PENRITH CITY COUNCIL

Rickabys Creek Catchment Flood Study

Final Report

Volume 1.1: Report

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Catchment Simulation Solutions

PENRITH CITY COUNCIL

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The preparation of the study was steered by Penrith City Council Floodplain Risk Management Committee whose members include councillors, council staff, community representatives and representatives from state agencies and adjacent councils. The study is the culmination of many months of investigation, analysis and flood modelling, which has been supported by valuable contributions from representatives of the Floodplain Risk Management Committee, community of Penrith and Penrith City Council.

It has been prepared by incorporating contributions from individuals from the local community and key stakeholders. Contributions from members of the Floodplain Risk Management Committee have been essential to the formation of management strategies that have been considered as part of the Study and are greatly appreciated. The collegial manner in which input has been provided to the project from representatives from the Penrith City Council has been critical to its success.

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 '*Flood Risk Management Manual' (NSW Government, 2023)*.

Under the Policy, the management of flood liable land remains the responsibility of Local Government. However, the State Government provides specialist technical advice to assist Local Government in its flood management responsibilities and subsidies to councils to complete the flood management process including implementation of flood mitigation works, if feasible, to alleviate existing problems.

The Policy provides for technical and financial support by the State Government through the flood risk management process which is outlined below.

The *Rickabys Creek Catchment Flood Study* represents the second 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

The '*Rickabys Creek Catchment Flood Study*' covers an area of 74 km2 within the Penrith City Council Local Government Area (LGA). As shown in **Figure 1**, the catchment includes the suburb of Londonderry, as well as sections of Agnes Banks, Castlereagh, Cranebrook, Llandilo and Berkshire Park.

Although much of the study area is occupied by open space, parkland and forested areas, there are also extensive residential and rural residential properties as well as scattered commercial and industrial developments. The higher density residential areas are contained towards the south of the catchment and are drained by a sub-surface stormwater system. Across the balance of the study area, runoff is typically drained via roadside swales to one of the drainage gullies / watercourses. This includes Torkington Creek which drains most of the northern sections of the study area and Rickabys Creek which drains most of the southern parts of the study area. Torkington Creek joins Rickabys Creek at Londonderry before Rickabys Creek continues to drain in a north-easterly direction and into the Hawkesbury River near Windsor.

During periods of heavy rainfall in the local catchment there is potential for the capacity of the local stormwater system and roadside swale system to be exceeded, leading to local overland flooding. There is also potential for "mainstream" flooding because of water overtopping the banks of the major creeks in the study area, as well as "backwater" inundation from the Hawkesbury River. Accordingly, the study area can be impacted by multiple different types of flooding.

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

Furthermore, several roads within the catchment serve as part of regional evacuation routes for the Hawkesbury-Nepean Valley (the regional evacuation routes are also shown on **Figure 1**). Therefore, flooding of these roads has the potential to impact on evacuation and emergency response across the broader Hawkesbury-Nepean floodplain.

Penrith City Council has engaged Catchment Simulation Solutions to prepare a flood study for the Rickabys Creek catchment. It documents flood behaviour across the catchment for a range of historic and design floods. This includes information on flood discharges, levels, depths and flow velocities. It also provides estimates of the variation in flood hazard and hydraulic categories across the catchment and provides an assessment of the potential impacts of climate change on existing flood behaviour.

The flood study comprises two volumes:

- Volume 1 (this document): contains the report text
- Volume 2: contains all figures and maps.

2 DATA COLLECTION AND REVIEW

2.1 Overview

A range of data were made available to assist with the preparation of the Rickabys Creek Catchment 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.

2.2 Previous Reports

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

The *'Penrith Overland Flow Flood "Overview Study"'* report was prepared by Cardno Lawson Treloar Pty Ltd for Penrith City Council. 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 flood 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 (based on the 1987 version of Australian Rainfall and Runoff) 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 greater 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 Rickabys Creek subcatchment was modelled using a 9 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 sub-surface stormwater system. Therefore, all flows within the urbanised sections of the model were assumed to travel overland. Although the Rickabys Creek catchment comprises a relatively small stormwater system, the overland flow estimations are considered to be conservative across the built-up sections of the catchment.

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 (this included 22 structures within the Rickabys Creek catchment). Therefore, flood behaviour around culverts and bridges may not be reliably represented (particularly near smaller culverts and bridges that were not represented in the model).

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 categorised into 10 percentile bands. The Overview Study identified some subcatchments within the Rickabys Creek catchment falling within the highest 20% of flood affected subcatchments across the LGA.

2.2.2 Torkington Creek, Londonderry - Flood Investigations (2013)

The *'Torkington Creek, Londonderry - Flood Investigations'* report was prepared by Molino Stewart Pty Ltd for Penrith City Council. The study was commissioned following a significant flood on 9 February 2012 within the Torkington Creek catchment that caused widespread damage.

The report notes the following flood impacts during the 2012 flood:

- 8 homes were inundated above floor level
- 19 people required rescue from floodwaters
- 38 people self-evacuated
- Several roads were closed including regional evacuation routes
- The SES received 288 calls for assistance across the broader Penrith area
- The Penrith LGA was declared a natural disaster area by the State Government

The report notes that much of the flooding occurred because of a 3-hour burst that commenced at 7pm on 9 February. During this period around 115 mm of rain fell. When compared to intensity-frequency-duration information published by the Bureau of Meteorology, this equates approximately to a 1% AEP rainfall event.

The study prepared an approximate inundation extent based on surveyed flood levels collected by Council following the flood as well as information provided by the community (a total of 29 flood levels were captured from these combined sources). The peak 1% AEP flood extent generated as part of the *'Penrith Overland Flow Flood "Overview Study"'* (2006) were superimposed and the surveyed flood levels were compared to the 1% AEP flood levels from the overland flow study. This suggested that the 2012 flood may have been larger than the design 1% AEP flood across some sections of the Torkington Creek catchment. The location of the surveyed flood marks is shown on **Figure 2**.

