulated depths and recorded depths is 0.02 metres.

Therefore, it is considered that TUFLOW model is providing a good reported flood behaviour for the 2010 flood.

5.4 November 2011 Flood

5.4.1 Rainfall

The 2011 flood was generated by a short burst of rainfall that occurred on the morning of the 25th November. Approximately 15 mm of rain fell over a 45-minute period. The main rainfall event was also preceded by over 20 mm of rainfall during the previous 18 hours.

Accumulated daily rainfall totals for each rainfall gauge that was operational during the 2011 event are presented in **Figure 12**. The daily rainfall totals were also used to develop a rainfall isohyet maps for the 2011 event, which is also presented in **Figure 12**. The isohyet map shows that between 35 and 48 mm of rain fell across the catchment within a 24-hour period. Due to the significant spatial variation in rainfall, the isohyet map shown in **Figure 12** was used as the basis for describing the spatial variation in rainfall in the TUFLOW model for the 2011 flood simulation

The temporal (i.e, time-varying) distribution of rainfall was applied based on the closest, active, continuous rainfall gauge. The closest continuous gauge was determined to be the Regentville Rural Fire Service gauge (Gauge #567163), which is located 5 kilometres west of the College, Orth and Werrington Creeks Catchment. The location of the gauge is shown in **Figure 12** and the pluviograph for the gauge is presented in **Appendix I**.

The continuous rainfall information for Gauge #567163 was also analysed relative to design rainfall-intensity-duration information for the catchment. This information is presented in **Appendix J** as **Figure J4** and indicates that the 2010 rainfall intensity approached a 1 in 2 year ARI event. Accordingly, the 2011 event was significantly less severe relative to the 2010 and 2012 events.

5.4.2 Downstream Boundary Conditions

As with the 2010 and 2012 flood simulations, no information describing peak water levels along the downstream reaches of Werrington Creek or South Creek was available for the 2011 simulations. Therefore, a "normal depth" boundary condition was applied to the downstream boundary of the TUFLOW model for the 2011 simulation.

5.4.3 Modifications to Represent Historic Conditions

Google Earth™ was used to assist in identifying the extent of changes that have occurred across the catchment since 2011. The outcomes of this review determined that there were negligible changes in catchment conditions between 2010 and 2011. Therefore, the modifications that were completed to the TUFLOW model to represent 2010 catchment conditions (refer Section 5.3.3) were retained for the 2011 simulation.

5.4.4 Antecedent Catchment Conditions

The rainfall hyetograph presented in **Figure J3** in **Appendix J** indicates that the main down pour during the 2011 event was preceded by some significant rainfall (i.e., over 20 mm). As a result, the catchment would have been "wet" prior to the main rainfall event. Therefore, no initial losses were applied for the 2011 flood simulation.

5.4.5 Structure Blockage

As noted in Section 5.4.1, the rainfall during the 2011 is considered to be approximately equal to a 1 in 2 year ARI. Therefore, blockage factors for the '>5% AEP' design range were adopted for the 2011 flood simulation based on the blockage calculations included in **Appendix F**. This equates to blockage factors of between 0% and 25%.

Blockage factors for stormwater pits were applied based upon Council's current blockage policy, which is summarised in **Table 5**.

5.4.6 Results

Validation of the TUFLOW computer model was attempted based upon four (4) reports of flood behaviour for the 2011 event. The validation was undertaken by routing the historic rainfall described in Section 5.4.1 through the TUFLOW model and comparing reported and simulated flood levels at each location.

Peak floodwater depths were extracted from the results of the 2011 flood simulation and are included on **Figure 13**. A comparison between the peak flood depths generated by the TUFLOW and the flood depths reported by the community for the 2011 flood is also provided in **Figure 13**. The flood depth comparison is also summarised in **Table 9**.

Table 9 Comparison between simulated and observed floodwater depths for the 2010 flood

Response #	Description of Flood Behaviour	Reported Depth* (m)	Confidence Level#	Simulated Depth (m)	Difference (m)
81	0.3m on Chapman and Landers Streets	0.30	Medium	0.28	-0.02
93	0.5m on fence and building walls	0.50	High	0.37	-0.13
104	Water under building that had to be pumped out. Garden destroyed	0.30	Medium	0.23	-0.07
217	0.15 at the fence and garden brick border	0.15	High	0.11	-0.04

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

The flood level comparison provided in **Table 9** shows that the TUFLOW model generally provides a reasonable reproduction of recorded floodwater depths. In all cases the TUFLOW model provides depths and levels that are within 0.13 metres of recorded depths and levels. The average difference between the simulated levels and depths and recorded levels and depths is -0.06 metres.

Therefore, it is considered that the TUFLOW model is providing a reasonable reproduction of the 2011 event.

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 College, Orth and Werrington Creeks catchment are outlined in the following sections.

6.2 Computer Model Setup

6.2.1 Boundary Conditions

Design Rainfall

Design rainfall for the 1 in 2-year ARI as well as the 20%, 10% 5%, 2%, 1%, 0.5% and 0.2% AEP events were extracted using standard procedures outlined in *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987). This involved extracting base design intensity-frequency-duration values at the centroid of the College, Orth and Werrington Creek catchment from Volume 2 of *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987) (refer **Table 10**).

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

Table 10 Design IFD Parameters

Parameter	Value
${}^{2}I_{1}$	29.57
${}^{2}I_{12}$	6.62
${}^{2}I_{72}$	1.89
${}^{50}I_{1}$	59.27
${}^{50}I_{12}$	13.11
${}^{50}I_{72}$	4.45
F2	4.3
F50	15.8
Skew	0.03

Table 11 Design Rainfall Intensities

DURATION	1 in 2 Year ARI	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMP
5 mins	96.5	126	143	166	196	220	N/A	N/A	N/A
6 mins	90.2	118	134	156	184	206	N/A	N/A	N/A
10 mins	73.7	96.3	110	127	150	168	N/A	N/A	N/A
20 mins	53.6	69.8	79.4	92.0	109	121	N/A	N/A	N/A
30 mins	43.5	56.6	64.4	74.6	88.0	98.4	104	119	440
1 hour	29.5	38.4	43.7	50.6	59.7	66.8	70.6	82.3	320
1.5 hour	23.2	30.2	34.4	39.8	47.0	52.5	55.8	66.1	273
2 hours	19.6	25.4	28.9	33.5	39.5	44.1	47.0	56.1	240
3 hours	15.3	19.9	22.6	26.1	30.8	34.3	36.7	44.1	193
4.5 hours	12.0	15.5	17.6	20.3	23.9	26.7	N/A	N/A	N/A
6 hours	10.1	13.0	14.8	17.1	20.1	22.4	24.0	29.0	130
9 hours	7.86	10.2	11.5	13.3	15.7	17.5	N/A	N/A	N/A
12 hours	6.57	8.54	9.70	11.2	13.3	14.8	N/A	N/A	N/A
24 hours	4.20	5.56	6.38	7.44	8.87	9.97	N/A	N/A	N/A
48 hours	2.58	3.52	4.10	4.86	5.88	6.67	N/A	N/A	N/A
72 hours	1.89	2.62	3.08	3.68	4.48	5.11	N/A	N/A	N/A

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

For all design storms up to and including the 0.2% AEP event, the design rainfall was uniformly distributed across the entire study area. That is, there was no spatial variation in design rainfall across the study area.

The design rainfall estimates were used in conjunction with standard design temporal patterns documented in *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987) to describe how the design rainfall varies with respect to time throughout each design storm.

Probable Maximum Precipitation

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 computer 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 PMP estimates were varied spatially and temporally based on the GSDM approach before application to the XP-RAFTS and TUFLOW models. The GSDM PMP calculations are included in **Appendix K**. The PMP rainfall intensities are also summarised in **Table 11**.

South Creek Boundary Conditions

The College, Orth and Werrington Creeks catchment drains into South Creek north-east of Dunheved Road. Accordingly, the prevailing water level within South Creek can have a significant impact on flood behaviour along the downstream reaches of Werrington Creek. Therefore, it is important to define a reliable South Creek boundary condition as part of the design flood simulations.

To ensure consistency with other flood studies that have been completed across the Penrith City Council LGA, it was assumed that floods of equivalent severity were occurring across the College, Orth and Werrington Creeks catchment at the same time as across the broader South Creek catchment during all events up to and including the 5% AEP event. The 5% AEP flood was adopted for South Creek during all College, Orth and Werrington Creeks catchment events greater than the 5% AEP flood. A summary of the adopted local catchment and South Creek design flood combinations that were considered as part of the study are provided in **Table 12**.

Peak design water levels for South Creek were extracted from the *"Updated South Creek Flood Study"* (WorleyParsons, 2015) for all events equal to and greater than the 5% AEP flood. This information is reproduced in **Table 13**. However, design flood level information for South Creek was not available for the 1 in 2 year ARI, 20% AEP and 10% AEP events. Therefore, it was necessary to derive estimates of South Creek design water levels for these smaller design events.

Table 12 Adopted South Creek Downstream Boundary Conditions for Design Simulations

College, Orth & Werrington Creeks Catchment Design Flood	1 in 2 Yr ARI South Creek Flood	20% AEP South Creek Flood	10% AEP South Creek Flood	5% AEP South Creek Flood
1 in 2 year ARI	☑	-	-	-
20% AEP	-	☑	-	-
10% AEP	-	-	☑	-
5% AEP	-	-	-	☑
2% AEP	-	-	-	☑
1% AEP	-	-	-	☑
0.5% AEP	-	-	-	☑
0.2% AEP	-	-	-	☑
PMF	-	-	-	☑

Table 13 Peak South Creek Design Flood Levels at Werrington Creek Confluence

South Creek Design Flood	Peak South Creek Flood Level (mAHD)
1 in 2 year ARI	20.50*
20% AEP	20.89*
10% AEP	21.06*
5% AEP	21.29#
2% AEP	21.46#
1% AEP	21.64#
0.5% AEP	21.81#
0.2% AEP	22.05#
PMF	26.68#

NOTE: * Peak water level estimated by using the South Creek XP-RAFTS model in conjunction with a rating curve developed specifically for this study
Peak water level extracted from the "Updated South Creek Flood Study" (WorleyParsons, 2015)

In this regard, the XP-RAFTS model that was updated as part of the "Updated South Creek Flood Study" (WorleyParsons, 2015) was used to simulate each of smaller design rainfall events across the South Creek catchment and generate a peak design discharge at the Werrington Creek and South Creek confluence for each event. The peak discharge was subsequently converted to a peak water level using a rating curve that was developed from hydraulic model outputs also generated as part of the "Updated South Creek Flood Study" (WorleyParsons, 2015). The rating curve is provided in **Appendix E** and the resulting design water level estimates are summarised in **Table 13**.

It was also noted that a section of the College, Orth and Werrington Creeks catchment located west of Werrington Road and north of the railway line can drain into South Creek via a gated culvert that runs beneath Werrington Road. The outlet of this culvert is located approximately 1.7 km south of the Werrington Creek and South Creek confluence. Therefore, the prevailing water level at this culvert location would likely be different relative to the Werrington Creek and South Creek confluence and separate South Creek water levels would need to be defined as part of the design flood simulations. Therefore, design South Creek flood levels were also extracted at this location from the "Updated South Creek Flood Study" (WorleyParsons, 2015) or using the XP-RAFTS model and a rating curve to convert the peak discharges to a peak water level estimate. The resulting design water levels for South Creek at the Werrington Road culvert outlet is summarised in **Table 14**.

Table 14 Peak South Creek Design Flood Levels at Werrington Road Culvert

South Creek Design Flood	Peak South Creek Flood Level (mAHD)
1 in 2 year ARI	22.11*
20% AEP	22.56*
10% AEP	22.78*
5% AEP	23.06#
2% AEP	23.33#
1% AEP	23.57#
0.5% AEP	23.80#
0.2% AEP	24.06#
PMF	26.68#

NOTE: * Peak water level estimated by using the South Creek XP-RAFTS model in conjunction with a rating curve developed specifically for this study
Peak water level extracted from the "Updated South Creek Flood Study" (WorleyParsons, 2015)

6.2.2 Hydraulic Structure Blockage

Culvert and Bridge Blockage

As outlined in Section 4.2.4, 'base' blockage factors for each bridge and culvert were estimated based upon recommendations in 'Blockage of Hydraulic Structures' (Engineers Australia, 2015) (refer **Appendix F**). 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 – 1 in 2 year ARI, 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

Stormwater Blockage

'Blockage of Hydraulic Structures' (Engineers Australia, 2015) does not include any recommendations regarding design blockage factors for stormwater pits. Therefore, stormwater pit and grate blockage factors were assigned based upon Penrith City Council's blockage policy (refer Section 4.2.6).

6.2.3 Flood Gate Operation

The College, Orth and Werrington Creeks catchment incorporates two culverts that include flood gates (also referred to as flood "flaps"). The gated culverts are located at:

- Werrington Road culvert (located approximately 200 metres north of railway – refer **Plate 13**);

- Werrington earthen levee culvert (located approximately 100 metres north of Reid Street and 50 metres south of Dunheved Road).

Plate 13 Photograph showing partially submerged flood gate on downstream side of Werrington Road culvert

The flood gates are designed to be held 'closed' when there are elevated water levels at the downstream end of the culvert, thereby preventing the elevated water levels "backing up" the culvert and inundating the upstream catchment areas. When the water level on the downstream side of the culvert is lower than the upstream side of the culvert, there is sufficient hydraulic "head" to force open the gates and allow water to drain from the upstream catchment.

However, the flood gates can "malfunction" during floods and not operate as intended. For example, debris can become trapped preventing the gates from fully closing. A review of the catchments located upstream of each flood gate shows that:

- The Werrington Road culvert drains a small, partially urbanised catchment. The channel that drains runoff to the culvert is grassed-lined with negligible adjoining tree coverage. Therefore, the potential for blockage by large pieces of debris is considered to be small and it was assumed that this flood gate was fully operational (i.e., "closed") during the design flood simulations.
- The Werrington earthen levee culvert drains a much larger, urbanised catchment. The channel located immediately upstream (i.e., south) of the culvert includes significant vegetation and tree coverage. Although a trash rack is located at the upstream end of the channel which would help to prevent urban debris obstructing the flood gates, there is still an increased probability of vegetative debris being mobilised along the channel and obstructing the flood gate. Therefore, it was assumed that the Werrington earthen levee flood gate was not operational (i.e., was "open") during each of the design flood simulations.

The impact that alternate flood gate operation assumptions had on flood behaviour was assessed as part of the sensitivity analysis.

6.2.4 Topography

Ground surface elevations within the TUFLOW model have largely been defined based upon 2011 LiDAR supplemented with 2002 ALS. However, some areas within the catchment have undergone significant modifications since these topographic datasets were collected, most notably the Caddens and French Street subdivisions. Therefore, the LiDAR or ALS information does not provide a reliable representation of existing or potential future topographic conditions across these areas.

Therefore, it was necessary to supplement the LiDAR and ALS information with work-as-executed survey (where available) or design terrain information to ensure a reliable representation of the finished topography across these areas was provided. The extent of the areas where work-as-executed survey and design terrain information was used is shown in **Figure 4**.

It should be noted that the design terrain information may not reflect the final topography. Therefore, the results shown in these areas should be considered correct with the landform currently present and subject to further confirmation once each subdivision is finalised.

6.3 Results

6.3.1 Critical Duration

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

The information contained in **Plate 14** shows that the 2-hour storm duration produces the highest 1% AEP flood levels across the majority of the catchment. The 30-minute storm duration is also critical across some localised sections of the upper catchment.

Across the downstream reaches of the catchment, the 6 and 9-hour storm durations generated the highest 1% AEP flood levels. However, it was noted that the area where the 9-hour duration is critical is dominated by backwater inundation from South Creek. That is, South Creek water levels will generate the highest water levels across this sections of the catchment rather than the 9-hour storm duration. Accordingly, only the 30-minute, 2-hour and 6-hour durations were simulated for each design flood.

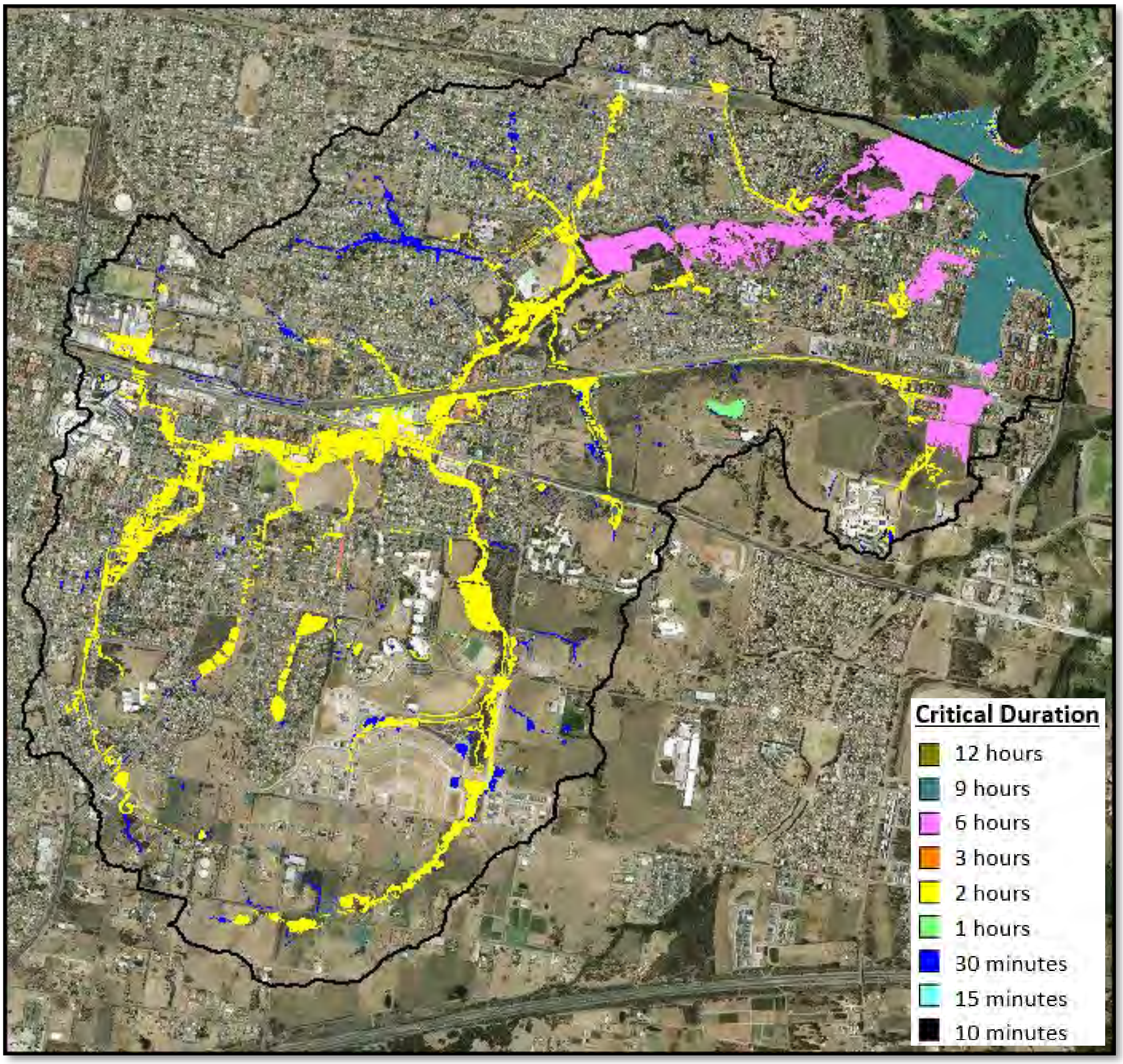


Plate 14 Spatial Variation in Critical Duration for the 1% AEP Storm

6.3.2 Design Flood Envelope

As discussed, a range of storm durations were simulated for each design flood to ensure the highest peak flood level was defined across all sections of the catchment. Consequently, a range of simulations were completed to ensure the worst case flood conditions were represented across all sections of the catchment for each design flood.

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

6.3.3 Presentation of Results

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

A minimum depth threshold of 0.15 metres has been adopted in other overland flood studies completed across the Penrith City Council. It was considered appropriate to retain this depth threshold as part of the current study for the following reasons:

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

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.

6.3.4 Ground Truthing of Preliminary Results

Preliminary floodwater depth maps were prepared for the 1% AEP flood based upon the depth and area filter criteria outlined above. 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.