The report also includes details of the following major culverts (the location of these culverts is shown in **Figure 2**). This includes dimensions as well as invert estimates.

- Isaac Smith Channel crossing of Hinxman Road
- Sheredan Road channel crossing of Hinxman Road
- Nutt Road crossing
- Torkington Road crossing
- Londonderry Road crossing (see also **Plate 1**)

Plate 1 Photo of Londonderry Road culvert catpured on 9 February 2012

The study concluded that the constructed drainage system has very limited capacity (the above culverts were determined to have less than a 5% AEP capacity). Filling across private properties was also determined to have likely impacted on the severity of flooding.

2.2.3 Hawkesbury-Nepean Valley Regional Flood Study (2019)

The *'Hawkesbury-Nepean Valley Regional Flood Study'* was prepared by WMAwater for Infrastructure NSW. The objective of the project was to provide an updated description of flood behaviour across the Hawkesbury-Nepean Valley, which is considered one of the most exposed floodplains in Australia. As discussed, the Rickabys Creek catchment drains into the

Hawkesbury River near Windsor and the northern parts of the study area can be impacted by backwater flooding from the Hawkesbury River.

The regional flood study built upon the 1996 Nepean River Flood Study and included an updated flood frequency analysis. Hydrology was also defined using a RORB hydrologic model that was calibrated and validated using rainfall and stream flow information for 7 historic floods.

The quasi 2-dimensional RUBICON hydraulic model that was developed for the 1996 Flood Study was used and updated as part of the study to define flood hydraulics. The model extended as far upstream as Camden; however accurate modelling begins downstream of Bents Basin. The RUBICON model was calibrated and verified using historic information for 10 flood events.

A Monte Carlo modelling framework was used to define design flood hydrology as part of the study. This Monte Carlo approach was implemented to better represent the observed variability in actual flood events. Variables that were sampled as part of this assessment included rainfall intensity and frequency, spatial pattern of rainfall, temporal pattern of rainfall, initial losses, pre-burst rainfall, Warragamba Dam drawdown, relative timing of tributary inflows and tides. This approach is very computationally intensive and is not often completed as part of most flood studies. However, given the high potential flood risk, this more comprehensive approach was considered necessary.

The study provides design flood levels, extents, depths, provisional hazard and hydraulic categories for the 1 in 5, 1 in 10, 1 in 20, 1 in 50, 1 in 100, 1 in 200, 1 in 500, 1 in 1000, 1 in 2000, 1 in 5000 AEP floods as well as the PMF.

The study also generated information on the rate of rise, time to rise, rate of fall, time to fall, time above critical levels and travel time for key locations on the floodplain to assist in assessment of risk to life and inform emergency response. Climate change induced rainfall intensity increases, and sea level rise sensitivity analysis was also undertaken.

2.2.4 Hawkesbury-Nepean River Flood Study (2024)

The *'Hawkesbury-Nepean River Flood Study'* was prepared by Rhelm and Catchment Simulation Solutions for the NSW Reconstruction Authority. The study builds on the foundations laid by the *'Hawkesbury-Nepean Valley Regional Flood Study'* (WMA Water, 2019) and included development of a fully 2-dimensional hydraulic model of the river system and floodplain.

A summary of peak design backwater levels for Rickabys Creek at Blacktown Road (i.e., adjacent to the northern end of the study area) is provided in **Table 1**. These levels can be useful in defining the downstream tailwater conditions within the Hawkesbury River for the design flood simulations.

Table 1 Peak Hawkesbury River backwater levels for Rickabys Creek at Blacktown Road

Design Flood	**Peak Water Level** (mAHD)
1 in 5 AEP	9.6
1 in 10 AEP	12.0
1 in 20 AEP	13.8
1 in 50 AEP	15.9
1 in 100 AEP	17.4
1 in 200 AEP	18.5
1 in 500 AEP	20.2
1 in 1,000 AEP	21.3
1 in 2,000 AEP	22.8
1 in 5,000 AEP	24.4
PMF	30.6

2.2.5 Hawkesbury Nepean Floodplain Management – Emergency Flood Evacuation Route Options for Richmond and Londonderry (1994)

This study was prepared by NSW Public Works for the Roads and Traffic Authority (now Roads and Maritime Services) to identify the potential for local catchment and backwater flooding to impact on evacuation from the Hawkesbury-Nepean floodplain. This information, in turn, could be used to inform two potential evacuation route options including where road/drainage upgrades would be required.

The study provides road overtopping depths reported by the SES. These range from 0.1 to 0.5 meters across Nutt Road to more than 1 metre across parts of Londonderry Road. However, no details are provided regarding when these depths were observed.

The report also includes details of 23 drainage structures/culverts. This includes dimensions, upstream inverts, lengths, and slopes. The location of each structure in shown on **Figure 2**. However, it is understood that some of these structures have been replaced over the past 30 years, most notably, those located along Londonderry Road.

An XP-RAFTS hydrologic model of the Rickabys Creek catchment was established as part of the project. This was used to simulate rainfall-runoff processes for the 20% AEP, 10% AEP, 5% AEP, 2% AEP and 1% AEP storms based on the 1987 version of Australian Rainfall and Runoff. No calibration/validation of the model was completed due to a lack of historic data.

The analysis confirmed that many culverts appear to be under-sized with overtopping of many roads occurring as frequently as a 20% AEP event. Overtopping of the road can last for around 1 hours to more than 20 hours at some locations, depending on the duration of the storm.

Overall, this report provides useful details on existing culverts, although most of the Londonderry Road culverts have since been upgraded (refer Section 2.6). Furthermore, the design flood results have since been superseded by the 2019 version of Australian Rainfall and Runoff.

2.2.6 Trunk Drainage Londonderry (1985)

This study was prepared by J. G. Nelson Pty Ltd on behalf of the Land Commission of NSW to support the potential for urban expansion across the North-West sector, which included the suburb of Londonderry and surrounds. The investigation was commissioned as flooding and drainage were identified as a key constraint that may limit where development could occur.

The study includes the size of 137 drainage structures (this included both Rickabys Creek as well as South Creek structures). However, no invert elevations are provided.

The study was supported by hydrologic modelling using the XP-RAFTS software, which was used to define 1% AEP flows for existing, proposed and ultimate catchment development conditions. This determined that for existing catchment conditions (at that time), the peak 1% AEP discharge for Rickabys Creek at Blacktown Road was 295 m^3/s. The extent of inundation at the peak of the 1% AEP flood was also estimated based on Manning's calculations at several locations across the Rickabys Creek catchment.

The study identified a number of problematic flooding locations. These included:

- extensive areas in the western part of the catchment
- swamps along The Driftway and Jockbett Road
- several points along Londonderry Road and The Northern Road

The study also noted that flooding problems could be expected at most road crossing as the existing culvert capacities appear to be inadequate.

Unfortunately, this study uses an outdated version of Australian Rainfall and Runoff (i.e., 1977 version) and very simplified hydraulic calculations. Further, as this study is now nearly 40 years old, there is uncertainty regarding whether the details of hydraulic structures have been modified during this period. Therefore, it is considered of limited use as part of the current study.

2.3 Hydrologic Data

2.3.1 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 and sub-daily rainfall records in the vicinity of the study area are available dating back to 1811 (Richmond - UWS Hawkesbury gauge).

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Table 2 Available rain gauges in the vicinity of the Rickabys Creek catchment

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Catchment (km)
67039	Ajana	Daily	BOM	01/01/1963	01/01/1964	4
67002	Castlereagh (Castlereagh Rd)	Daily	BOM	30/08/1939		5.4
67021	Richmond - UWS Hawkesbury	Daily	BOM	01/01/1811		5.8
67021	Richmond - UWS Hawkesbury	Sub-Daily	BOM	01/01/1811		5.8
567159	Mount Pleasant (Cranebrook Reservoir)	Sub-Daily	SW	13/08/1991		6
67106	Berkshire Park First Rd	Daily	BOM	29/06/1992	30/03/1995	6.4
567085	Richmond STP	Sub-Daily	SW	13/08/1991		7.4
67113	Penrith Lakes AWS	Sub-Daily	BOM	29/08/1995		7.5
567155	Shanes Park (International Transmitter Station)	Sub-Daily	SW	23/08/1991		7.9
67025	St Marys Mwsdb	Daily	BOM	30/01/1947	29/12/1973	8.3
63248	Grose Wold Rd	Daily	BOM	30/07/1969	28/06/1994	8.5
67033	Richmond RAAF	Daily	BOM	29/09/1928	29/10/1994	8.5
67033	Richmond RAAF	Sub-Daily	BOM	01/01/1953	29/10/1994	8.5
567087	St Marys STP	Sub-Daily	SW	01/01/1991		8.5
567098	North Richmond WPS (Hawkesbury River)	Sub-Daily	BOM	14/05/1999		8.6
567107	Penrith STP	Sub-Daily	SW	31/08/1991		8.7
67116	Willmot (Resolution Ave)	Daily	BOM	29/09/1995	1/01/1998	9.3
67116	Willmot (Resolution Ave)	Sub-Daily	BOM	21/05/1999		9.3
563146	Winmalee STP	Sub-Daily	SW	01/01/1991		9.6
567047	Penrith (Nepean River)	Sub-Daily	BOM	20/05/1999		10
67031	Windsor Bowling Club	Daily	BOM	30/07/1897		10.2
63247	Glenara	Daily	BOM	01/01/1899	01/01/1969	10.3
67018	Penrith Ladbury Avenue	Daily	BOM	01/01/1890	01/01/1994	10.8
67004	Emu Plains	Daily	BOM	01/01/1880	01/01/1973	10.9
67054	Windsor The Peninsula	Daily	BOM	29/06/1861	29/12/1916	11
63286	Winmalee (Pentlands Drive)	Daily	BOM	01/01/1985		11.3
67096	Glenroy	Daily	BOM	1/01/1917	1/01/1923	11.5
63105	Grose Vale	Daily	BOM	1/01/1954	1/01/1971	11.7
67024	St Marys Bowling Club	Daily	BOM	29/06/1897	29/12/1984	11.8
67067	Emu Plains	Daily	BOM	01/01/1911	01/01/1996	12.3
63078	Journeys end	Daily	BOM	01/01/1946	01/01/1956	12.5
67083	Mount Druitt Francis Street	Daily	BOM	29/11/1970	29/12/1976	13
63183	Valley Heights (Sun Valley Rd)	Daily	BOM	30/08/2002		13.1
67118	Oakhurst (Lawton Place)	Daily	BOM	27/02/1991	30/05/1999	13.2
67003	Colyton (Carpenter St)	Daily	BOM	29/09/2000		13.5
63120	Freemans Reach (Dorothy St)	Daily	BOM	30/10/1960	08/11/2000	13.5
63042	Kurrajong Post Office	Daily	BOM	30/10/1932	28/04/1991	13.5