The outcomes of the ground truthing is summarised in **Appendix L**.

6.3.5 Peak Depths, Levels and Velocities

The final floodwater depth mapping for the 1 in 2 year ARI as well as the 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP and PMF events are presented in **Figures 14** to **22** respectively. Peak flood levels and peak flow velocities were also extracted from the results of the design modelling for each flood and are presented in **Figures 23** to **31** and **Figures 32** to **39** respectively. It should be noted that floodwater depths across bridge and culvert crossings reflect the depth of water across the top of the roadway and bridge deck. Therefore, where no water depths are displayed across roadways, it indicates all flows are conveyed beneath the roadway and bridge deck. Where depths are displayed, it indicates that water is predicted to overtop roadway and bridge deck.

Peak flood levels and depths were also extracted at various locations across the catchment and are tabulated in **Appendix M**.

Peak design water levels were also extracted along each of the major creeks in the catchment and are presented as water surface profiles in **Figure 41**.

6.3.6 Peak Discharges

Plot Output (PO) lines were incorporated within the TUFLOW model to allow peak discharges to be extracted at various locations across the catchment for each design flood. The peak discharges that were extracted from the TUFLOW model results are summarised in **Appendix M**. It should be noted that the peak discharges include both piped flows as well as overland flows.

6.3.7 Stage Hydrographs

Stage hydrographs, describing the time variation in water level during each design flood, were extracted upstream of key roadway crossings and are presented in **Appendix N**. Key details of the hydraulic structure at each crossing such as culvert obvert and roadway elevations are also superimposed to help identify if a roadway may become submerged during a particular design flood and, if so, how much warning time may be available.

6.3.8 Inundated Properties

The number of properties inundated during each design flood was also determined. This information is summarised in **Table 15** (there are 5,896 properties contained within the College, Orth & Werrington Creeks catchment). The information presented in **Table 15** indicates that approximately 17% of properties located within the catchment will be at least partly inundated at the peak of the 1% AEP flood. This is predicted to increase to over 37% during the PMF. Accordingly, major flooding has the potential to impact a significant number of properties within the catchment.

Table 15 Number of Inundated Properties

Event	Number of Inundated Properties	Percentage of Total Number of Properties
1 in 2 Year ARI	559	9.5%
20% AEP	699	11.9%
10% AEP	769	13.1%
5% AEP	852	14.5%
2% AEP	938	15.9%
1% AEP	1038	17.5%
0.5% AEP	1548	26.3%
0.2% AEP	1698	28.8%
PMF	2186	37.1%

6.4 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.

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 42**. As shown in **Figure 42**, 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 42** 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 1 in 2 years ARI while other sections of the stormwater system are able to convey flows in excess of the 1% AEP event. In general, the major trunk drainage lines where flows are concentrated appear to have a lower capacity than the minor drainage lines. **Figure 42** also indicates that the pipe capacity rather than pit capacity appears to be the limiting factor in the performance of the stormwater system.

6.5 Results Verification

The TUFLOW model developed as part of this study was validated against recorded and observed flood information for three historic floods. In general, the model was found to provide a good reproduction of historic flood mark elevations. However, the outcomes of the calibration only 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 TUFLOW model was completed by comparing the results generated by the TUFLOW model against past studies as well as alternate computer modelling approaches.

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

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 College, Orth & Werrington Creeks catchment. This includes:

- Penrith Overland Flow Flood "Overview Study" (Cardno, 2006).
- Well Precinct Hydrology and Catchment Management Study (Cardno Willing, 2006).
- Werrington Subdivision, Cnr of French Street & Great Western Highway, Kingswood – Civil, Flooding and Stormwater Management Report (Cardno ITC, 2011)
- Updated South Creek Flood Study (WorleyParsons, 2015)

Hydrology

Peak 1% AEP discharges were extracted from the above reports and were compared against peak 1% AEP discharges produced by the TUFLOW model. The peak discharge comparison is provided in **Table 16**.

Table 16 Comparison between peak 1% AEP discharges generated by TUFLOW model and 1% AEP discharges documented in past studies.

Location	Updated South Creek Flood Study XP-RAFTS (Table B1) (m^3/s)	WELL Precinct XP-RAFTS (Table A4) (m^3/s)	TUFLOW (m^3/s)
Orth Crk @ Bringelly Rd	N/A	24.4	27.9
College Ck @ Caddens Rd	N/A	9.8	10.7
College Ck @ O'Connell St	N/A	22.9	17.6
College Ck @ Great Western Hwy	N/A	32.5	20.9
Park Ave Overland Flow Path	N/A	9.9	7.9
Werrington Ck @ Victoria Rd	N/A	79.2	66.1
Herbert St Overland Flow Path	N/A	11.2	13.7
Werrington Ck @ South Ck	167	126	98.4

The comparison presented in **Table 16** indicates the WELL Precinct and TUFLOW discharges generally agree to within 20%. Overall, the TUFLOW model produces peak 1% AEP discharges that are slightly lower than the peak discharges produced by the XP-RAFTS model developed for the WELL Precinct study. This is likely to be associated with the TUFLOW model including additional "micro" storage (e.g., small storages behind roadway embankments) that are not explicitly represented in the XP-RAFTS model. In addition, the XP-RAFTS model does not include a representation of the Caddens and French Street subdivisions and the associated stormwater infrastructure (including formal detention basins).

Both the Well Precinct and TUFLOW peak 1% AEP discharges for Werrington Creek at South Creek are considerably lower than the corresponding 1% AEP discharge extracted from the Updated South Creek Flood Study. This is likely to be associated with the Updated South Creek Flood Study XP-RAFTS model not including any of the detention basins or storages that are scattered across the Werrington Creek catchment, which would serve to attenuate downstream flows. It is considered that the more detailed TUFLOW and WELL Precinct XP-RAFTS models provide a better representation of the attenuation afforded by the various storages across the catchment.

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 at various locations across the catchment. The water level comparisons are provided in **Table 17** to **Table 20**.

The Penrith Overland Flow Flood "Overview Study" (Cardno, 2006) provides the most comprehensive flood level information across the catchment. The comparison between the peak 1% AEP flood levels documented in this study and the current study is presented in **Table 17**. The comparison indicates that both studies generate similar peak 1% AEP flood levels (i.e., levels generally agree too better than 0.2 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).

Some more significant differences in flood levels were observed at a handful of locations. This included:

- Cox Avenue (difference = -0.33 metres): The lower water level at this location is associated with the current study including twin 1.2 metre diameter stormwater pipes
- Orth Street (difference = -0.29 metres): The lower water level at this location is associated with the current study including twin 1.8 metre diameter stormwater pipes
- Chapmans Gardens (difference = 0.34 metres): The difference in water level at this location is associated with additional earthworks that were completed across Chapman Gardens since the Overview Study was completed. The earthworks involved construction of an embankment which results in elevated water levels across Chapman Gardens in the current model.
- Upstream of Werrington Creek crossing of Railway (difference = -0.99 metres): A decisive reason for this difference could not be established. However, the peak 1% AEP water level results documented in the Overview Study showed unusual "jumps" in water elevation in the vicinity of the railway line. This may be a localised anomaly in the overview modelling results as the water level results elsewhere in the area show a relatively close correlation.
- South of Werrington Creek Station (difference = -0.5 metres). This difference is considered to be associated with the current model including the culvert and pipe system draining beneath the railway line. The overview study model did not include this culvert, resulting in higher design water levels.
- Dunheved Road (difference = 0.86 metres). This difference is associated with different South Creek tailwater elevations being adopted in each study.

Table 18 provides a comparison between peak 1% AEP flood levels generated by the TUFLOW model developed for the current study and peak 1% AEP flood levels extracted from a waterRIDE WRB file that was produced as part of the Updated South Creek Flood Study (WorleyParsons, 2015). The WRB file was created from 1% AEP results generated by a RMA-2 hydrodynamic model of the South Creek catchment.

Table 17 Comparison between TUFLOW and Overland Overview Study 1% AEP water levels.

Location	Overview Study Flood Level (mAHD)	TUFLOW Flood Level (mAHD)	Difference (metres)
Piper Cl (near Tent St)	60.82	60.69	-0.12
Peppermint Reserve (near Yeelanna Pl)	55.00	55.08	0.08
Tent St (near Smith St)	54.36	54.29	-0.07
Sandringham Ave (near Dundee Street)	51.66	51.68	0.02
Cox Ave (near Phillip Street)	51.27	50.94	**-0.33**
Manning St Detention Basin	50.09	50.04	-0.05
Jamison Road (near Clemson St)	49.29	49.34	0.05
Somerset St (near Rodgers St)	46.95	47.05	0.10
Derby St (near Hargrave St)	45.04	44.92	-0.13
Orth St (near Bringelly Road)	43.37	43.08	**-0.29**
Edna St (near Manning St)	43.07	43.10	0.04
Western Sydney University Dam	43.05	43.05	0.00
Kingswood Sports Club	39.76	39.74	-0.02
College Creek (US Second Ave)	39.19	39.14	-0.05
Webley Ave (near Neeta Ave)	38.73	38.65	-0.08
Campton Ave (near Weatherby Ave)	37.42	37.46	0.03
Park Ave	37.29	37.21	-0.08
Orleton Pl	37.21	37.23	0.02
Chapman Gardens	37.07	37.41	**0.34**
French St Subdivision (US Railway)	36.71	36.65	-0.06
Cosgrove Cres Overflow Channel	36.68	36.73	0.05
Werrington Ck (US Railway)	35.69	34.70	**-0.99**
Lockyer Ave (near Fawkner Pl)	34.82	34.91	0.09
Herbert St (near William St)	30.91	30.92	0.01
Armstein Cres	27.55	27.47	-0.07
Lack Pl	27.54	27.50	-0.04
Prince St (near John Oxley Ave)	24.93	24.95	0.01
South of Werrington Station	24.28	23.78	**-0.50**
Werrington Ck (DS John Oxley Ave)	24.24	24.21	-0.03
Dunkley Place	22.02	21.97	-0.05
Werrington Ck (US Dunheved Rd)	20.80	21.66	**0.86**

Table 18 Comparison between TUFLOW and Updated South Creek Flood Study 1% AEP water levels.

Location	Updated South Creek Flood Study Water Level (mAHD)	TUFLOW Water Level (mAHD)	Difference (metres)
Werrington Ck / South Creek Confluence	21.65	21.64	-0.01
Werrington Ck @ Dunheved Rd	21.72	21.66	-0.06
Werrington Ck @ John Oxley Dr	24.69	24.32	-0.37
Werrington Ck @ Burton St	27.46	27.39	-0.07
Lake Werrington	28.59	27.78	**-0.81**

Table 18 shows that the current study produced peak flood levels that are comparable but lower than peak 1% AEP flood levels documents in the Updated South Creek Flood Study. This is likely associated with the current study including detailed creek survey, which better reflects the conveyance characteristics of the creek system.

A more significant difference was evident at Lake Werrington, where the current study produces a peak 1% AEP level that is 0.86 metres lower than Updated South Creek Flood Study. A review of the terrain information utilised in the Updated South Creek Flood Study hydraulic model does not appear to provide a realistic representation of the terrain across the lake. More specifically, the hydraulic model includes a channel that is "carved" through the lake as well as an unrealistic "ridge" of higher ground extending across a part section of the lake (refer **Plate 15**). The higher ground effectively holds back water within the lake and artificially increases water levels. Therefore, it is considered the Updated South Creek Flood Study hydraulic model is producing unrealistic model results across the lake.

Table 19 provides a comparison between peak 1% AEP flood levels generated by the TUFLOW model developed for the current study and peak 1% AEP flood levels generated by a MIKE-11 model developed for the WELL Precinct study (Cardno Willing, 2006). The comparison shows that the TUFLOW and MIKE-11 models produce comparable 1% AEP water level results at most locations. The only major water level difference occurs immediately upstream of the Werrington Creek crossing of the railway line. At this location, the TUFLOW model is predicting a 1% AEP water level that is 0.78 metres higher than the MIKE-11 model.

The outcomes of the model sensitivity analysis (refer Section 8) indicates that the railway crossing of Werrington Creek is potentially the most sensitive area within the catchment to variations in model input parameters. As a result, small changes in model inputs such as Manning's "n", culvert blockage or flows can produce a significant change in model results. It is considered that differences in model flows at the railway crossing are the most likely source for the water level difference. More specifically, the TUFLOW model predicts flows will "build up" behind the railway embankment near the French Street subdivision and travel west towards Werrington Creek (refer **Plate 16**). This 2-dimensional movement of water cannot be well represented in the 1-dimensional MIKE-11 model. As a result, the TUFLOW model is predicting higher flows entering Werrington Creek upstream of the railway resulting in higher 1% AEP water levels.

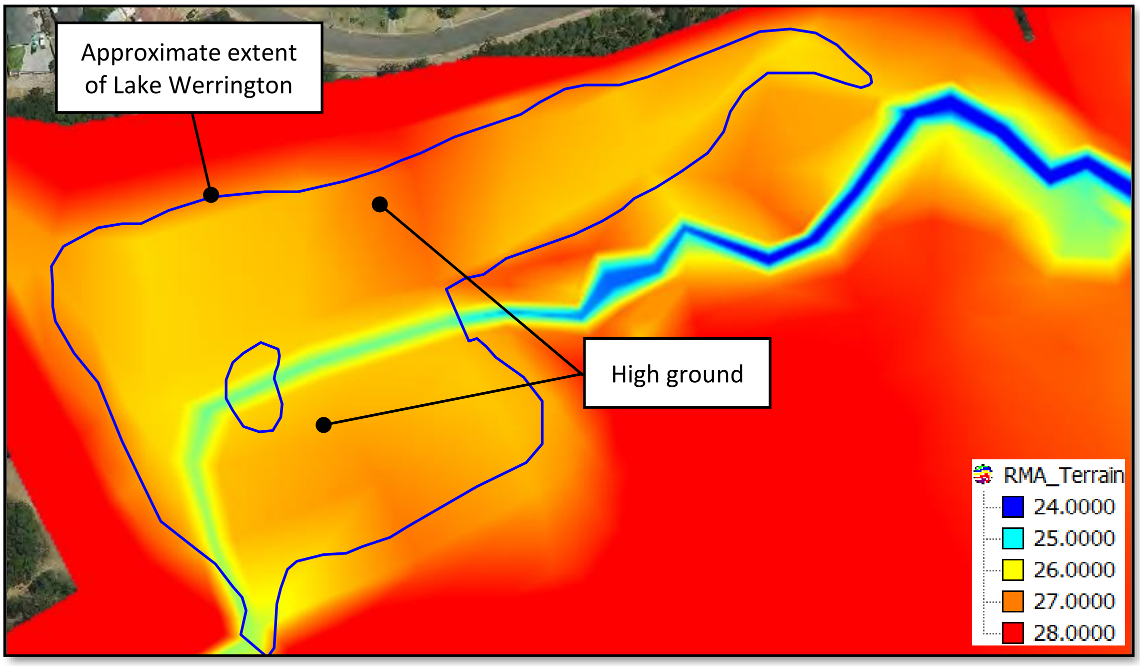


Plate 15 Updated South Creek Flood Study Terrain Representation across Lake Werrington

Table 19 Comparison between TUFLOW and WELL Precinct 1% AEP water levels.

Location	WELL Precinct Study		TUFLOW Water Level (mAHD)	Difference (metres)
	Mike-11 Cross-section	**Water Level** (mAHD)		
College Ck @ Second Ave	3.21	39.35	39.22	-0.13
College Ck @ Great Western Highway	3.71	36.53	36.41	-0.12
Werrington Ck @ Railway	4.00	33.91	34.69	**0.78**
Werrington Ck @ Victoria St	4.26	32.11	32.06	-0.05
Lake Werrington	5.12	27.47	27.78	0.31

Table 20 provides a comparison between peak 1% AEP flood levels generated by the TUFLOW model developed for the current study and peak 1% AEP flood levels generated by a HEC-RAS hydraulic model developed for the "Werrington Subdivision Flooding and Stormwater Management Report" (Cardno ITC, 2011). The comparison provided in **Table 20** shows a reasonable correlation at most locations. However, a more significant difference upstream of the Great Western Highway is noted.

A review of the TUFLOW and HEC-RAS model setup indicates that the differences in flood levels are associated with differences in the representation of the Great Western Highway culvert in both models. Both models share a similar upstream culvert invert elevation, however, the downstream culvert invert in the HEC-RAS model is ~0.5 metres lower than the TUFLOW model. Consequently, the HEC-RAS culvert has a slope in excess of 3% (compared with the TUFLOW culvert slope of 1.2%), resulting in supercritical flow through the culvert and lower water levels at the upstream end of the culvert. A review of the HEC-RAS cross-sections shows the culvert invert located approximately 0.2 metres below the invert of the downstream channel cross-section (refer **Plate 17**). Accordingly, the downstream culvert invert in the HEC-RAS model appears to be too low, which is resulting in artificially high flow velocities through the culvert and lower upstream water levels.

Table 20 Comparison between TUFLOW and French Street Subdivision 1% AEP water levels.

Location	French Street Study Water Level (mAHD)	TUFLOW Water Level (mAHD)	Difference (metres)
Upstream Great Western Highway	45.06	45.64	**0.58**
Upstream Internal roadway culvert	41.13	41.10	-0.03
Upstream Railway	36.80	36.61	-0.19

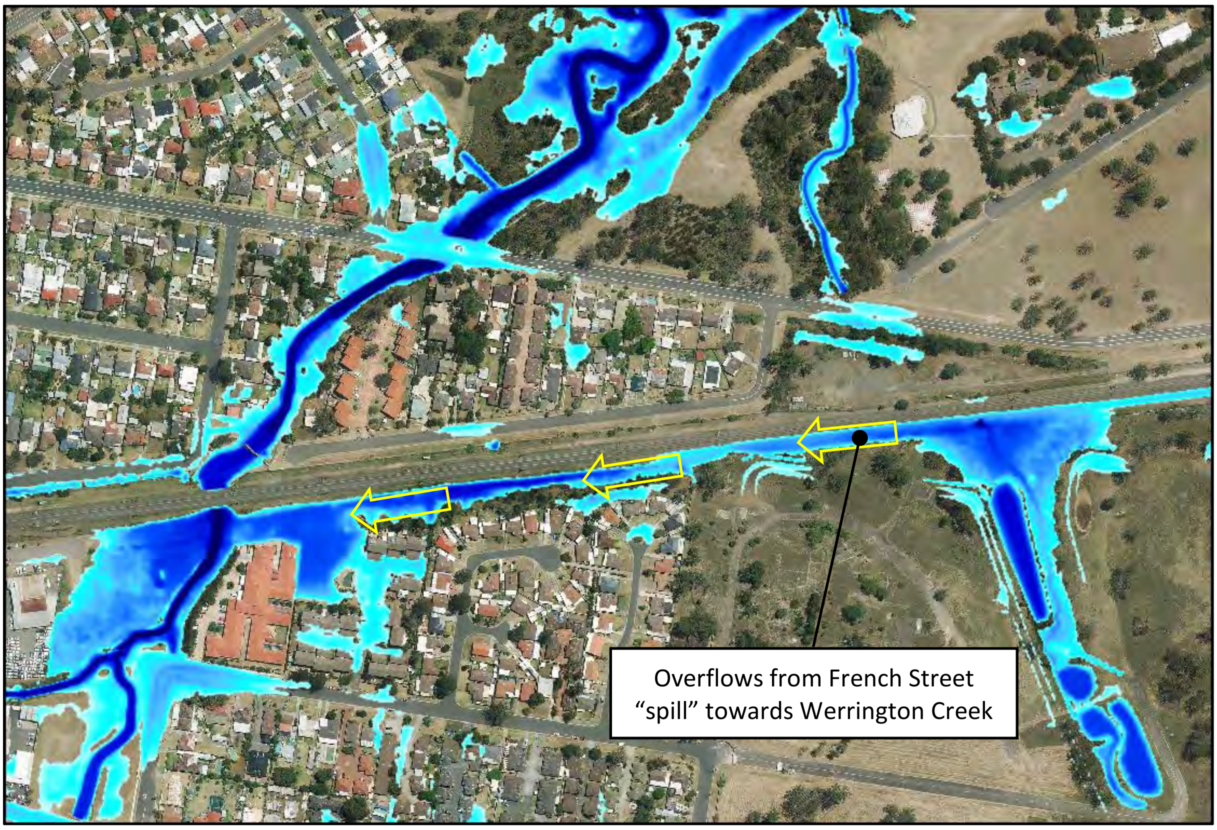


Plate 16 1% AEP floodwater depths showing contribution of flows to Werrington Creek from French Street subcatchment

6.5.2 Comparison with Other Modelling Approaches

XP-RAFTS Hydrologic Model

The ability of the TUFLOW model to represent rainfall-runoff processes was also verified against a hydrologic model of the College, Orth and Werrington Creeks catchment that was established specifically for the study using the XP-RAFTS software. Detailed information on the XP-RAFTS model setup is provided in **Appendix O**.