Gauge Number	Gauge Name	Gauge Type	Source*	Start of Records	End of Records	Distance from Catchment (km)
67101	Box Hill Junction Rd	Daily	BOM	30/05/1985	29/12/1995	13.9
67115	Glenmore Park (Cartwright Place)	Daily	BOM	01/01/1995		14.3
67030	Glen Rock	Daily	BOM	30/10/1938	29/12/1966	14.5
63077	Springwood (Valley Heights)	Daily	BOM	01/01/1883		14.6
63077	Springwood (Valley Heights)	Sub-Daily	BOM	29/04/1951	28/06/1961	14.6
63185	Glenbrook Bowling Club	Daily	BOM	01/01/1963		14.8
63230	Blaxland Western Highway	Daily	BOM	29/04/1968	29/12/1980	15
67037	Schofields Boundary Rd	Daily	BOM	01/01/1963		15

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

2.3.2 Water Level Data

There are no stream gauges located within the Rickabys Creek catchment. Accordingly, no water level information could be uncovered for the study area. However, there is a stream gauge located on the Hawkesbury River near Windsor (Gauge 212903) just downstream of the Rickabys Creek outlet. This gauge could be useful for defining downstream water levels for historic/validation simulations

2.4 Topographic and Survey Information

2.4.1 LiDAR

LiDAR data was captured across Sydney between April and July 2019 by the NSW Government's Land and Property Information Department. This included the full extent of the Rickabys Creek 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 4.0 elevation points per square metre.

A digital elevation model (DEM) was developed from the 2019 LiDAR information and is shown in **Figure 4**. It shows that although the very southern sections of the catchment (e.g., around Cranebrook) are located as high as 70 mAHD, most of the northern parts of the catchment are located between 10 and 30 mAHD. Therefore, there is very little topographic relief across the northern parts of the catchment. The DEM also show evidence of significant earthworks across Agnes Banks because of quarrying activities. The DEM does not include any landform changes post 2019 (including illegal filling which has been noted as occurring).

To validate the accuracy of the LiDAR, the elevations from the DEM shown in **Figure 4** were compared against 91 survey marks (state survey marks and permanent survey marks). This comparison showed that the average difference between LiDAR elevations and survey mark elevations was -0.02 metres with a standard deviation of 0.23 metres. The LiDAR elevations were most commonly lower than the ground survey points, particularly across the urbanised southern section of the catchment. This was most commonly a result of the survey mark being located on the top side of the kerb while the LiDAR often picked up the adjacent gutter (i.e., a typical difference of -0.15 m).

It was noted that LiDAR differed from the survey marks by more than 0.5 metres at 3 locations. A further review of these locations showed that the survey mark was obscured by tree canopy. Therefore, the survey mark comparison indicates that the LiDAR provides reasonable vertical accuracy in areas of limited vegetation cover but affords less accuracy in areas obscured by dense vegetation.

To further understand the relative density of LiDAR ground points, **Plate 2** provides an example of the LiDAR point density in the vicinity of Torkington and Nutt Roads. **Plate 2** shows a high LiDAR point density across grassed and roadway areas (i.e., areas clear of trees). **Plate 2** also shows no ground points across buildings (indicating non-ground points have been correctly removed). **Plate 2** does confirm reduced point density in the vicinity of dense vegetation. Therefore, there will be a less detailed representation of the variation in terrain in areas of dense vegetation.

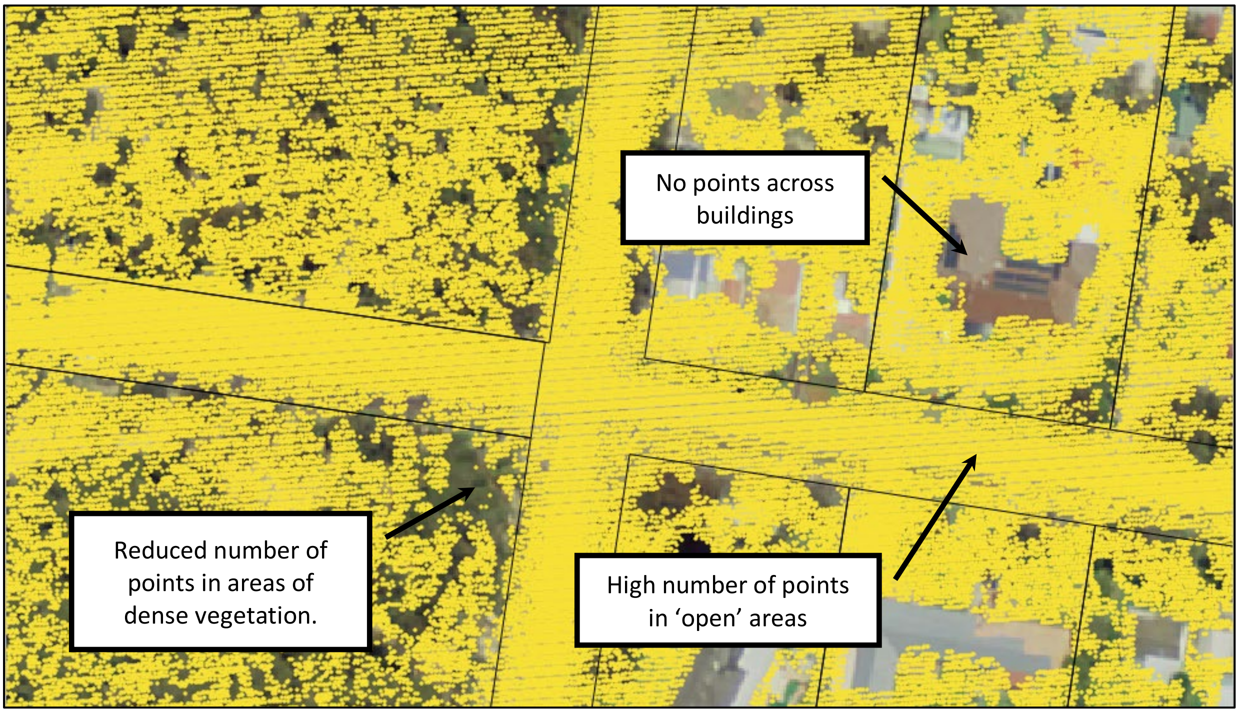

Plate 2 LiDAR data points at intersection of Torkington and Nutt Roads

To confirm the maximum potential resolution of a DEM developed from the LiDAR, the LiDAR ground point densities were extracted from the raw LiDAR information and this information is included on **Plate 3**. This shows that the point density across areas of open space (e.g., roads, grass) is most commonly above 2 ground points per square metre (in some areas, the point density exceeds 4 points per square metre). However, in areas of dense vegetation coverage (e.g., western sections of the study area), the point density is generally contained between 1 and 2 ground points per square metre. Therefore, there appears to be sufficient point density to support a minimum DEM resolution of at least 1 metre across the full study area.

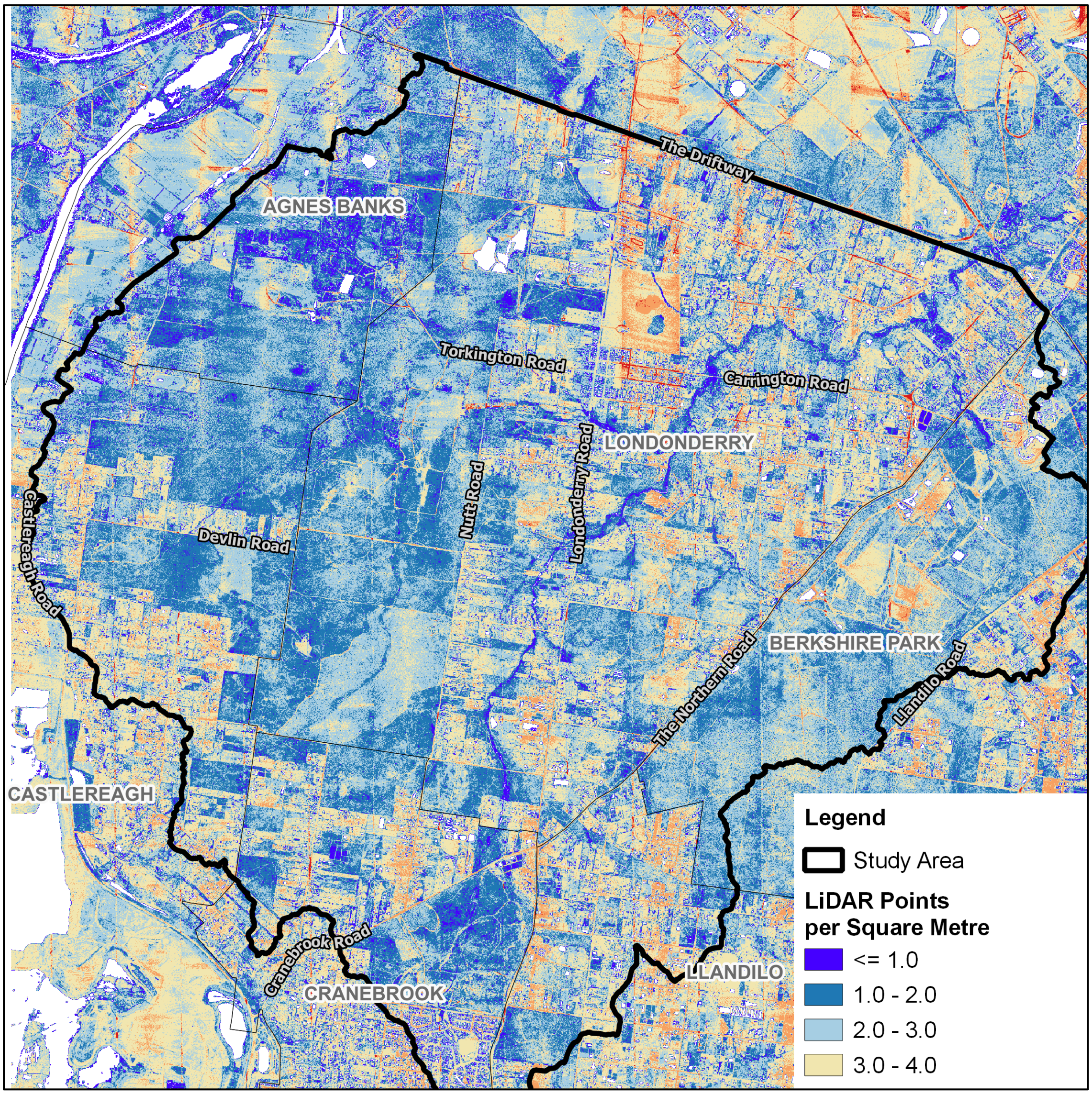

Plate 3 LiDAR ground point density

However, as noted above, the reliability of the LIDAR reduces in areas of tree canopy. This includes many of the watercourses draining through the study area. Therefore, to ensure the best representation of these major conveyance areas is provided, ground survey of the main creek lines is necessary. This is discussed further in Section 2.8.

2.4.2 Hydraulic Structure Survey

Cardno Lawson Treloar (CLT) were engaged by Penrith City Council to collect details of hydraulic structures across the LGA in 2020. This largely involved field measurements where structures were accessible or estimates of structure sizes where access was limited. The location of the structures that were captured are shown in **Figure 2**.

The data collected by CLT included dimensions for 22 drainage structures within the Rickabys Creek catchment. However, there are no details on invert elevations and the measured dimensions are not considered to be of a survey standard. However, the dimensions are considered suitable for flood estimation purposes and when combined with invert estimates from LiDAR, can be used to provide a reasonable description of these structures in the hydraulic model.