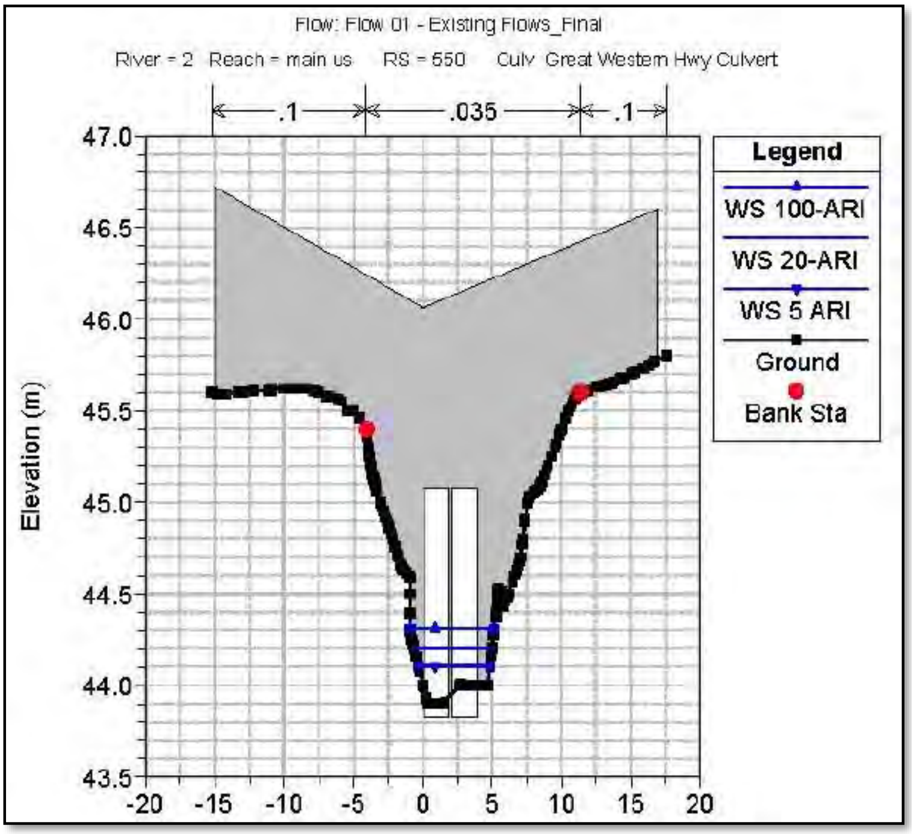


Plate 17 HEC-RAS representation of Great Western Highway showing culvert invert located below channel invert level.

The XP-RAFTS model was used to simulate the 1% AEP flood using the same hydrologic inputs as the TUFLOW model (i.e., design rainfall, rainfall losses, impervious proportion etc). Peak 1% AEP discharges were extracted from the XP-RAFTS model at key locations across the catchment for the 2-hour storm duration and are presented in **Appendix O**. The corresponding TUFLOW 1% AEP discharges at each location is also provided in **Appendix O** for comparison. The peak discharge comparison provided in **Appendix O** shows that TUFLOW model produces peak 1% AEP discharges that are within 20% of the XP-RAFTS model (in most cases the peak discharges agree to within 10%).

Full discharge hydrographs showing the time variation in flows at discrete locations throughout the catchment were also extracted from the XP-RAFTS and TUFLOW model results and are included in **Appendix O**. The hydrograph comparison shows that the overall hydrograph shapes, time of peak flow and volume of runoff (represented by the area under the hydrograph) generally compare well. It was noted that the TUFLOW hydrographs shows a greater delay before the hydrograph begins to rise relative to the XP-RAFTS model. This is likely to be associated with the TUFLOW model providing a more comprehensive representation of “micro” storage across the catchment (e.g., small depressions) that are difficult to represent in a lumped hydrologic model such as XP-RAFTS model.

Overall, the results of the verification indicate that the TUFLOW model is providing a reasonable reproduction of rainfall-runoff processes across the College, Orth and Werrington Creek catchment.

HEC-RAS Hydraulic Model

As noted in Section 6.5.1 there was some uncertainty associated with defining design flood levels immediately upstream of the Werrington Creek culvert crossing of the railway line. Therefore, it was considered important to ensure the culvert hydraulics at the railway crossing of Werrington Creek was being reliably represented in the TUFLOW model. Therefore, a separate HEC-RAS hydraulic computer model was developed to verify the TUFLOW model results across this section of Werrington Creek. Detailed information on the HEC-RAS model setup is provided in **Appendix P**.

The HEC-RAS model was used to simulate the 1% AEP flood. Input parameters were extracted from the TUFLOW model and applied to the HEC-RAS model. This included terrain information, Manning's "n" roughness coefficients as well as inflows. A comparison between the 1% AEP water level generated by the HEC-RAS model is provided in **Appendix P**. The comparison indicates that the HEC-RAS and TUFLOW models produce a peak 1% AEP water level upstream of the railway that agrees to within 0.07 metres. Accordingly, this indicates that the TUFLOW model is providing a reasonable reproduction of the hydraulics of the railway culvert.

Additional HEC-RAS model runs were also completed with a ±10% change in 1% AEP flows. The results of this sensitivity assessment are also contained in **Appendix P** and confirms that this area is sensitive to changes in flow values. More specifically, a 10% change in flow is predicted to alter peak 1% AEP flood levels by over 0.2 metres. This tends to confirm that this location is sensitive to changes in model input parameters and may help to explain the variation in modelling results documented in past studies across this area.

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 1.0% for any simulation
- 1D mass balance error did not exceed 0.3% for any simulations
- 2D mass balance error did not exceed 0.7% 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 Summary

The outcomes of the results verification presented in this section indicates that the TUFLOW model developed for this study is generally producing hydraulic and hydrologic results that compare favourably with past studies as well as alternate modelling techniques.

Some more significant differences were identified at isolated locations. However, most of these differences are likely associated with limitations with past models or differences in model input parameters. 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 College, Orth and Werrington Creeks catchment.

7 Flood Hazard and Hydraulic Categories

7.1 Flood Hazard

7.1.1 Overview

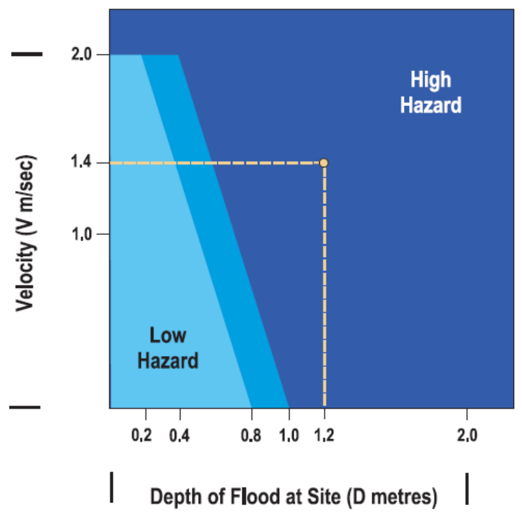
Flood hazard defines the potential impact that flooding will have on development and people across different sections of the floodplain.

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

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

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

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



Notes

The degree of hazard may be either -

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

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

Example:

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

FIGURE L2 - Provisional Hydraulic Hazard Categories

7.1.2 Provisional Flood Hazard

The TUFLOW hydraulic software was used to automatically calculate the variation in provisional flood hazard across the catchment based on the criteria shown in Figure L2 for the 5% AEP and 1% AEP floods as well as the PMF. These hazard category maps are shown in **Figures 43**, **44** and **45**.

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

7.1.3 Flood Emergency Response Classifications

The provisional hazard mapping presented in **Figures 43**, **44** and **45** can provide an indication of the risk to life and property across different sections of the catchment based on the depth and the velocity of floodwaters. Those areas subject to a low flood hazard can, if necessary, be evacuated by trucks and able-bodied adults would have little difficulty wading to safety (NOTE: evacuation by car may not be possible). Those areas of the floodplain exposed to a high flood hazard would have difficulty evacuating by trucks, there is potential for structural damage to buildings and there is possible danger to personal safety (i.e., evacuation by wading may not be possible).

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

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

The guideline provides a basis for the categorisation of floodplain communities into various Emergency Response Planning (ERP) classifications. The ERP classifications are summarised in **Plate 18** and can be used to provide an indication of the type of emergency response required.

Each allotment within the catchment was classified based upon the flow chart provided in the ERP guideline for the 5% AEP, 1% AEP and PMF (refer **Plate 18**). This was completed in an automated fashion using proprietary software based upon consideration of:

- whether evacuation routes/roadways get "cut" (a 150mm 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 50 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).

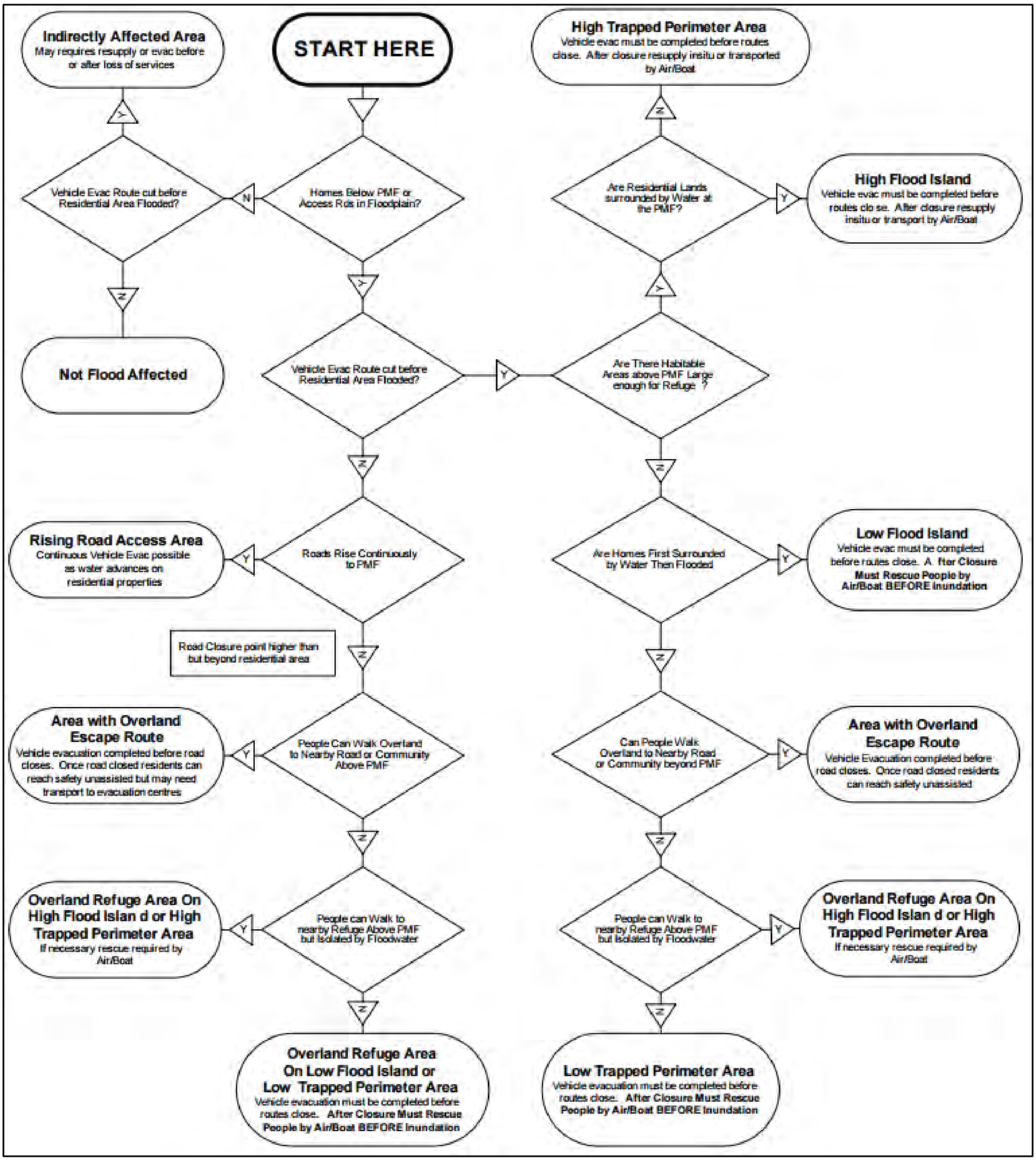


Plate 18 Flood Emergency Response Classification Flow Chart (Department of Environment & Climate Change, 2007)

The resulting ERP classifications for each design flood are provided in **Figures 46**, **47** and **48**. 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 information is also presented in **Figures 46**, **47** and **48**.

Discussions with the SES during the course of the study indicates that a new national emergency response classification guideline had been published (Australian Emergency Management Institute, 2014). However, the new classifications were yet to be widely adopted by the SES across New South Wales. Nevertheless, the SES may move towards this new classifications system in the future. Therefore, the new emergency response classifications were also prepared for the 5% AEP and 1% AEP floods as well as the PMF and these are presented in **Appendix Q** to assist with future emergency response planning.

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

7.1.4 Preliminary True Flood Hazard

The provisional hazard mapping presented in **Figures 43**, **44** and **45** was used in conjunction with the ERP classifications to prepare preliminary true hazard categories for the College, Orth and Werrington Creeks catchment. The preliminary true hazard categories reflect consideration of the depth and velocity of floodwaters as well as other emergency response factors that influence flood hazard, including the potential for isolation and evacuation difficulties.

In general, the provisional hazard categories were retained in the preliminary true hazard mapping. However, the provisional flood hazard was changed from low hazard to a high hazard when subject to the following ERP classifications (due to the flood liability of the land in conjunction with potential evacuation difficulties):

- Low Flood Island;
- Low Trapped Perimeter Area;

The preliminary true hazard mapping for the 5% AEP and 1% AEP floods as well as the PMF is presented in **Figures 49**, **50** and **51**.

It should be noted that the true hazard categories provided in **Figures 49**, **50** and **51** are preliminary and will be finalised during the subsequent Floodplain Risk Management Study for the catchment.

7.2 Hydraulic Categories

7.2.1 Overview

The NSW Government's *'Floodplain Development Manual'* (NSW Government, 2005) also characterises flood prone areas according to the hydraulic categories presented in **Table 21**. The hydraulic categories provide an indication of the potential for development across different sections of the floodplain to impact on existing flood behaviour and highlights areas that should be retained for the conveyance of floodwaters.

7.2.2 Adopted Hydraulic Categories

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

Table 21 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

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 21** 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.

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

The resulting hydraulic category maps for the 5% AEP and 1% AEP flood as well as the PMF are shown in **Figures 52**, **53** and **54**.

8 SENSITIVITY AND CLIMATE CHANGE ANALYSIS

8.1 General

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 6.5, the TUFLOW model was validated against recorded and observed flood information for three historic events and was further verified against alternate calculation approaches and results documented in past studies. In general, this information confirmed that the model was 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 Loss / Antecedent Conditions

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

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

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

The TUFLOW model was used to re-simulate the 1% AEP event with the modified initial losses. Peak water levels were extracted from the results of the modelling and were compared against peak water flood levels for "base" design conditions. This allowed water level difference mapping to be prepared showing the magnitude of any change in water levels associated with the change in initial loss values. The difference mapping is presented in **Plate 19** and **Plate 20** for the “dry” and “wet” catchment scenarios respectively. Decreases in 1% AEP “design” flood levels are shown in shades of blue and increases in 1% AEP flood levels are shown in shades of yellow and red.

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 22**. As shown in **Table 22**, the flood level differences are reported as a series of percentiles. For example, the lower initial 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.

Peak 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented in **Table 23**.

Table 22 Percentile Change in 1% AEP Flood Levels Associated with Changes to TUFLOW Model Input Parameters

Sensitivity Analysis	1st	5th	10th	25th	50th	75%	90th	95th	99th
Lower Initial Rainfall Loses (Wet catchment)	0.00	0.00	0.00	0.00	0.02	0.03	0.06	0.09	0.21
Higher Initial Rainfall Loses (Dry catchment)	-0.19	-0.11	-0.06	-0.04	-0.02	-0.01	0.00	0.00	0.00
Lower Continuing Loss Rates	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
Higher Continuing Loss Rates	-0.04	-0.02	-0.01	-0.01	0.00	0.00	0.00	0.00	0.00
Manning’s “n” reduced by 20%	-0.04	-0.02	-0.02	-0.01	0.00	0.01	0.02	0.03	0.07
Manning’s “n” increased by 20%	-0.05	-0.02	-0.01	-0.01	0.00	0.01	0.02	0.02	0.04
No Blockage of Hydraulic Structures	-0.08	-0.04	-0.01	0.00	0.00	0.01	0.02	0.02	0.06
Complete Blockage of Hydraulic Structures	-0.30	-0.09	-0.01	0.00	0.04	0.12	0.22	0.45	1.39
Lower South Creek Water level	-0.95	-0.78	-0.74	-0.07	0.00	0.00	0.00	0.00	0.00
Higher South Creek Water level	0.00	0.00	0.00	0.00	0.00	0.36	0.83	0.88	1.02
All Flood Gates Operational	-0.92	-0.54	-0.26	0.00	0.00	0.00	0.00	0.00	0.00
All Flood Gates Not Operational	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.02
New Design Rainfall Data	-0.13	-0.08	-0.05	-0.03	-0.02	-0.01	0.00	0.00	0.00

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 less than ±0.02 metres.

The most significant differences tend to be concentrated in the immediate vicinity of the railway line. More specifically, localised differences of over 0.2 metres are predicted at the Werrington Creek culvert crossing of the railway and immediately south of Werrington Railway station.

Table 23 Peak 1% AEP Flood Levels from Sensitivity Simulation at Various Location across the Catchment

	Location (refer to Plates 19 to 32 for locations)	"Base" Case (mAHD)	Lower Initial Loses (mAHD)	Higher Initial Loses (mAHD)	Lower Continuing Loses (mAHD)	Higher Continuing Loses (mAHD)	Lower Manning's "n" (mAHD)	Higher Manning's "n" (mAHD)	No Blockage (mAHD)	Complete Blockage (mAHD)	Lower South Creek Level (mAHD)	Higher South Creek Level (mAHD)	All Flood Gates Operational (mAHD)	All Flood Gates Not Operational (mAHD)	New Design Rainfall (mAHD)
1	Smith Street, Kingswood	54.04	54.05	54.03	54.04	54.04	54.05	54.04	54.04	54.11	54.04	54.04	54.04	54.04	54.03
2	Jamison Rd, Kingswood	49.25	49.25	49.23	49.25	49.24	49.28	49.24	49.23	49.27	49.24	49.25	49.24	49.25	49.23
3	Stafford St, Kingswood	46.79	46.81	46.77	46.79	46.79	46.81	46.79	46.72	46.87	46.79	46.79	46.79	46.79	46.77
4	Bringelly Rd, Kingswood	42.98	43.01	42.93	42.98	42.97	42.98	42.97	42.98	43.07	42.98	42.98	42.98	42.98	42.94
5	Orth St, Kingswood	45.40	45.41	45.39	45.40	45.40	45.39	45.41	45.42	45.33	45.40	45.40	45.40	45.40	45.39
6	Cox Ave, Kingswood	50.94	50.97	50.89	50.94	50.93	50.94	50.94	50.88	51.38	50.94	50.94	50.94	50.94	50.90
7	Chapman Gardens	37.59	37.60	37.58	37.59	37.59	37.59	37.59	37.58	37.62	37.59	37.59	37.59	37.59	37.59
8	Edna St, Kingswood	43.10	43.11	43.09	43.10	43.10	43.09	43.11	43.10	43.15	43.10	43.10	43.10	43.10	43.09
9	O'Connell St, Kingswood	45.99	46.04	45.93	46.00	45.98	45.98	46.00	45.69	46.13	45.99	45.99	45.99	45.99	45.95
10	Second Ave, Kingswood	39.07	39.18	38.92	39.09	39.05	39.05	39.10	38.92	39.45	39.07	39.07	39.07	39.07	38.99
11	Great Western Hwy	36.38	36.44	36.26	36.39	36.36	36.38	36.37	36.39	36.65	36.38	36.38	36.38	36.38	36.31
12	Railway (Werrington Ck)	34.57	34.81	34.29	34.59	34.52	34.65	34.51	34.63	36.41	34.57	34.57	34.57	34.57	34.35
13	Victoria St, Kingswood	32.00	32.08	31.91	32.02	31.99	32.00	32.01	31.95	32.44	32.01	32.01	32.01	32.00	31.93
14	Wrench St, Cambridge Park	39.45	39.45	39.44	39.45	39.44	39.43	39.46	39.44	39.50	39.45	39.45	39.45	39.45	39.43
15	Railway (French St)	36.55	36.63	36.39	36.56	36.53	36.52	36.57	36.58	36.73	36.54	36.55	36.54	36.54	36.47
16	Wembley Ave, Cambridge Park	38.67	38.65	38.64	38.65	38.65	38.64	38.66	38.65	38.73	38.65	38.65	38.65	38.67	38.64
17	Orleton Pl, Cambridge Park	37.23	37.24	37.22	37.24	37.23	37.22	37.24	37.23	37.29	37.23	37.23	37.23	37.23	37.22
18	Glencoe Ave, Cambridge Park	33.55	33.56	33.52	33.55	33.54	33.54	33.55	33.55	33.62	33.55	33.55	33.55	33.55	33.53
19	Burton St, Werrington	27.43	27.48	27.38	27.44	27.42	27.44	27.42	27.45	27.94	27.43	27.43	27.43	27.43	27.39
20	John Oxley Ave, Werrington	24.27	24.33	24.20	24.28	24.25	24.29	24.25	24.28	24.92	24.27	24.27	24.27	24.27	24.21
21	Lockyer Ave, Werrington Cty	34.81	34.91	34.89	34.90	34.90	34.89	34.91	34.90	34.98	34.90	34.90	34.90	34.81	34.89
22	Werrington Levee, Werrington	21.47	21.53	21.49	21.51	21.50	21.53	21.50	21.52	21.64	20.77	22.34	21.29	21.49	21.49
23	Dunkley Pl, Werrington	21.87	21.89	21.87	21.88	21.88	21.87	21.96	21.88	22.08	21.88	22.27	21.91	21.87	21.88
24	Walker Pl, Werrington	23.78	23.91	23.56	23.80	23.77	23.76	23.80	23.71	24.41	23.78	23.84	23.77	23.78	23.68
25	Dunheved Rd, Werrington	21.32	21.33	21.32	21.32	21.32	21.32	21.32	21.32	21.50	20.58	22.08	21.32	21.32	21.32

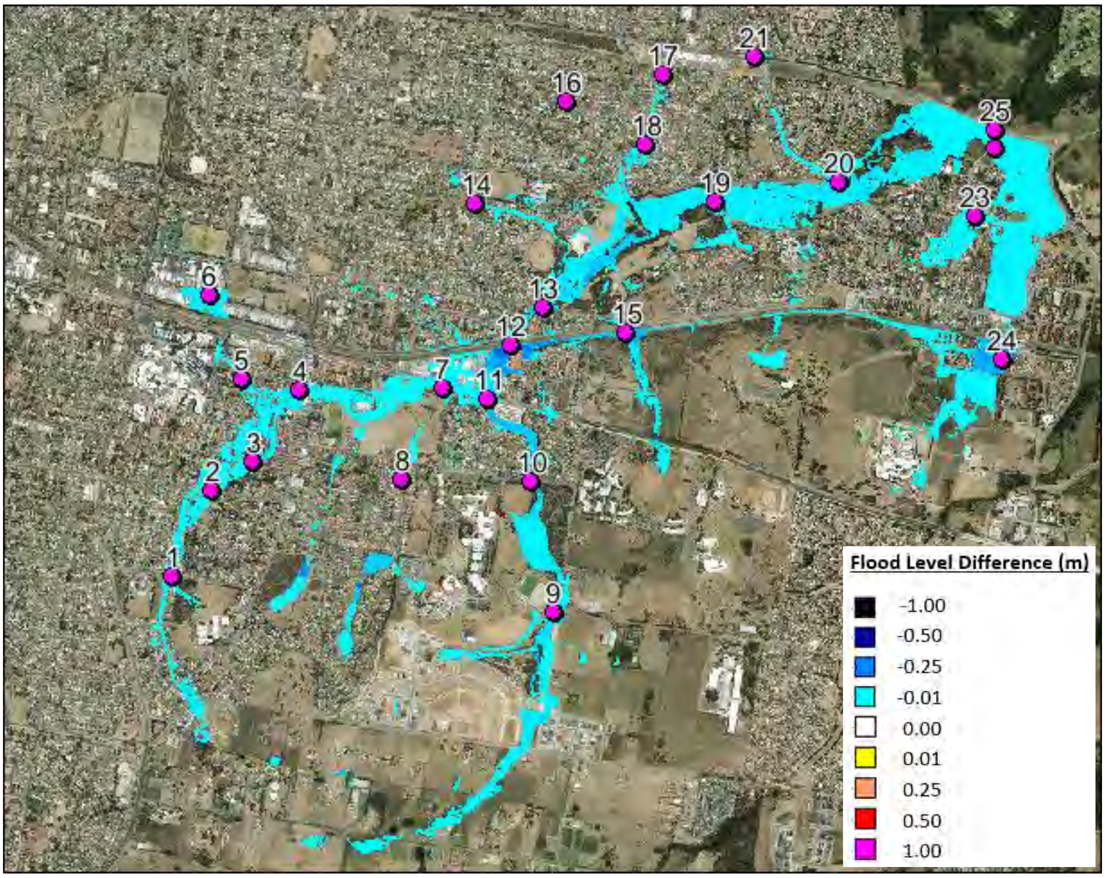


Plate 19 Flood level difference map with higher initial rainfall losses (i.e., dry catchment)

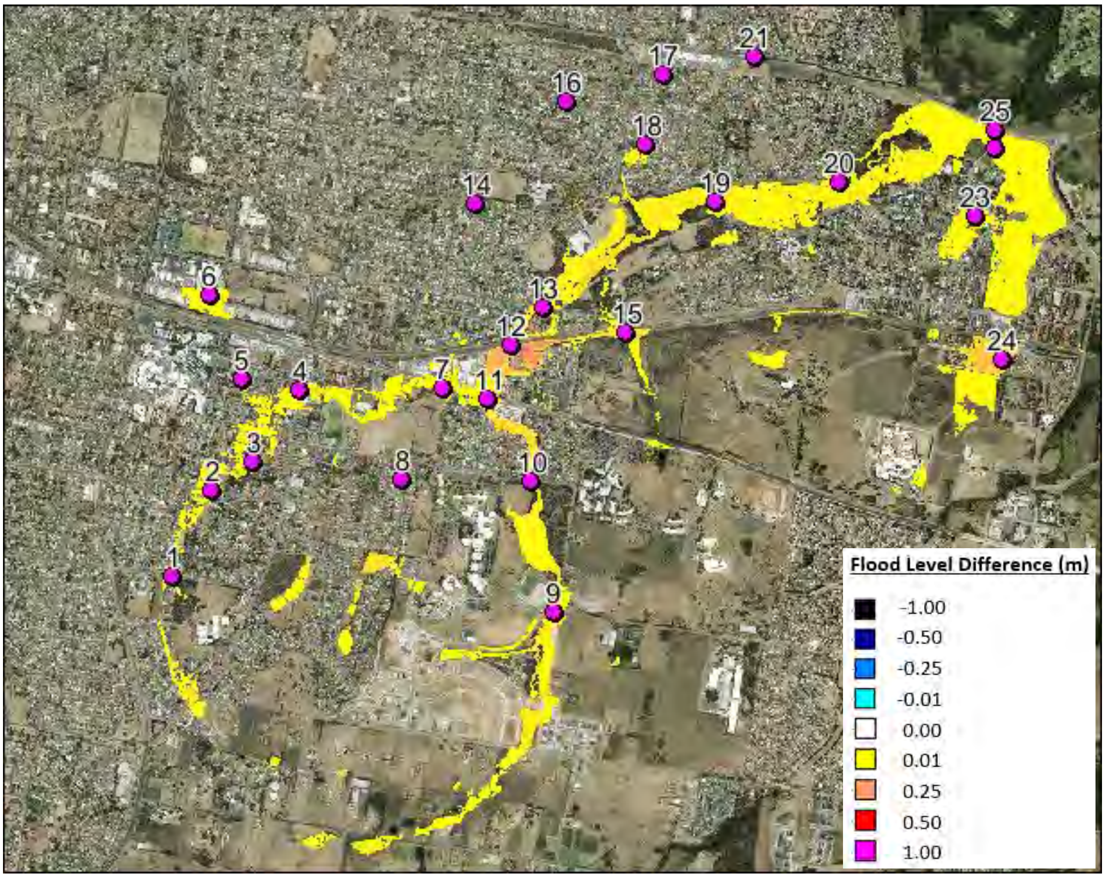


Plate 20 Flood level difference map with lower initial rainfall losses (i.e., wet catchment)

Overall, it can be concluded that the model is relatively sensitive to changes in the adopted initial losses across the majority of the catchment. Nevertheless, areas located upstream of the railway line show more significant sensitivity to the adopted initial rainfall losses. '*Australian Rainfall & Runoff*' (Engineers Australia, 1987) suggests adopting an initial loss of between 10 mm and 30 mm for design flood estimation. The adopted initial loss of 10 mm is at the lower end of the suggested range and would, therefore, provide reasonably conservative design flood level estimates across the catchment.

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

The TUFLOW model was used to re-simulate the 1% AEP flood with the modified continuing loss rates. Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping, which is presented in **Plate 21** and **Plate 22**. The difference maps were also statistically analysed and the outcomes of the analysis are presented in **Table 22**.

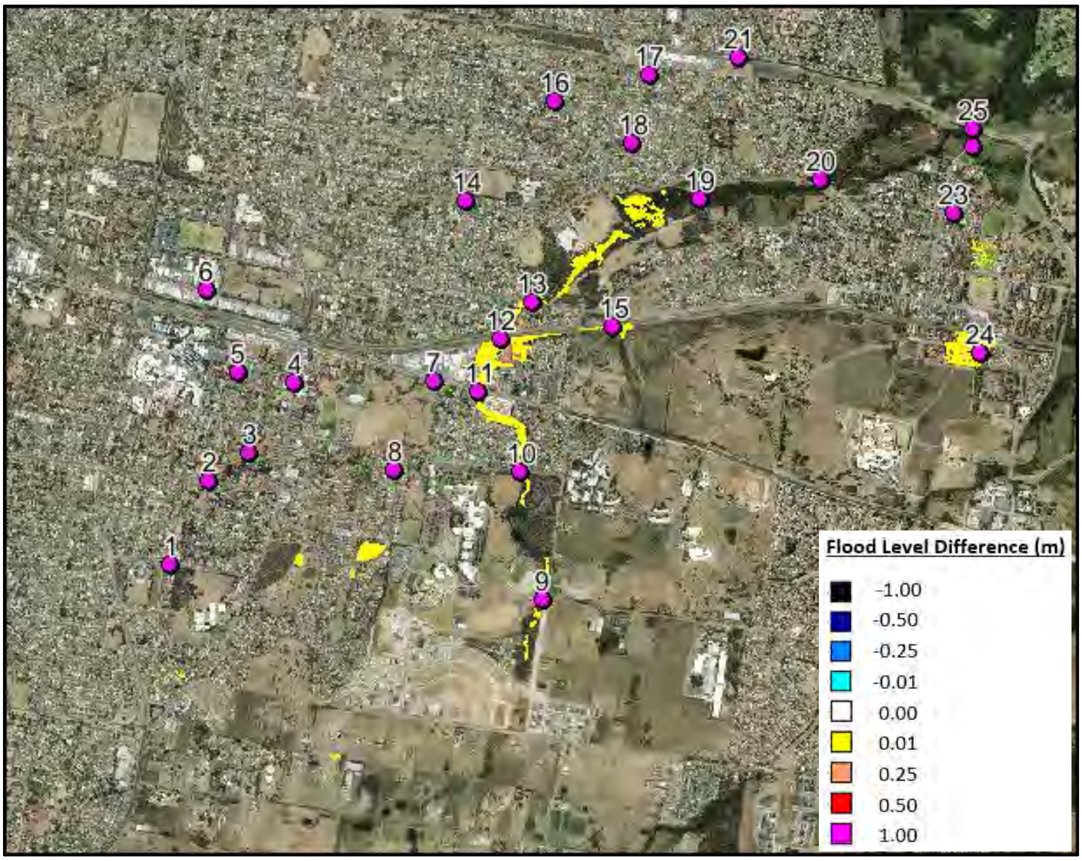


Plate 21 Flood level difference map with reduced continuing loss rates

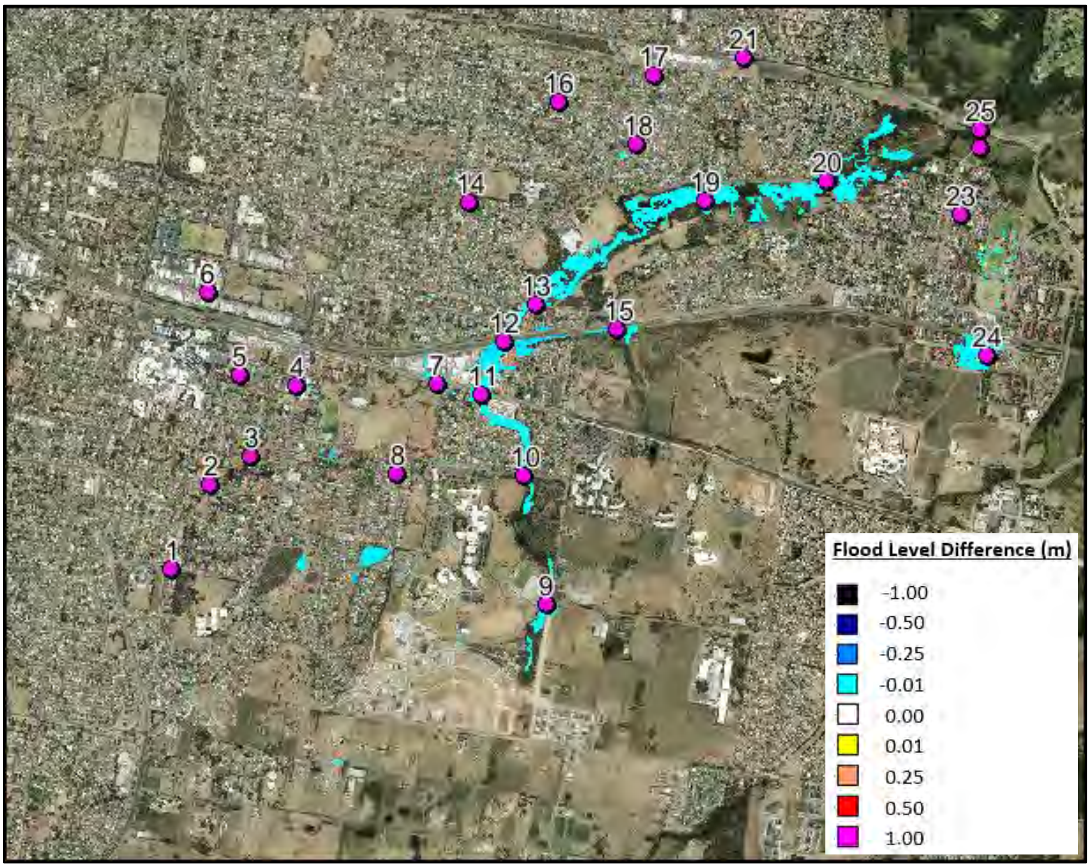


Plate 22 Flood level difference map with increased continuing loss rates

Peak 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented in **Table 23**.

The results of the sensitivity analysis show that the TUFLOW model is relatively insensitive to changes in continuing loss rates. More specifically, **Table 22** shows that only relatively 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 not predicted to have a significant impact on the results generated by the TUFLOW model.

8.2.3 Manning's "n"

Manning's' "n" roughness coefficients are used to describe the resistance to flow afforded by different land uses 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 1% AEP simulations were completed with the modified "n" values. Flood level difference mapping was prepared based on the results of the revised simulations and are presented in **Plate 23** and **Plate 24**.

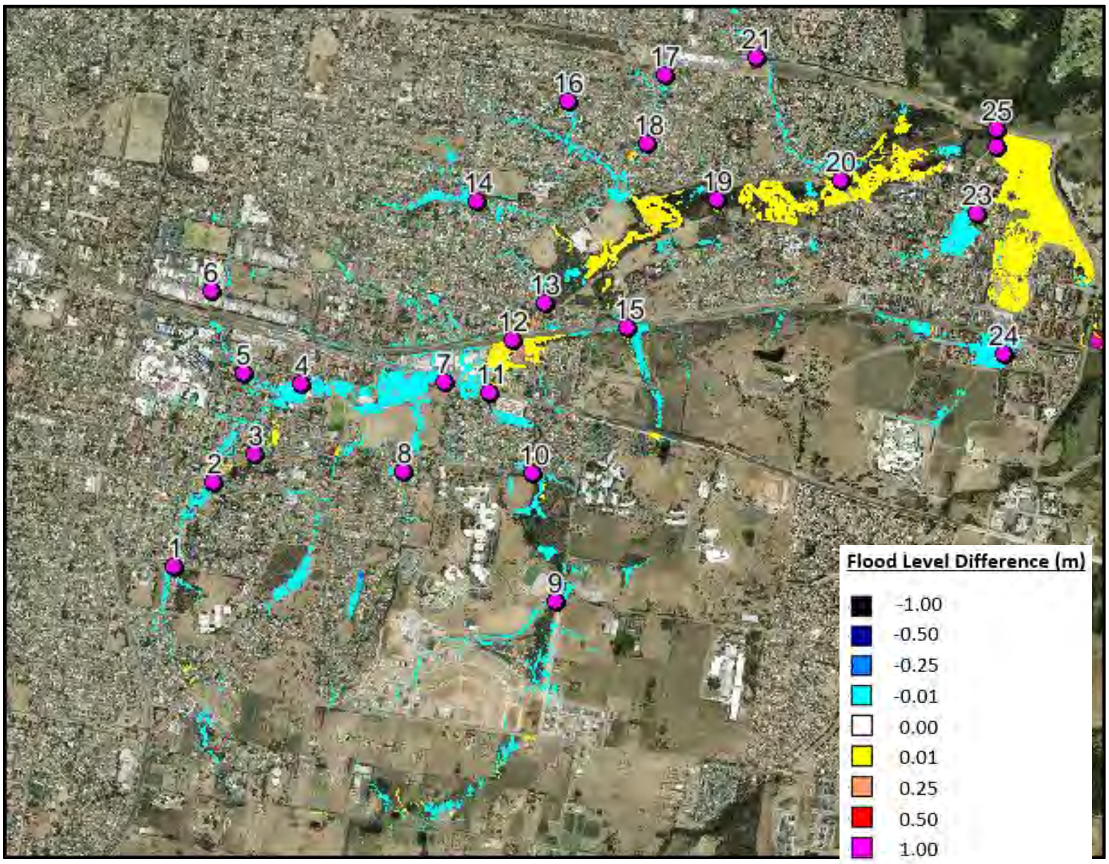


Plate 23 Flood level difference map with decreased Manning's "n" roughness values

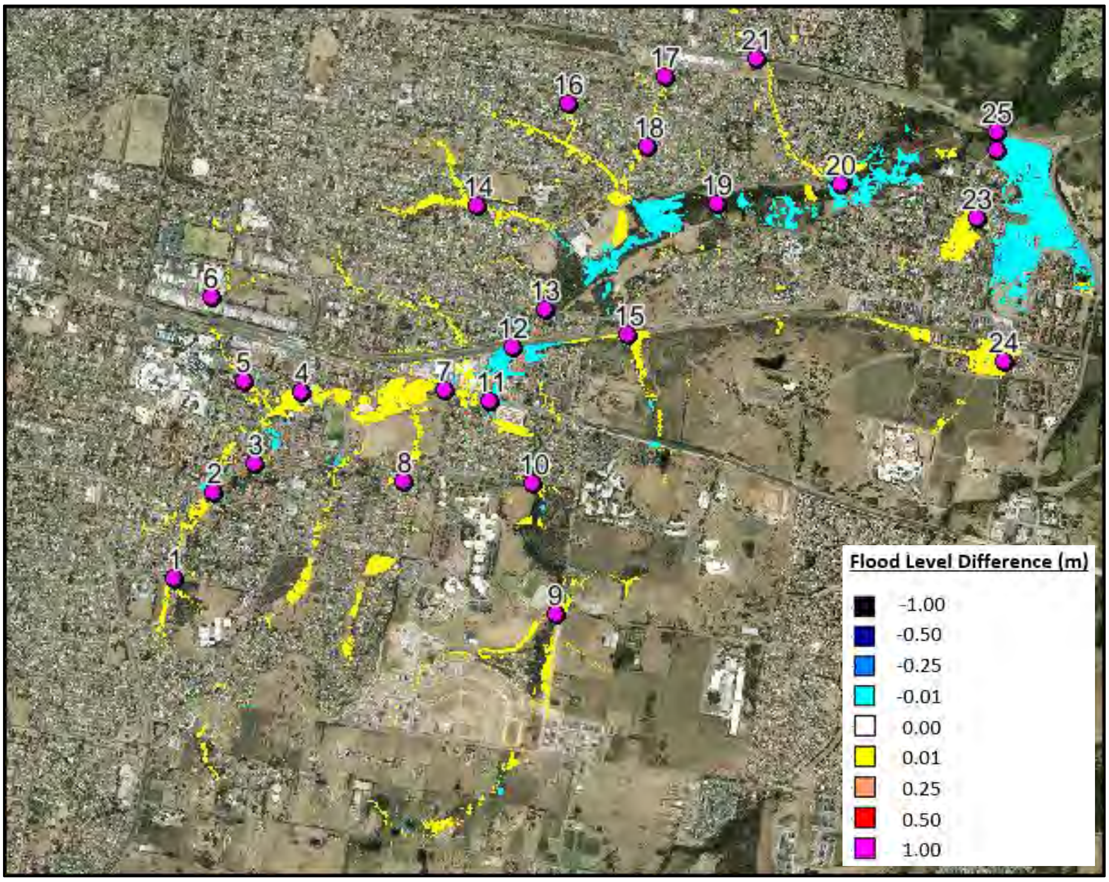


Plate 24 Flood level difference map with increased Manning's "n" roughness values

The difference maps were statistically analysed and the outcomes of the analysis are presented in **Table 22**. Peak 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented in **Table 23**.