However, there are more than 180 culverts without any details, including around 70 culverts that are likely to have a significant influence on local flood behaviour. Therefore, there is insufficient information contained in this dataset to fully define all culverts within the catchment, even when considered in conjunction with data contained on engineering plans (refer Section 2.6). Therefore, survey of culverts was required as part of the project. Further details on the culvert survey are provided in Section 2.8.

2.5 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
- Cadastre
- Roads
- SES Regional Evacuation Routes
- Zoning
- Stormwater pits (within the catchment from both from Penrith City Council and Hawkesbury City Council)
- Stormwater pipes (within the catchment from both from Penrith City Council and Hawkesbury City Council)

Most of the above layers served as a basis for preparing the various figures displaying the study results (most notably the aerial photography and cadastre). In this regard, the layers are considered fit for purpose.

However, the stormwater layers served as key inputs in the development of the hydraulic model for the study. As a result, these layers were reviewed in detail and the outcome of this review is provided below.

2.5.1 Stormwater Pits

The extent of the stormwater pits layer is shown in **Figure 2**. It included a total of 1,353 data points. Of these, 741 were stormwater inlets/pits and the balance were headwalls.

A review of the GIS layer indicated some limitations. This included:

- 246 pits did not include an invert elevation. Of the remaining pits that included invert elevations, 75 invert elevations were higher than the DEM level at the pit location.
- 460 pits did not include information describing the size of the lintel/kerb opening.
- 444 pits included no grate size information.

Therefore, more than half the stormwater inlets that are represented in the GIS layer do not contain sufficient information to fully define their capacity and/or may contain unreliable data. Therefore, it was necessary to collect survey of existing stormwater pits to ensure they could be appropriately represented in the flood model. Further details on the stormwater pit survey are included in Section 2.8.

Stormwater pits provided for Hawkesbury City Council were not reviewed as they are not anticipated to be used as part of this study.

2.5.2 Stormwater Pipes

The extent of the stormwater pipes layer is shown in **Figure 2**. It included a total of 1,029 pipes. This included 12 box culverts and 1,017 circular pipes.

A review of the pipes GIS layer identified the following limitations:

- 705 pipes contained no pipe diameters. However, a “pipe configuration” field was populated for 289 pipes that would allow pipe diameters to be extracted. The remaining pipes with no diameter information are shown in yellow in **Figure 2**.
- 454 pipes did not contain upstream and downstream invert elevations.
- Some pipes were digitised in the wrong direction (the hydraulic model requires pipes to be digitised from upstream to downstream) (refer **Plate 4**).

Plate 4 Example of incorrect pipe direction

Those pipes that were digitised in the incorrect direction were manually updated. For the balance of the pipes, survey was completed to ensure the conveyance afforded by the stormwater system was represented accurately. Further details on the stormwater pit survey are included in Section 2.8.

Stormwater pipes provided for Hawkesbury City Council were not reviewed as they are not anticipated to be used within the hydraulic model developed as part of this study and will only be used to help inform the contributing catchment draining into the study area from the Hawkesbury LGA.

2.6 Engineering Plans

Penrith City Council provided design and work-as-executed plans for several locations within the study area. The plans are summarised below and provide details on culverts, drainage swales as well as stormwater pits and pipes at several locations across the catchment:

- New Richmond Bridge and Traffic Improvements, Stage 1 – The Driftway, Detailed Design (2023)
- The Driftway: Upgrade of The Driftway from Londonderry Road (MR630) to Blacktown Road (MR537) (2022)
- Proposed Table Drain at 6-12 Mills Road, Londonderry (2018)
- Proposed Drainage Works at Muscharry Road, Londonderry (2018)
- Drainage Improvement at Eighth Avenue and Second Avenue, Llandilo (2017)
- Drainage Improvement Works at Reynolds Road, Londonderry (2016)
- Drainage Improvement Works at Devlin Road and Rickards Road, Castlereagh (2016)
- Proposed Drainage Upgrade at the Intersection of Fifth Avenue and Northern Road (2016)
- Kenmare Road, Londonderry: Road, Drainage and Kerb Gutter Works (2014)
- Proposed Drainage Works at Hinxman Road, Castlereagh (2014)
- Church Street, Castlereagh: Proposed Box Culvert Drainage Upgrade (2014)
- Drainage Improvement from 135-201 Rickards Road, Castlereagh (2012)
- Flood Security Upgrading: Londonderry Road Culvert Site L4 (1999)

The locations covered by the engineering plans are shown in **Figure 2**.

2.7 Land Use

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

Land use zoning was also used to assist in assigning broad-scale land uses/material across the study area. For example, the "environmental" and "national park" zoned areas were assigned a background land use of "long grass" while "residential" and "rural residential" areas were assigned a background land use of "short grass". The more detailed information extracted from the LiDAR (e.g., tree canopy and buildings) where subsequently "stamped" on top pf these base land use layers.

The final land use map that was developed is shown in **Figure 5**.

2.8 Survey

2.8.1 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, culverts and the stormwater network. Consulting surveyors, Metropolis City Surveyors, collected the additional survey information.

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

2.8.2 Hydraulic Structures

The details of sixty-nine (69) culverts were also collected as part of the survey. The location of each culvert that was surveyed is shown on **Figure 6**.

The survey collected key attributed of each culvert 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.

2.8.3 Creek Cross-Sections

As discussed in Section 2.4.1, LiDAR can provide a less reliable description of the variation in terrain in areas of dense vegetation, including the major creeks within the study area. Therefore, cross-sections were surveyed at key locations along the major creeks where vegetation was prevalent to ensure a reliable description of the conveyance capacity of these waterways could be provided in the computer model.