The results show that altering the Manning 's "n" values has the potential to both increase and decrease "design" 1% AEP flood levels. **Plate 23** shows that decreasing the "n" values will typically lower flood levels along major flow paths and waterways as water is able to "escape" more readily from these areas. However, this can result in localised increases in water level across volume sensitive sections of the catchment where flow is concentrated (e.g., behind the railway line).

In general, the changes in 1% AEP flood levels are predicted to be less than 0.1 metres. As a result, it is considered that the model is relatively insensitive to changes in Manning's 'n' values.

8.2.4 Hydraulic Structure Blockage

As discussed in Section 6.2.3, blockage factors ranging between 0% and 100% 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.

Flood level difference mapping was prepared based on the results of the blockage sensitivity simulations and is presented in **Plate 25** and **Plate 26**. The difference maps were also statistically analysed and the outcomes of the analysis are presented in **Table 22**. Peak 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented in **Table 23**.

Plate 25 shows that no blockage will generally produce localised decreases in 1% AEP water levels upstream of major hydraulic structures and increase water levels downstream of major hydraulic structures as well as along major watercourses. In general, 1% AEP flood levels are predicted to change by less than 0.1 metres.

Plate 26 shows that complete blockage will cause some significant changes to 1% AEP flood levels. 1% AEP flood levels are predicted to increase by over 1.3 metres at some locations and are driven by the significantly elevated embankments in some areas (e.g., Railway line). There are predicted to be some commensurate decreases in water level downstream of these significant embankment structures and are associated with the "damming" effect provided by the embankment. However, complete blockage is predicted to increase water levels across the vast majority of the catchment.

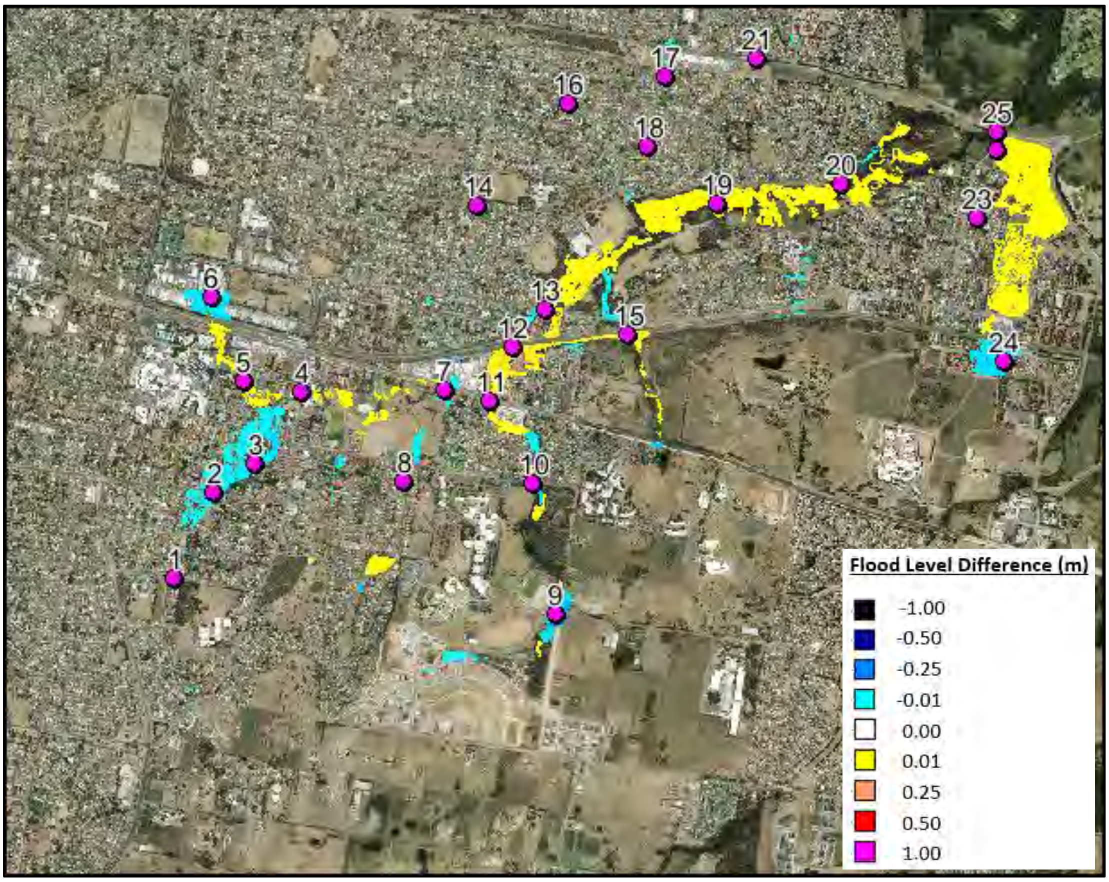


Plate 25 Flood level difference map with no blockage of hydraulic structures

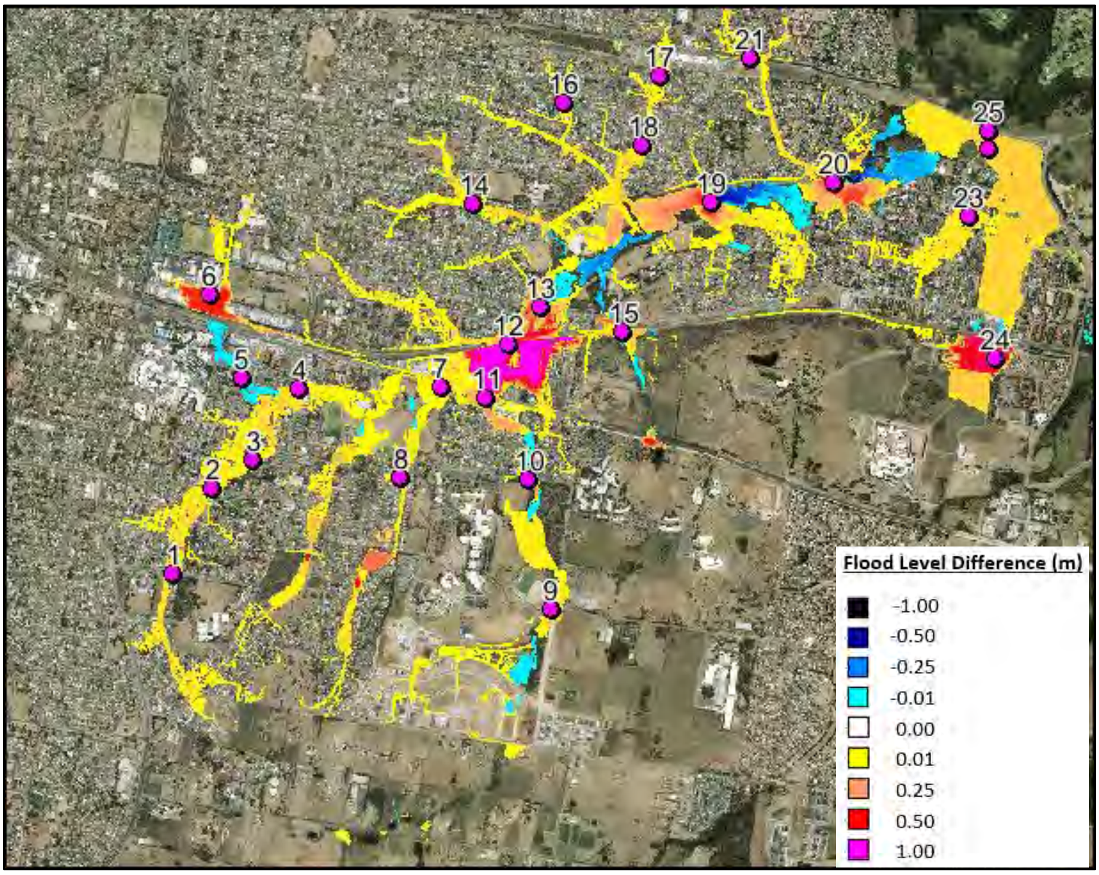


Plate 26 Flood level difference map with complete blockage of hydraulic structures

In general, changes to stormwater inlet blockage are not predicted to have a large impact on 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 major sub-surface pipe systems.

Overall, it is considered that the TUFLOW model is not particularly sensitive to stormwater inlet blockage. However, it should be noted that the stormwater system will convey a significant proportion of flow during more frequent rainfall events. Therefore, it is still important for the stormwater system to be well maintained to ensure it is capable of carrying the majority of flows during these more frequent events.

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. Areas located upstream of the railway line are predicted to be the most significantly impacted. 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).

8.2.5 South Creek Level

The College, Orth and Werrington Creeks catchment drains into South Creek, which forms the downstream boundary of the catchment. The "base" simulations assumed that a 5% AEP flood (peak South Creek water level = 21.29 mAHD) was occurring along South Creek at the same time as a 1% AEP flood within the Werrington Creek catchment. However, if the prevailing water level within South Creek at the time of a Werrington Creek flood was different, it has the potential to impact on results across the downstream sections of the College, Orth and Werrington Creeks catchment.

Therefore, additional sensitivity simulations were completed to assess the sensitivity of the model results to variations in the adopted South Creek water level. The simulations included:

- 1 in 2 year ARI water level within South Creek (water level = 20.5 mAHD); and,
- 0.2% AEP water level within South Creek (water level = 22.05 mAHD).

The TUFLOW model was used to re-simulate the 1% AEP flood with the different South Creek water levels. Flood level difference mapping was also prepared based on the results of the revised simulations and is presented in **Plate 27** and **Plate 28.** A water surface profile was also extracted along the downstream reaches of Werrington Creek for each simulation and is presented in **Plate 29**. Additional water surface profiles are also included in **Plate 29** for simulations that were completed with no flooding along South Creek as well as a 1% AEP tailwater along South Creek.

The difference maps were also statistically analysed and the outcomes of the analysis are presented in **Table 22**. Peak 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented in **Table 23**.

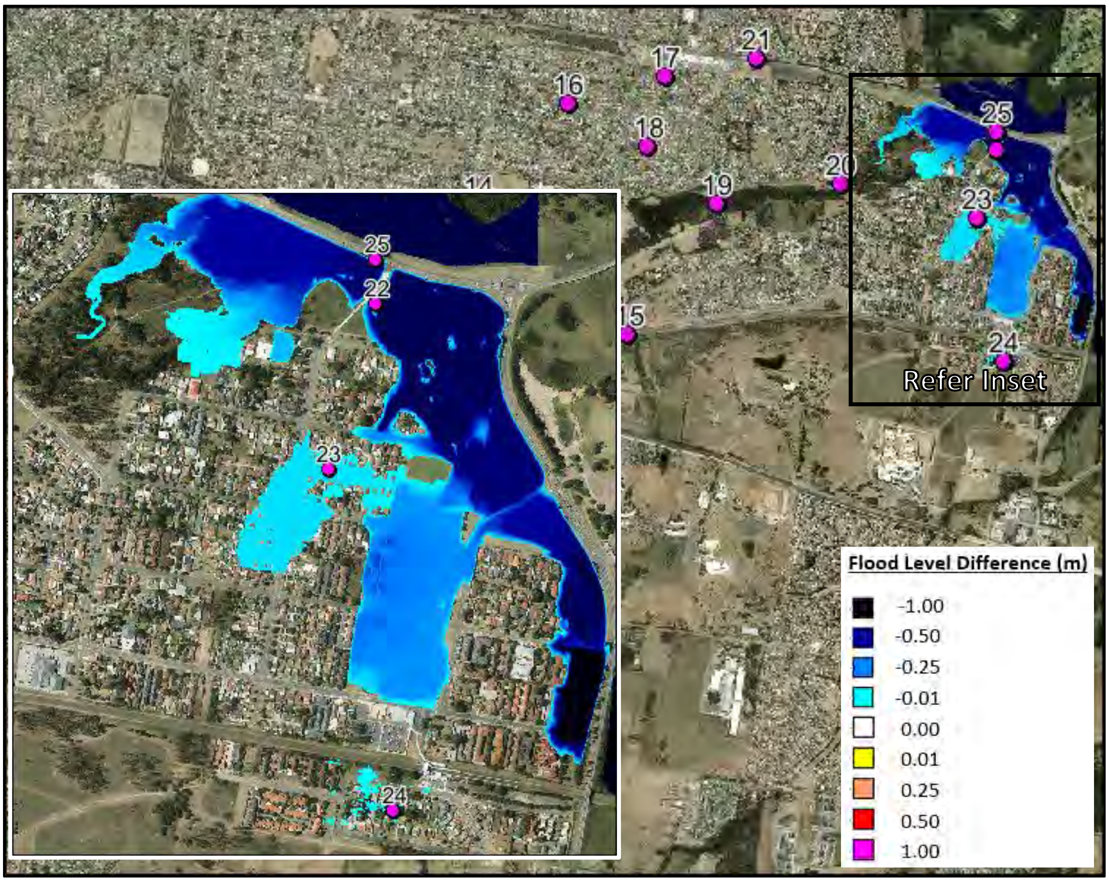


Plate 27 Flood level difference map with lower South Creek water level

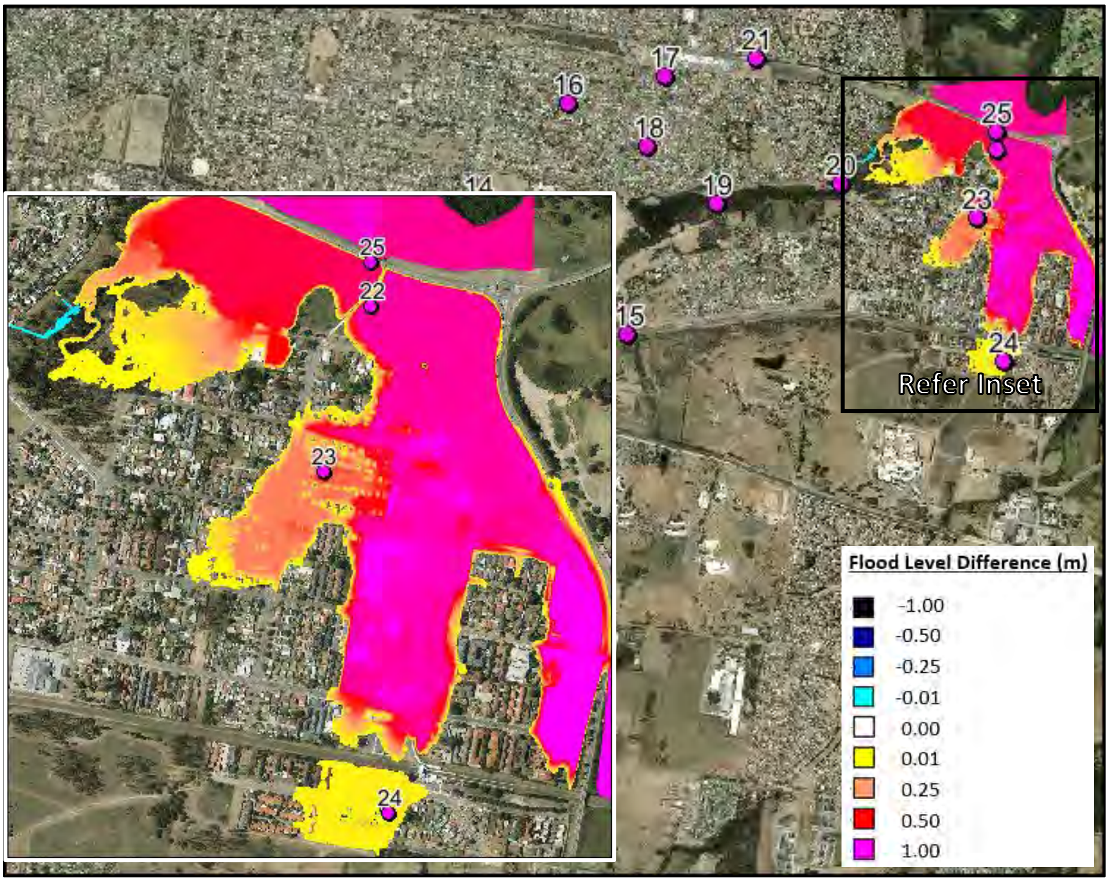


Plate 28 Flood level difference map with higher South Creek water level

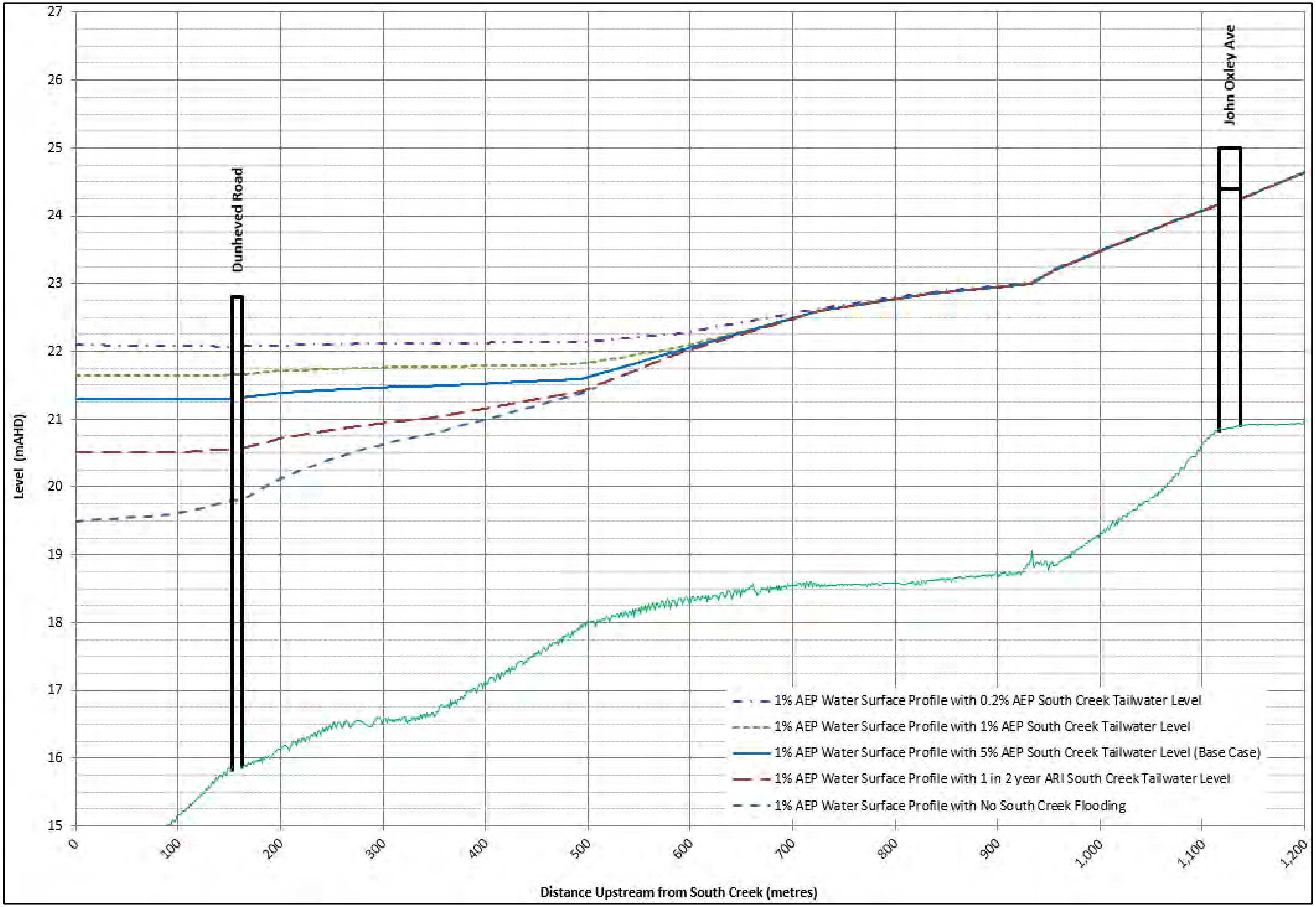


Plate 29 Peak 1% AEP water surface profile along Werrington Creek showing the impact of South Creek tailwater levels

The difference mapping indicates that variation in South Creek tailwater level can have a significant impact on flood levels across the downstream sections of the catchment. Areas of Werrington located upstream of the Werrington Road levee and earthen levee are predicted to be the most significantly impacted. Across these areas, changes in 1% AEP water level of around 1 metre are anticipated.

Along Werrington Creek, the South Creek tailwater level is predicted to influence 1% AEP water levels extending approximately 600 metres upstream of Dunheved Road. Upstream of this point, the South Creek water level is not predicted to have an impact on 1% AEP water levels (i.e., catchment runoff dominates).

Overall, it can be concluded that the 1% AEP flood levels across the downstream sections of the catchment are sensitive to changes in the adopted South Creek level. However, flood level impacts across the upstream sections of the catchment are predicted to be negligible.

8.2.6 Flood Gates

As discussed in Section 6.2.4, the College, Orth and Werrington Creeks catchment incorporates two culverts that include flood gates (also referred to as flood "flaps"). The gated culverts are located at the following locations and serve to prevent "backwater" inundation from South Creek:

- Werrington Road culvert (located approximately 200 metres north of railway); and
- Werrington earthen levee culvert (located approximately 100 metres north of Reid Street and 50 metres south of Dunheved Road).

As part of the 'base' design flood simulations, it was assumed that the Werrington Road flood gate remained closed but the Werrington levee gate "failed" (i.e., remained open). However, to gain an understanding of how flood levels upstream of each flood gate may be altered under different flood gate operation scenarios, two additional flood gate sensitivity simulations were completed:

- Both flood gates fully operational; and,
- Both flood gates not operational.

The TUFLOW model was updated to include both flood gate scenarios and was used to re-simulate the 1% AEP flood. Flood level difference mapping was also prepared based on the results of each simulations and is presented in **Plate 30** for the fully operational scenario and **Plate 31** for the not operational scenario.

The difference maps were also statistically analysed and the outcomes of the analysis are presented in **Table 22**. Peak 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented in **Table 23**.

The difference mapping shows that if the flood gates remain fully functional, it would reduce peak 1% AEP water levels by up to 1 metres across those sections of Werrington located behind the Werrington Road and Werrington earthen levee. The sensitivity of the model results across this area are associated with the significant flood storage volume that is consumed when elevated water levels from South Creek "back up" and inundate the areas surrounding the Parkes Avenue Sporting complex.

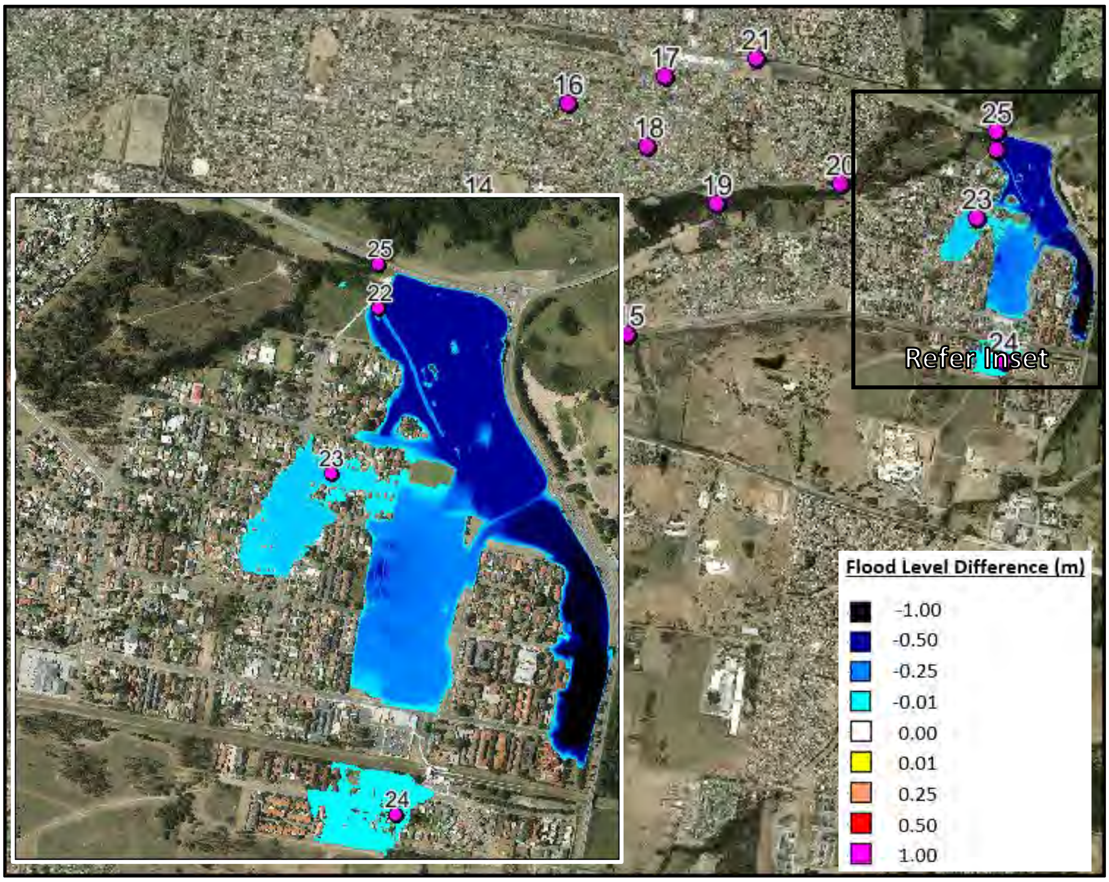


Plate 30 Flood level difference map with all flood gates operational

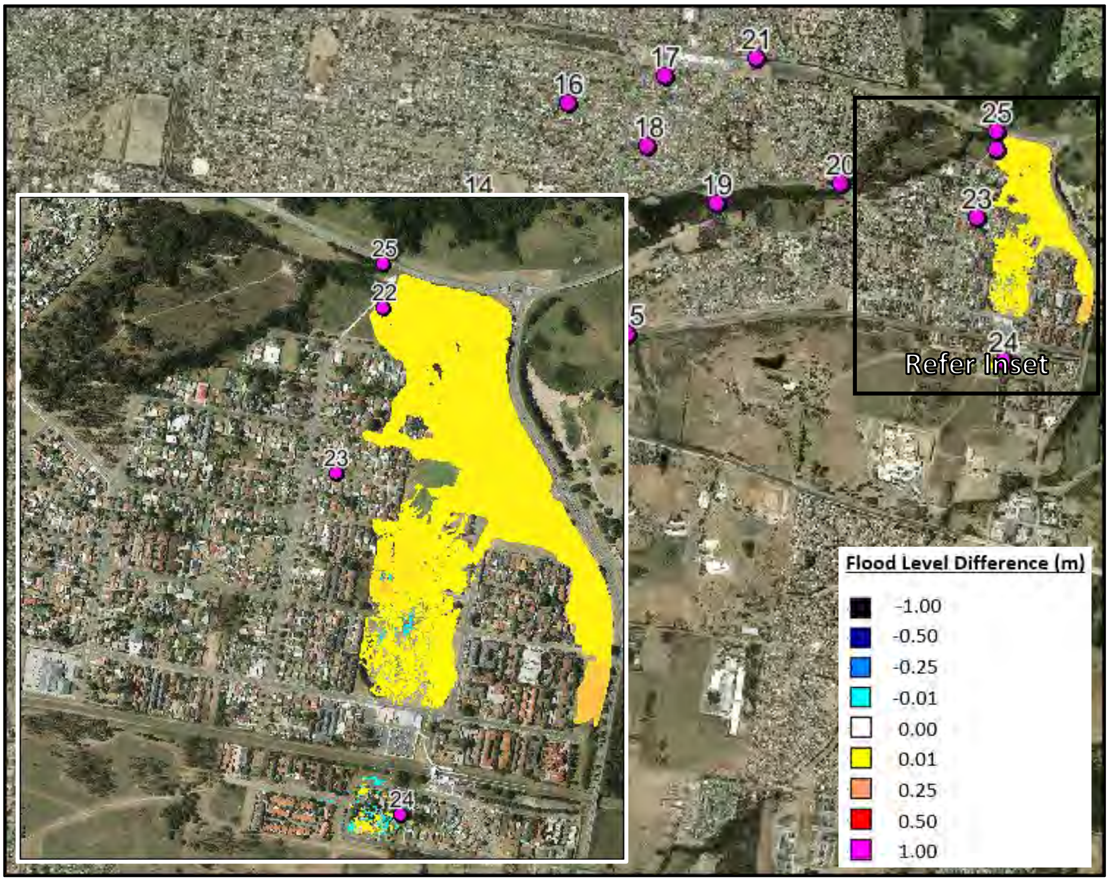


Plate 31 Flood level difference map with all flood gates not operating

Conversely, if neither of flood gates are operational, peak 1% AEP flood levels are predicted to increase by less than 0.1 metres. The lack of sensitivity of the model results under this scenario is likely associated with the relatively small size of Werrington Road culvert versus the Werrington levee culvert. That is, the impact of the Werrington Road flood gates remaining open is largely "drowned out" by the comparatively large Werrington levee culvert.

Overall, the flood gates appear to provide a significant benefit in reducing design flood levels behind the levee system when they are fully operational. However, as discussed in Section 6.2.4, debris can prevent the flood gates from operating as intended. Therefore, the flood gates should also be subject to regular clearing and maintenance to reduce the potential for debris accumulation.

8.2.7 Revised Design Rainfall

Design rainfall was applied to the TUFLOW model based upon standard procedures documented in *"Australian Rainfall and Runoff – A Guide to Flood Estimation"* (Engineers Australia, 1987). However, at the time this study was being prepared, a new version of *"Australian Rainfall and Runoff"* was in the process of being released. The revised version includes new intensity-frequency-duration (IFD) information that takes advantage of over 30 years of additional rainfall information. Although the revised IFD data has been released, the Bureau of Meteorology and Engineers Australia suggests that the revised IFD data should not be used for design flood estimation until the full suite of revised techniques is released as part of the new version of *"Australian Rainfall and Runoff".* Nevertheless, the Bureau of Meteorology and Engineers Australia recommends that the revised IFD data be used as part of sensitivity testing.

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

Flood level difference mapping was prepared based on the outcomes of the 1% AEP simulation with the revised IFD values and is presented in **Plate 32**.

The difference mapping was also statistically analysed and the outcome of this assessment is presented in **Table 22**. Peak 1% AEP flood levels were also extracted from the results of the sensitivity simulations at various locations across the catchment and are presented in **Table 23**.

The results presented in **Plate 32** show that the revised IFD values are predicted to generate reductions in 1% AEP flood levels. The reductions are typically less than 0.15 metres and are contained along the main overland flow areas and creek lines.

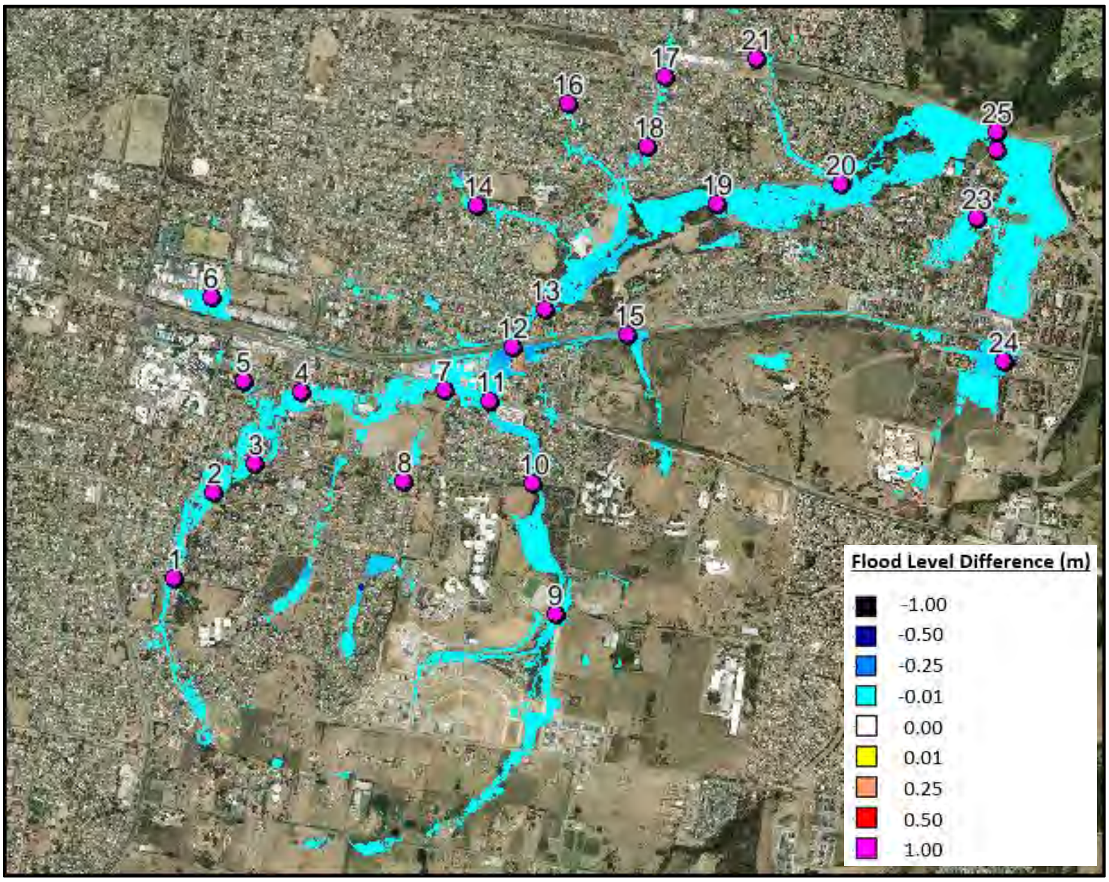


Plate 32 Flood level difference map with 2013 IFD Data

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

8.3 Climate Change Analysis

8.3.1 Overview

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

To gain an understanding of the potential impact that climate change-induced rainfall intensity increases may have on flood behaviour across the catchment, additional climate simulations where completed. Due to the wide potential variability of future rainfall intensities, the *'Practical Consideration of Climate Change'* (Department of Environment and

Climate Change, 2007) recommends that additional simulations should be completed with 10%, 20% and 30% increases in rainfall intensities to quantify the potential impacts associated with climate change. The outcomes of the additional climate change simulations are presented below.

8.3.2 Rainfall Intensity Increases

The TUFLOW model was used to perform additional simulations including 10%, 20% and 30% increases in 1% AEP rainfall intensities. In addition to increases in rainfall intensity across the local catchment, the South Creek tailwater elevations were also increased to reflect increases in rainfall across the broader South Creek catchment. This was achieved by:

- updating the South Creek XP-RAFTS model with the rainfall intensity increase;
- re-simulation of the 1% AEP event to derive a revised peak discharge estimate for South Creek and the Werrington Creek confluence; and,
- conversion of the peak discharge to an equivalent flood level using the rating curve discussed in Section 6.2.1.

Peak floodwater levels were extracted from the results of the modelling and were compared against peak water flood levels for 'base' 1% AEP conditions. This allowed water level difference mapping to be prepared showing the magnitude of any change in water levels associated with the increases in rainfall intensity. The difference mapping is presented in **Plate 33**, **Plate 34** and **Plate 35**.

The difference maps were also statistically analysed and the outcomes of the analysis are presented in **Table 24**.

Table 24 Percentile Change in 1% AEP Flood Levels Associated with Climate Change

Climate Change Scenario	1st	5th	10th	25th	50th	75%	90th	95th	99th
10% increase in 1% AEP rainfall	0.00	0.01	0.01	0.02	0.05	0.09	0.12	0.14	0.31
20% increase in 1% AEP rainfall	0.00	0.01	0.02	0.04	0.10	0.22	0.28	0.30	0.65
30% increase in 1% AEP rainfall	0.01	0.02	0.03	0.06	0.14	0.35	0.44	0.46	0.92

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

The results show that a 10% increase in rainfall intensity has the potential to increase peak 1% AEP water levels by over 0.3 metres at some locations. This is predicted to rise to over 0.9 metres during the 30% increase in rainfall scenario.

Again, the most significant changes in flood level are concentrated upstream of the railway line and, in particular, the Werrington Creek culvert crossing. However, areas adjoining South Creek (e.g., Werrington) are also predicted to be significantly impacted, particularly during the 30% increase in rainfall scenario where increases in South Creek "backwater" levels are significant.

Table 25 Peak 1% AEP Flood Levels from Climate Change Simulation at Various Location across the Catchment

Location (refer to Plates 33 to 35 for locations)		**Base Case** (mAHD)	**10% Increase in Rainfall** (mAHD)	**20% Increase in Rainfall** (mAHD)	**30% Increase in Rainfall** (mAHD)
1	Smith Street, Kingswood	54.04	54.06	54.07	54.08
2	Jamison Rd, Kingswood	49.25	49.27	49.28	49.31
3	Stafford St, Kingswood	46.79	46.82	46.85	46.87
4	Bringelly Rd, Kingswood	42.98	43.04	43.10	43.16
5	Orth St, Kingswood	45.40	45.42	45.44	45.46
6	Cox Ave, Kingswood	50.94	51.01	51.07	51.14
7	Chapman Gardens	37.59	37.60	37.61	37.62
8	Edna St, Kingswood	43.10	43.12	43.14	43.15
9	O'Connell St, Kingswood	45.99	46.05	46.09	46.14
10	Second Ave, Kingswood	39.07	39.20	39.31	39.41
11	Great Western Hwy	36.38	36.45	36.52	36.57
12	Railway (Werrington Ck)	34.57	34.89	35.26	35.56
13	Victoria St, Kingswood	32.00	32.10	32.18	32.26
14	Wrench St, Cambridge Park	39.45	39.46	39.49	39.51
15	Railway (French St)	36.55	36.63	36.68	36.71
16	Wembley Ave, Cambridge Park	38.67	38.68	38.70	38.72
17	Oreton Pl, Cambridge Park	37.23	37.26	37.28	37.29
18	Glencoe Ave, Cambridge Park	33.55	33.58	33.61	33.63
19	Burton St, Werrington	27.43	27.51	27.57	27.62
20	John Oxley Ave, Werrington	24.27	24.35	24.43	24.49
21	Lockyer Ave, Werrington County	34.81	34.93	34.95	34.96
22	Werrington Levee, Werrington	21.47	21.63	21.79	21.95
23	Dunkley Pl, Werrington	21.87	22.06	22.11	22.13
24	Walker Pl, Werrington	23.78	23.93	24.08	24.17
25	Dunheved Rd, Werrington	21.32	21.41	21.54	21.67

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.