As previously discussed, cross-sections were collected of the upstream and downstream channel at all culverts that were surveyed. An additional two (2) cross-sections were collected in areas away from culverts for a total of 140 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 assigning roughness coefficients to the hydraulic model.

2.8.4 Stormwater Network

Survey of all stormwater pits (and connecting pipes) within the catchment was completed. This involved the survey of 484 pits and all pipes connecting to these 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.

2.9 Community Consultation

2.9.1 General

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

As discussed in Section 2.2.2, flood marks for the 2012 flood are available within the Torkington Creek catchment. However very little historic flood data is available across the remainder of the study area. It was considered that the community may be able to provide information on past floods to assist with the computer model validation. Therefore, several community consultation devices were developed to inform the community about the study and to obtain information from the community about their past flooding experiences. Further information on each of these consultation devices is provided below.

2.9.2 Community Information Brochure and Questionnaire

A community information brochure and questionnaire were prepared and distributed to owners of all residential and business properties in the catchment. This resulted in brochures and questionnaires being distributed to approximately 3,000 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 157 questionnaire responses were received. A summary of all questionnaire responses is provided in **Appendix A**.

The responses to the questionnaire indicate that:

- The majority of respondents have lived in or around the catchment for at least 10 years with a large proportion residing in the area for over 30 years.
- Nearly 50% of respondents have experienced some form of inundation or disruption as a result of flooding in the study area. This includes (also refer **Plate 5**):
  - 64 respondents have experienced traffic disruptions;
  - 39 respondents have had their front or back yard inundated;
  - 13 respondents have had their garage inundated; and,
  - 4 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 in multiple questionnaire responses:
  - Clark Rd/Reynolds Rd, Londonderry
  - Torkington Road
  - Nutt Road
  - Smeeton Road
- A number of respondents believe inundation in the catchment is exacerbated by:
  - Blockage of the creek, stormwater inlets and/or drains (51 respondents)
  - Limited capacity of the existing stormwater system (48 respondents)
  - Insufficient creek capacity (42 respondents)
  - Overland flow obstructions (e.g., fences, buildings) (18 respondents)
  - Illegal filling/diversion of flow within private properties or major developments in the Hawkesbury Nepean floodplain (6 respondents)

- Respondents were able to provide descriptions of flood behaviour and/or provide photos for the past floods:
  - March 2022 (29 respondents)
  - March 2021 (10 respondents)
  - February 2020 event (10 respondents)
  - January 2016 (1 respondent)

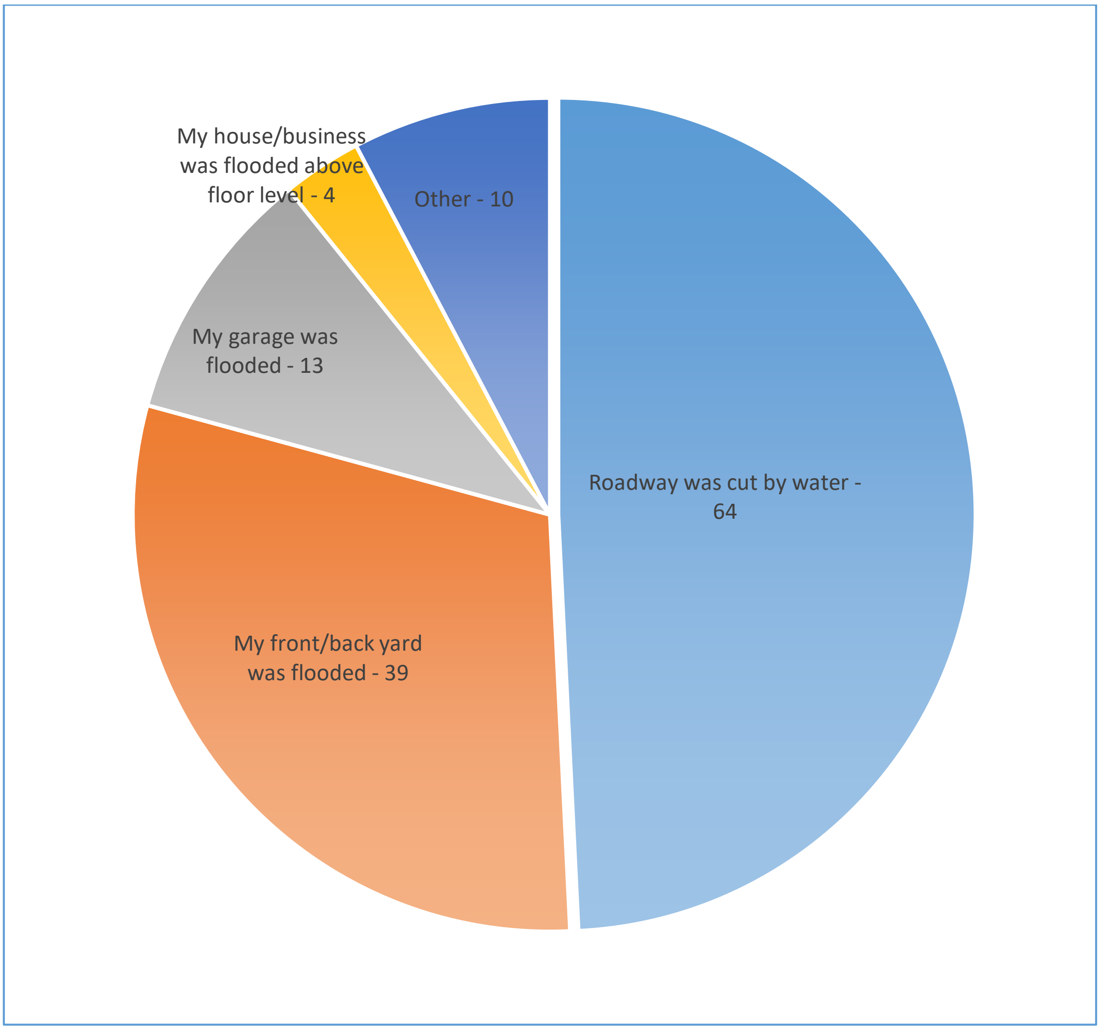

Plate 5 Type of Flood Impact Reported by Questionnaire Respondents

2.9.3 Public Exhibition

The draft '*Rickabys Creek Catchment Flood Study'* was placed on Public Exhibition from the 17 July 2025 until 14 August 2025. A copy of the draft report was made available for review on Council's www.yoursaypenrith.com.au website during the public exhibition period.