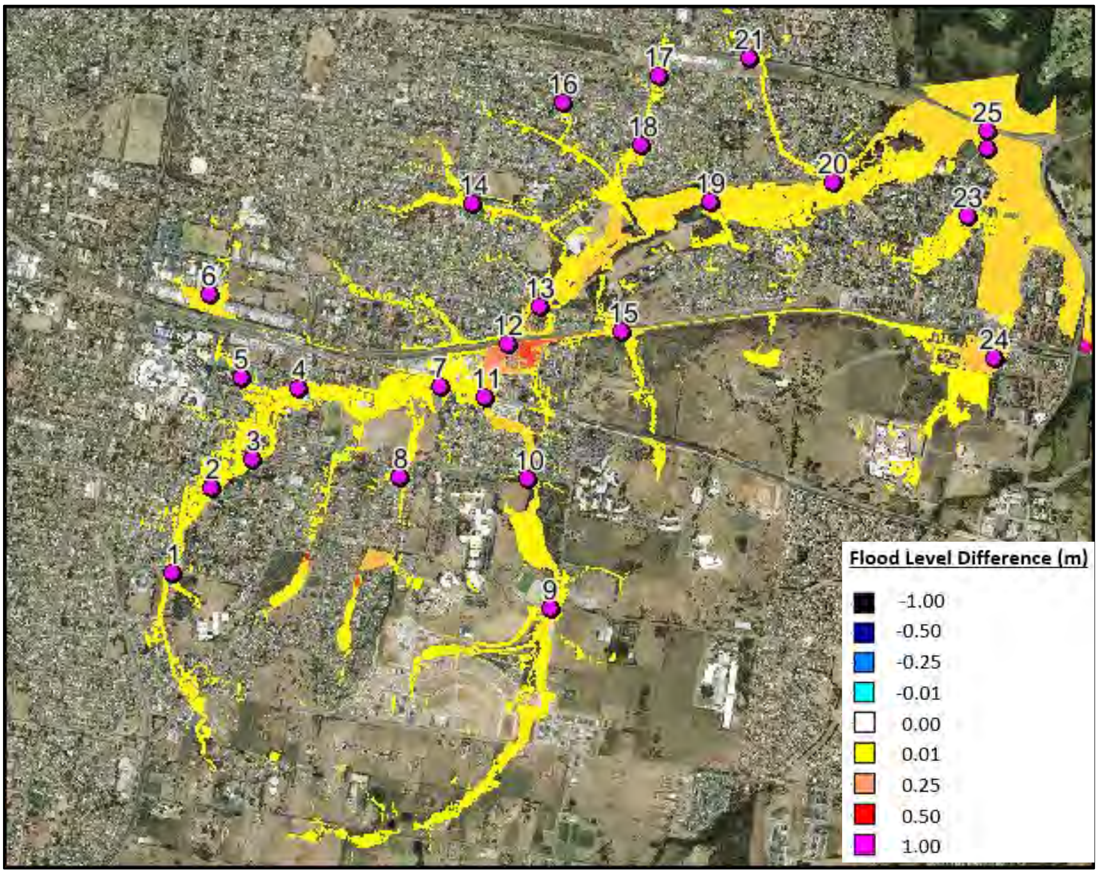


Plate 33 Flood level difference map with 10% increase in Rainfall

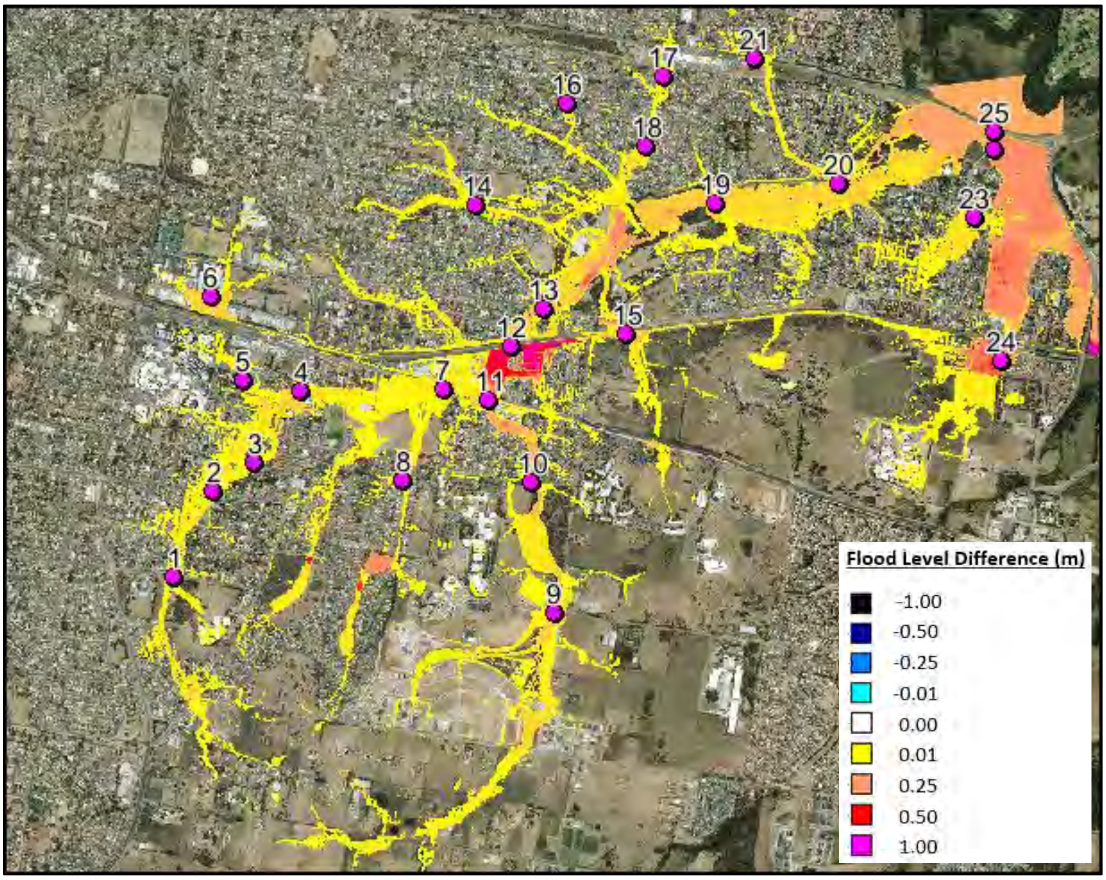


Plate 34 Flood level difference map with 20% increase in Rainfall

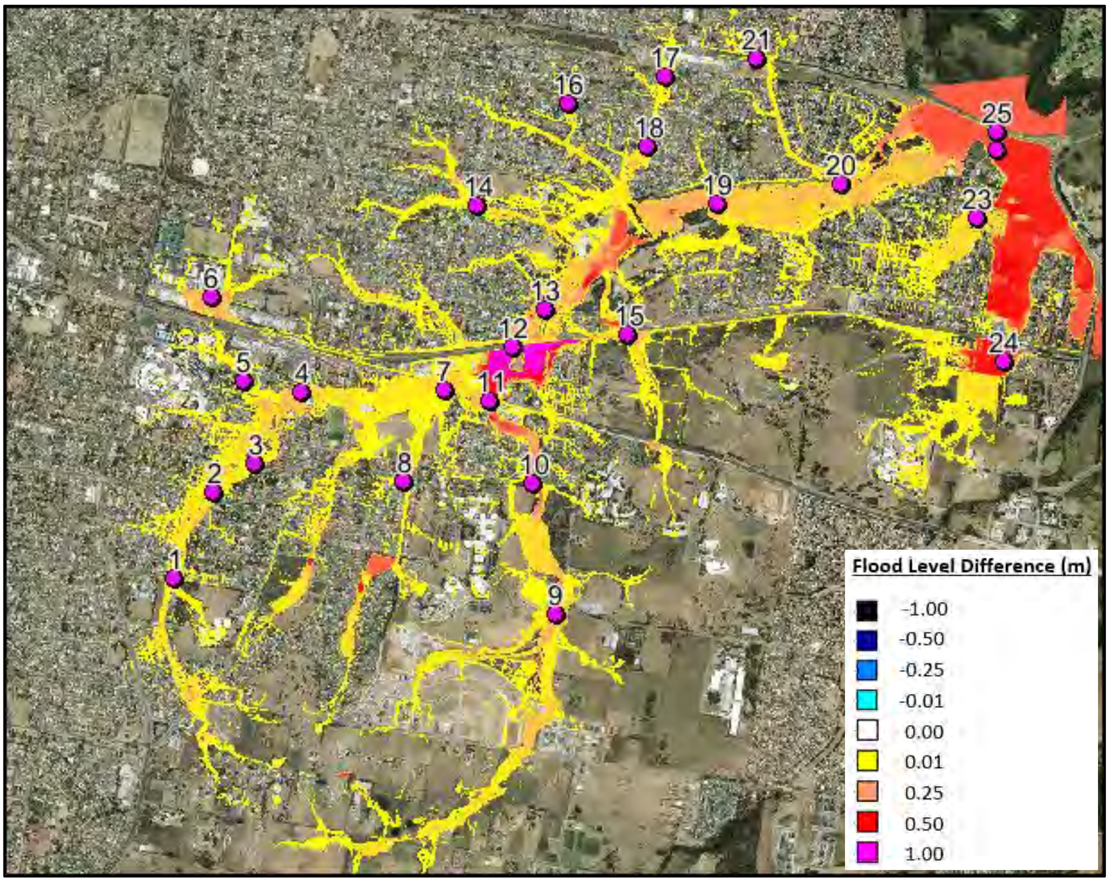


Plate 35 Flood level difference map with 30% increase in Rainfall

9 Flood Planning Area

9.1 Computer Model Confidence Limits

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

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

In recognition of this uncertainty, additional statistical analyses were completed based upon the outcomes of the various sensitivity and climate change simulations in an attempt to assign "confidence limits" to the peak 1% AEP flood level estimates.

In order to reliably define confidence limits to the 1% AEP results, it would be necessary to undertake thousands (potentially tens of thousands) of simulations to reflect the numerous combinations of potential parameter estimates and provide a sufficiently large population to enable meaningful statistical analysis. Unfortunately, the long simulation times only permit a limited number of parameter scenarios to be investigated.

In instances where a sufficiently large "population" of results is not available, it is still possible to derive confidence limits using the Student's t-test (Zhang, 2013). This approach involves interrogating peak flood level estimates from all 1% AEP simulations at each TUFLOW grid cell. This information is used to calculate a mean water level and standard deviation at each grid cell. This information can then be combined with the number of degrees of freedom (i.e., number of different 1% AEP simulations minus 1) and a "t-table" to develop 95% confidence limit estimates at each TUFLOW grid cell.

The resulting "99% Confidence Limit" grid is shown in **Plate 36**. Yellow colours indicate small confidence limits (i.e., more confidence in results) and red colours indicate higher confidence limits (i.e., less confidence in results). It is noted that the Student's t-test assumes that the population of results is "normally" distributed with the majority of the parameters and results located in close proximity to the mean. However, the sensitivity analysis typically adopts parameter values that are at the extremes of realistic ranges. As a result, the population of water level results is unlikely to be normally distributed. As a result, the calculated confidence limits are likely to be conservative.

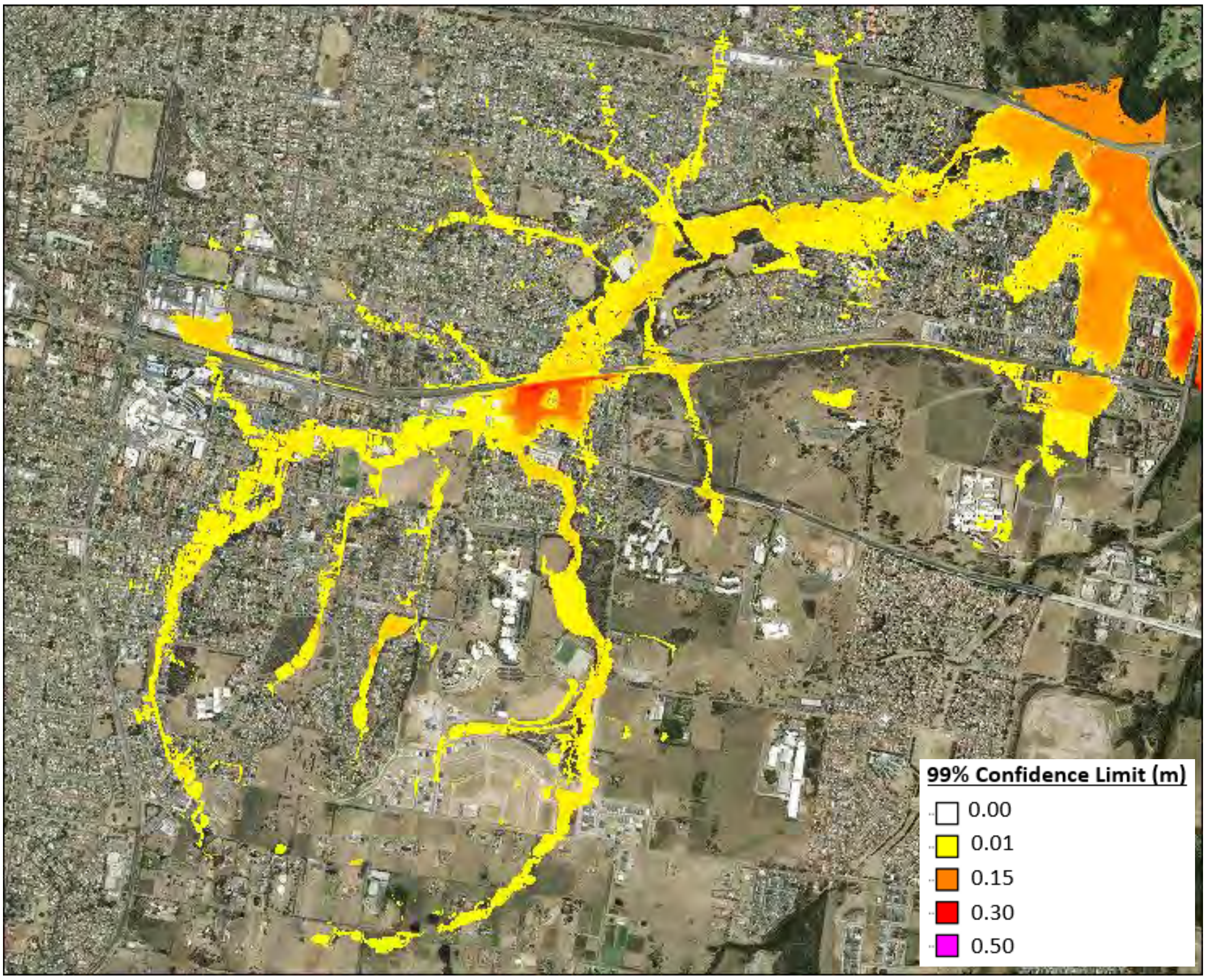


Plate 36 99% Confidence Interval Grid Developed Based Upon Student's t-test

The confidence interval grid provided in **Plate 36** shows that across the majority of the catchment, the confidence interval is better than 0.10 metres. That is, we are 99% confident that the "true" 1% AEP flood level is contained within ±0.10 metres of the "base" design simulations documents in Section 6 across the majority of the catchment.

However, some localised areas are subject to greater uncertainty (i.e., larger confidence limits). This includes the Werrington Creek crossing of the railway line where the confidence limits approach 0.5 metres and the area enclosed behind the Werrington Road and Werrington earthen embankment where the confidence limits are around 0.3 metres.

9.2 Flood Planning Area

9.2.1 Flood Planning Level

Flood Planning Levels (FPLs) are an important tool in the management of flood risk. FPLs are derived by adding a freeboard to the "planning" flood. 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.

As discussed, 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. This is consistent with the “Guideline on Development Controls on Low Flood Risk Areas – Floodplain Development Manual” (Department of Planning, 2007) which states that “…*unless there are exceptional circumstances, councils should adopt the 100 year flood as the FPL for residential development”*. Accordingly, the 1% AEP flood is considered to be appropriate for application as the planning flood to the College, Orth and Werrington Creeks catchment.

Freeboard is a factor of safety that is used to account for uncertainties in deriving the planning flood levels. Penrith City Council currently adopts a 0.5 metre freeboard for all flood study areas. Accordingly, Council wished to confirm the suitability of adopting a 0.5 metre freeboard across the College, Orth and Werrington Creeks catchment.

Freeboard is used to account for the following uncertainties:
- Model parameter uncertainty;
- Climate change;
- “Local” factors that can’t be explicitly represented in the computer modelling (e.g., small flow paths less than the model grid size); and
- Wave action (e.g., wind, boat or car induced waves)

As discussed, the results of the sensitivity and climate change assessment were used to develop a model confidence interval grid which can be used to quantify the uncertainty associated with model parameters and climate change (refer **Plate 36**). The confidence interval grid shows that in areas of significant inundation depths (i.e., depths greater than 0.3 metres), modelling uncertainty can be significant. More specifically, the 95th percentile uncertainty is predicted to be 0.27 metres. Therefore, a freeboard of at least 0.3 metres is considered to be necessary across areas of significant inundation depths to account for modelling and climate change uncertainty.

Unfortunately, the uncertainty associated with the remaining factors 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 37**, 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.

Overall, it is considered that a freeboard that accounts for the following uncertainties would be approriate:
- Modelling and climate change uncertainty = 0.30 metres; and
- *‘Other’* uncertainty (e.g., wave action) = 0.15 metres

Plate 37 Example of cars driving through flood waters and generating waves

Accordingly, a minimum freeboard of 0.30 metres + 0.15 metres = 0.45 metres is considered to be reasonable. Therefore, the adoption of a 0.5 metres appears to suitably account for uncertainty in 1% AEP flood level estimates across areas of significant water depth.

Consideration could be given to implementing a higher freeboard for the area located immediately upstream of the Werrington Creek railway culverts where modelling uncertainty alone is predicted to exceed 0.5 metres. 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 detail investigation through the subsequent floodplain risk management study process.

9.2.2 Flood Planning Area

The 0.5 metres freeboard was added to the peak 1% AEP water level results grid generated by the TUFLOW model 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 flow.

The resulting flood planning area is shown in **Figure 55**.

9.2.3 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.

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 is summarised below and includes the following areas:

- Jamison Road and Somerset Street to Bringelly Road, Kingswood;
- Chapman Gardens and Great Western Highway, Kingswood;
- Cox Avenue, Kingswood; and
- Railway Street, Landers Street and Walker Street, Werrington.

A list of potential flood and drainage mitigation measures are prepared for the 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. The outcomes of the mitigation measures assessment are also presented in the following sections.

10.2 Flooding “Hot Spots” and Potential Mitigation Measures

10.2.1 Jamison Road and Somerset Street to Bringelly Road, Kingswood

As shown in **Figure 19.3**, a major overland path extends through a number of residential and commercial properties between Jamison and Bringelly Roads at Kingswood. A secondary flow path also extends from Somerset Street and joins the primary flow path near the corner of Orth Street and Bringelly Road. At the peak of the 1% AEP flood, floodwater depths are predicted to approach 1 metre at some locations.

Figure 37.3 also shows that peak flow velocities are predicted to exceed 2 m/s at a number of locations. Fortunately, the most significant depths and highest velocities tend to be concentrated across areas of open space and roadways, where water is able to flow “unabated”. However, significant velocities are also predicted in areas where overland flow is squeezed between buildings.

As shown in **Figure 42.3,** the stormwater system has a capacity of less than the 1 in 2 year ARI storm through most of this area. **Figure 42.3** also shows that the pipe system is discontinuous immediately north of Jamison Road. Accordingly, during significant rainfall events, water is predicted to start surcharging into the detention basin located on the northern side of Jamison Road. This is predicted to cause overland flows to start “backing up” into Jamison Road. This water is then predicted to overtop the kerb in Jamison Road and spill through properties adjoining the detention basin.

This failure mechanism is illustrated in **Plate 38**, which shows water depths 40 minutes after the initial onset of rainfall during the 1% AEP, 120-minute simulation. It shows water depths backing up across Jamison Road and spilling through the properties on the western side of the basin. Significant ponding depths are also evident near the intersection of Bringelly Road and Orth Street during the early stages of the flood.

Plate 38 1% AEP Depths between Jamison and Bringelly Roads after 40mins of rainfall.

Similarly, the trunk drainage system between Somerset Street and Bringelly Road is predicted to be exceeded during events equal to or greater than the 1 in 2 year ARI storm. **Plate 39** shows that water begins to surcharge near the corner of Rodgers Street and Somerset Street as well as part way along Rodgers Street. Water is then predicted to overtop the gutter to the west of 32 Rodgers Street and discharge south through vacant land to Orth Street (although water is predicted to spill through an existing residential property located at 32 Rodgers Street).

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 39 1% AEP Depths between Somerset and Orth Streets after 35mins of rainfall.

There are some areas of "open space" scattered across the area, most of which are located in close proximity to the overland flow paths. These may be suitable locations for the construction of "dry" detention basins. Areas considered to be suitable for consideration as detention basins are shown on **Figure 56.1** and include:

- North-western corner of Kingswood High School;
- South of Stafford Street;
- North of Stafford Street;
- Corner of Bringelly Road and Orth Street;
- Car parking area near the corner of Somerset and Rodgers Street; and
- 27-31 Orth Street.

Moreover, it may be possible to augment existing basins (e.g., Jamison Road basin) to provide additional flood storage capacity. This could be achieved by lowering the base of the existing basin.

Stormwater upgrades across areas of open space and roadways may also afford some benefits, particularly during more frequent rainfall events. Most notably, the construction of a new low flow pipe beneath the Jamison Road basin is likely to reduce the frequency of floodwaters "backing up" the stormwater system and inundating properties adjoining Jamison Road. Other areas that would benefit from stormwater upgrades are shown in **Figure 56.1** and include:

- Stormwater inlet and pit upgrades at the following locations:
  - Jamison Road basin
  - Low point in Stafford Street
  - Near the intersection of Somerset and Rodgers Streets

- Stormwater pipe upgrades at the following locations:
  - Stapley Street to Jamison Road
  - Derby Street to First Street

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 should also be noted that the Cox Avenue "hot spot" (discussed further in section 10.2.3) drains to Somerset Street. Therefore, any mitigation options that are implemented to reduce flooding across the Cox Avenue area may also impact on flooding across this area.

10.2.2 Cox Avenue, Kingswood

Cox Avenue at Kingswood is located north of the railway line and is located within an industrial area. In the vicinity of Phillip Street, the topographic relief is very subtle and a number of overland flow impediments are evident including buildings as well as Colorbond-type fencing. As shown in **Figure 19.7**, these catchment characteristics can result in significant inundation depths and extents at the peak of the 1% AEP event.

The "sag" point in Cox Avenue is drained by a 1.2 metre diameter pipe that discharges runoff into the railway reserve. The pipe capacity mapping shown in **Figure 42.7** indicates that this pipe system only has sufficient capacity to convey the 1 in 2 year ARI event. Furthermore, the downstream end of the 1.2 metre appears to be partly blocked by silt and debris which would serve to further reduce the capacity of the pipe system (refer **Plate 40**). Responses that were received as part of the community questionnaire confirms that inundation across this area occurs relatively frequently.

Plate 40 View looking north showing partially blocked outlet of 1.2 metre diameter pipe draining Cox Avenue area.

A review of the flood modelling results indicates that the capacity of the stormwater system in Cox Avenue is overwhelmed relatively early during the 1% AEP event (i.e., after about 20 minutes) with water ponding at the sag point in Cox Avenue and spilling into adjoining properties to the south of Cox Avenue (refer **Plate 41**). Peak water depths are typically experienced 40 minutes after the initial onset of rainfall with water fully receding after approximately 1 hour and 15 minutes. Accordingly, flooding across this area is "flashy" with little opportunity to elevate stock and equipment or evacuate to higher ground.

Plate 41 1% AEP Depths in vicinity of Cox Avenue after 35mins of rainfall.

Potential Mitigation Options

As with the other "hot spot" areas discussed above, the inundation problems in the vicinity of Cox Avenue are primarily associated with the limited capacity of the stormwater system. Therefore, mitigation measures should focus on providing additional capacity to drain the sag point in Cox Avenue and/or reduce the amount of runoff reaching this section of the catchment (e.g., through detention basins or storage areas).

The only areas of open space within this subcatchment that may be suitable for flood storage areas include the St Dominic's College sports fields and Penrith General Cemetery. Unfortunately, the sports fields are elevated well above the adjoining roadways. Therefore, significant earthworks would be necessary to utilise these areas for flood storage. The cemetery is also actively utilised, so the provision of a flood storage area across this area may be considered undesirable regardless of whether the flood storage is only active for short and relatively infrequent periods. The south-western corner of the cemetery is currently unused and includes an existing outlet structure so may be suitable for a small storage area (refer **Figure 56.2**).

Drainage upgrades are likely to afford some benefits during more frequent rainfall events. The drainage upgrades would be concentrated around the Cox Avenue sag point. The location where drainage upgrades could be implemented is shown in **Figure 56.2** and includes:

- Upgrade of the existing 1.2m diameter pipe; and
- Construction of a new drainage line directly from the sag point into the railway reserve.

As discussed in Section 10.2.1, most stormwater systems (upgraded or not) are unlikely to have sufficient capacity to convey all events up to and including the PMF. Therefore, there will still be a need to ensure overland flows can be safely conveyed through the area. Therefore, there may be benefits in reducing any urban overland flow impediments such as solid Colorbond™ type fencing to ensure overland flows can more freely drain from the area (refer **Figure 56.2**).

As discussed in Section 10.2.1, any mitigation works that are completed across this area has the potential to impact on flooding across the Somerset Street "hot spot". However, two major culvert and pipe system are located between Cox Avenue and Somerset Street which serve as major hydraulic controls. These structures and the storage afforded behind the railway and Great Western Highway embankments will likely serve to attenuate any increase in flows from Cox Avenue. It may also be possible to provide additional storage within the railway reserve to further reduce any potential impacts.

As shown in **Plate 40**, there is evidence of debris accumulation within the existing drainage system, which is likely to be impeding the performance of the existing system. Therefore, in the short term, it is recommended that maintenance is performed on the existing drainage infrastructure to ensure it can operate at optimum efficiency.

10.2.3 Chapman Gardens to Railway, Kingswood

As shown in **Figure 19.3** and **19.5**, significant inundation is also predicted across Chapman Gardens as well as along the Great Western Highway during the 1% AEP flood. Fortunately, Chapman Gardens comprises sporting fields and areas of open space. However, properties on either side of the Highway, which includes both commercial and residential buildings are predicted to be significantly impacted by floodwaters.

An embankment was constructed around the north-eastern corner of Chapman Gardens in 2009 to help attenuate downstream flows. **Figure 19.3** shows that significant storage is afforded behind the embankment at the peak of the 1% AEP flood. Nevertheless, the computer modelling completed for this study indicates that the embankment would be overtopped at the spillway location (near the corner of the Great Western Highway and Cosgrove Crescent) during a 20% AEP event. **Plate 42** also shows that water is predicted to overtop the embankment approximately 150 metres further to the west of the main spillway.

Once the Chapman Gardens spillway or embankment is overtopped, water is predicted to travel east along the highway. Some of this flow continues to travel east along the highway inundating the front yards of existing residential properties on the southern side of the highway. Some water is also predicted to spill through the gap in the highway median strip (located at the Cosgrove Crescent intersection) and travel north into the car dealerships located on the northern side of the highway.

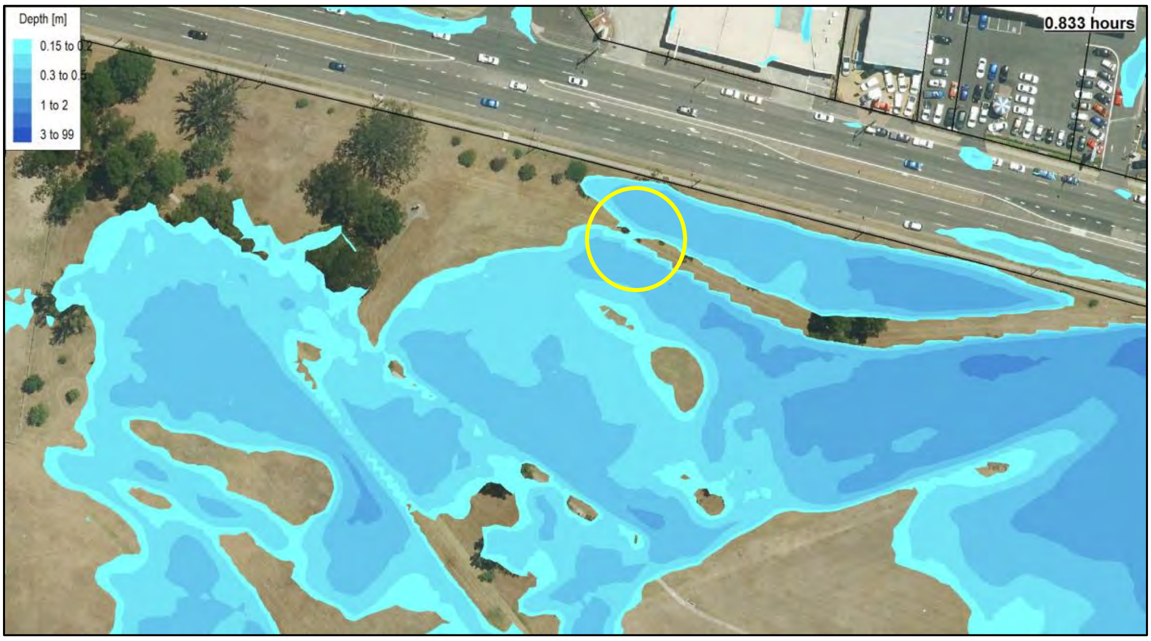


Plate 42 1% AEP Depths in vicinity of Chapman Gardens after 50mins of rainfall showing embankment overtopping location 40 minutes after the start of rainfall.

The flows from Chapman Gardens and the highway ultimately drain into either Orth Creek (located east of the car dealerships) or College Creek (located approximately 200 metres east of Chapmans Gardens). College Creek and Orth Creek meet approximately 100 metres north of the highway (forming Werrington Creek) before draining towards the railway line. The railway embankment serves as a major hydraulic control with the railway culvert having only a limited hydraulic capacity. Therefore, during large floods, water in excess of the capacity of the culvert "ponds" behind the railway embankment and inundates a significant land area.

Figure 42.3 indicates that the stormwater system in this area has sufficient capacity to convey a 1 in 2 year ARI event. According, during events greater than the 2 year ARI, the excess flow is expected to fill the storage areas contained within Chapman Gardens. During events greater than the 20% AEP event, water is predicted to overtop the Chapmans Gardens embankment, potentially inundating properties adjoining the Great Western Highway and resulting in significant inundation between the highway and railway.

Potential Mitigation Options

Chapman Gardens already affords some flood mitigation benefits. However, the benefits are only significant during smaller floods. Therefore, mitigation options would need to focus on providing increased mitigation during larger floods. The location of potential mitigation options that have been identified to achieve this outcome are shown in **Figure 56.3** and include:

- Elevating the existing spillway and embankment to provide additional storage in the north-eastern corner of Chapman Gardens.
- Inclusion of additional embankments elsewhere across Chapman Gardens to create "cascading" basins and distribute the flood storage across a larger area. This could be supplemented with earthworks to lower existing sporting fields and provide additional storage.

- Lowering of the Great Western Highway to help ensure the majority of overland flows are contained within the roadway and directed towards College Creek and not into adjoining properties. This would increase the depth and frequency of inundation across the highway and would also cause disruption to traffic during construction which may reduce the feasibility of this option.
- Increasing the size of the main culvert outlet structure to help ensure a greater proportion of flow is conveyed below ground. However, as noted in Sections 8 and 9, the downstream railway culvert is very sensitive to changes in flows. Therefore, care will need to be taken to ensure any increase in conveyance does not adversely impact on properties near the railway line.
- Lowering the vacant land between the highway and railway line to provide additional storage capacity.
- Upgrading the railway culvert to increase conveyance capacity (however, care will need to be taken to ensure downstream properties are not adversely impacted).

10.2.4 Railway Street, Landers Street and Walker Street, Werrington

Figure 19.6 shows that a part section of Werrington located south of the railway and north of Walker Street is predicted to be exposed to water depths approaching 1 metre at the peak of the 1% AEP event. In general, velocities across this area are not predicted to exceed 2 m/s. Nevertheless, the significant depths of inundation indicate that there is potential for property damage during significant rainfall events.

Drainage in this area is hampered by the railway embankment, which is generally elevated at least 1 metre above the ground surface elevations between Railway Street and Walker Street. Consequently, when the capacity of the existing 1.2 metre diameter pipe that drains runoff north beneath the railway line is exceeded, the excess runoff "builds up" behind the embankment inundating the upstream properties. The limited capacity of this pipe is partly contributed to by the downstream pipe system which only has a 1 in 2 year ARI capacity.

The capacity of the street drainage system across Railway Street and Walker Street is also limited. **Figure 42.6** shows that the stormwater pipe system in this area has less than a 1 in 2 year ARI capacity. As shown in **Plate 43**, the limited capacity of the stormwater system is predicted to result in significant ponding depths in Railway Street 45 minutes after the start of rainfall during the 1% AEP event. **Plate 43** also shows that the existing detention basin located to the south of the railway is only storing a small volume of runoff at this point in the flood.

Potential Mitigation Options

Finding measures that can reduce the impact of flooding across this area is hampered by elevated water levels to the north of the railway line (which "backs up" the pipe system and prevents it from draining). Therefore, even if larger or additional stormwater pipes were installed across this northern part of the catchment, any beneficial impact may be negated if water fills these pipes prior to the peak of the flood arriving. Nevertheless, upgrading the existing pipe beneath the railway as well as the downstream pipe system (refer **Figure 56.4**) may assist in draining this area during events with no coincidental South Creek flooding and/or when the flood gates are fully operational (which would prevent floodwaters "backing up" the pipe system).

Plate 43 1% AEP Depths in vicinity of Railway Street after 45mins of rainfall.

As discussed, the railway embankment provides a significant flow impediment. Therefore, reducing the height of the embankment would allow water from Railway Street to more readily overtop the embankment, thereby reducing upstream ponding depths. However, this would reduce the level of service provided by the railway and would cause significant disruptions to rail services during construction. Therefore, it is unlikely to be feasible.

An existing culvert (located near north-west of Railway Street – refer **Plate 43**) currently drains runoff from the north side of the railway into an open channel on the south side of the railway. Blocking and/or redirecting flow from this culvert so that runoff is retained on the northern side of the railway may assist in reducing flood impacts across the southern side of the railway line.

As discussed, the existing detention basin located south of Walker Street appears to have capacity to store additional runoff during the early stages of the flood. Therefore, there may be opportunities to reduce the flows exiting this basin (e.g., through installation of an orifice plate on the pipe outlet). There may also be opportunities to increase the storage volume provided by this basin by increasing the height of the existing basin wall (**Figure 56.4**).

Finally, if drainage upgrade options are found to be unfeasible and existing design flood levels cannot be reduced, the potential to raise low lying dwellings could be investigated. A preliminary review of the most significantly impacted buildings in the area indicate that they are typically older style, clad buildings that may be suitable for voluntary house raising.

11 CONCLUSION

This report documents the outcomes of investigations completed to quantify overland and mainstream flood behaviour across the College, Orth and Werrington 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 direct rainfall computer model that was developed using the TUFLOW software. The computer model included a full representation of the stormwater drainage system and all bridges and culverts. Major overland flow impediments including buildings, fences and road and rail embankments were also included in the model.

The computer model was validated using historic rainfall and reported descriptions of flood behaviour that were provided by the community for floods that occurred in 2010, 2011 and 2012. 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 1 in 2-year ARI flood as well as the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods. 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 South Creek. Flooding east of John Oxley Drive is typically dominated by South Creek backwater levels while flooding west of John Oxley Drive and south of the railway line is typically dominated by runoff from the local catchment.
- 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 less than 2 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.
- The catchment incorporates older and newer subdivisions. The newer subdivisions (e.g., Caddens) have been designed to modern engineering design standards with frequent storms being conveyed by the stormwater system and flows in excess of the capacity of the stormwater system being conveyed along roadways. Accordingly, minimal flooding issues have been identified across the newer subdivisions. However, across the older subdivision, the stormwater capacity is limited (typically only having capacity to convey the 1 in 2 year ARI event. Therefore, during large storms, considerable overland flow is predicted which discharges through a number of

properties. Hazard and velocity mapping prepared as part of the study indicates that flow velocities may exceed 2 m/s along some of these overland flow paths, which may pose a danger to adults, young children and the elderly.

- Inundation of over 1,000 properties is predicted at the peak of the 1% AEP flood (out of a total of 5,896 properties located within the catchment). The most notable flooding "hot spots" include:
  - Jamison Road to Bringelly Road, Kingswood
  - Somerset Street to Bringelly Road, Kingswood
  - Cox Avenue, Kingswood
  - Chapman Gardens to the main western railway line, Kingswood
  - Railway Street, Landers Street and Walker Street, Kingswood
- 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. It is recommended that these measures be assessed in detail as part of the floodplain risk management study.

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13 GLOSSARY

annual exceedance probability (AEP)	the chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. Eg, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger events occurring in any one year (see ARI).
Australian Height Datum (AHD)	a common national surface level datum approximately corresponding to mean sea level.
average annual damage (AAD)	depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
average recurrence interval (ARI)	the long-term average number of years between the occurrence of a flood as big as or larger than the selected event. For example, floods with a discharge as great as or greater than the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
catchment	the land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
disaster plan (DISPLAN)	a step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	the rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
effective warning time	The time available after receiving advice of an impending flood and before floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	a range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.

flash flooding	flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves and their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	the remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	is synonymous with flood prone land, i.e., land susceptible to flooding by the PMF event. Note that the term flood liable land covers the whole floodplain, not just that part below the FPL (see flood planning area).
flood mitigation standard	the average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	the measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	a management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at state, division and local levels. Local flood plans are prepared under the leadership of the SES.
flood planning area	the area of land below the FPL and thus subject to flood related development controls.

flood planning levels (FPLs) are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans.

flood proofing a combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.

flood prone land land susceptible to flooding by the PMF event. Flood prone land is synonymous with flood liable land.

flood readiness Readiness is an ability to react within the effective warning time.

flood risk potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below.

existing flood risk**:** the risk a community is exposed to as a result of its location on the floodplain.

future flood risk: the risk a community may be exposed to as a result of new development on the floodplain.

continuing flood risk: the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.

flood storage areas those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.

floodway areas those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.

freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.

hazard
a source of potential harm or a situation with a potential to cause loss. In relation to this study the hazard is flooding which has the potential to cause damage to the community.

Definitions of high and low hazard categories are provided in Appendix L of the *Floodplain Development Manual* (*2005*).

historical flood
a flood which has actually occurred.

hydraulics
term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.

hydrograph
a graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.

hydrology
term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.

local overland flooding
inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.

local drainage
smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.

mainstream flooding
inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.

major drainage
councils have discretion in determining whether urban drainage problems are associated with major or local drainage. Major drainage involves:

- the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or
- water depths generally in excess of 0.3m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or
- major overland flowpaths through developed areas outside of defined drainage reserves; and/or
- the potential to affect a number of buildings along the major flow path.

mathematical / computer models
the mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.

minor, moderate and major flooding

Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood.

minor flooding: Causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded.

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

major flooding: Appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.

peak discharge

the maximum discharge occurring during a flood event.

probable maximum flood (PMF)

the PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.

probable maximum precipitation (PMP)

the PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.

probability

A statistical measure of the expected chance of flooding (*see annual exceedance probability*).

risk

chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.

runoff

the amount of rainfall which actually ends up as streamflow, also known as rainfall excess.

stage

equivalent to water level (both measured with reference to a specified datum).

stage hydrograph

a graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.

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

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

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

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