A community workshop was held during the exhibition period (on 30 July 2025) at the Andromeda Community Centre, where copies of the draft report and posters were available for viewing, and a presentation outlining the study process and outcomes was provided. Following the presentation, the community was given an opportunity to ask questions/raise concerns. The workshop was attended by no less than 57 community members.

A total of twenty (20) submissions were received during the public exhibition period. The submissions, as well as the comments that were raised during the community workshop, generally related to the following issues:

- Maintenance of Council stormwater assets.
- Impact to insurance/rates/house prices.
- Illegal filling worsening flood behaviour.

Overall, no modifications to the draft '*Rickabys Creek Catchment Flood Study'* were required to address the submissions received.

3 COMPUTER FLOOD MODEL

3.1 General

Computer models are the most common method of simulating flood behaviour through a particular area of interest. They can be used to predict flood characteristics such as peak discharges, flood level and flow velocity.

Two computer models were developed to simulate flood behaviour across the Rickabys Creek catchment:

- A hydrologic model was developed to simulate the transformation of rainfall into runoff across the catchment. The hydrologic model was developed using the WBNM software.
- A hydraulic model was developed to simulate how the runoff from the hydrologic model would be distributed/move across the catchment. The hydraulic model was developed using the TUFLOW software.

The following sections describe the model development process.

3.2 WBNM Model Development

3.2.1 Subcatchment Delineation

The Rickabys Creek catchment was subdivided into a number of smaller subcatchments to provide a detailed description of the spatial variation in hydrologic characteristics across the catchment. The subcatchments were delineated with the assistance of the CatchmentSIM software (Catchment Simulation Solutions, 2018) using a 2 metre Digital Elevation Model (DEM) that was developed based on the 2019 LiDAR dataset described in Section 2.4.1.

The catchment includes urbanised as well as rural/natural bushland areas. As such, the subcatchment delineation was altered to suit each of these land uses (i.e., more detailed subcatchments in urban areas and a lower density of subcatchments in rural areas). In urbanised areas, the subcatchment delineation targeted incorporating subcatchment outlets once the contributing catchment area reached approximately 1 to 2 hectares to ensure all potential overland flow paths were captured. In rural and natural bushland areas, a contributing catchment area of roughly 20 hectares was targeted.

A total of 1,208 subcatchments were ultimately delineated across the catchment, noting again that the catchment extends beyond the study area towards the north into the Hawkesbury LGA. The final subcatchment layout is presented in **Figure 7**.

3.2.2 Subcatchment Parameterisation

Key hydrologic properties including area and impervious proportion were calculated automatically for each subcatchment using CatchmentSIM in conjunction with the remote sensing land use information shown in **Figure 5**. As shown in **Figure 5**, the catchment was subdivided into eight (8) different land use types:

- Buildings
- Waterbodies
- Creeks
- Trees
- Long Grass
- Short Grass
- Roadways
- Gravel.

Percentage impervious was assigned to each land use (refer **Table 4**) and was used to calculate weighted average percentage impervious for each subcatchment. The adopted subcatchment parameters are summarised in **Appendix B**.

Table 3 Adopted Impervious Percentage for Hydrologic Model

Land Use Description	Impervious (%)
Buildings	100
Waterbodies	100
Creeks	100
Trees	0
Long Grass	0
Short Grass	0
Roads	100
Gravel	100

A lag parameter of 1.6 was adopted for all subcatchments (the recommended lag parameter documented in Boyd et al, 2017).

Effective Impervious Area

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

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

Accordingly, Book 5 of ARR2019 advocates the use of EIA when modelling urbanised portions of catchments to ensure urban runoff volumes and peak flows are not overestimated. Although ARR2019 presents a range of approaches for estimating the EIA, the most straight forward approach is estimating the EIA as a percentage of the TIA. Section 3.4.2.2 of Book 5 outlines that EIA will typically be 50% to 70% of the TIA. That is, only 50% to 70% of the total impervious area is directly connected to the drainage system. The remaining 30% to 50% of the impervious area is, therefore, indirectly connected and has additional infiltration opportunity.

For this study, the 70% adjustment factor (i.e., the most conservative factor) was adopted for all urban subcatchments (i.e.: subcatchments covering the urbanised areas of Cranebrook and higher density residential area of Londonderry). For these areas, the total impervious areas that were calculated for each subcatchment were multiplied by 0.70 to develop a revised "EIA version" of the model.

3.2.3 Stream Routing

The sub-catchment area, percentage impervious and lag factor that are input into the WBNM model are used by the model to simulate the transformation of rainfall into runoff for each subcatchment. In addition to local subcatchment runoff, most sub-catchments will also carry flow from upstream catchments along the main watercourses.

WBNM incorporates a non-linear routing calculation to account for routing of flows along watercourses within each subcatchment. A stream lag adjustment factor of 1.00 is recommended for natural channels. However, this factor can be adjusted for individual subcatchments to reflect alternate channel types that might route flows more efficiently (e.g., roadway gutters).

Each watercourse within the catchment was reviewed and the most common channel type within each subcatchment was identified, and the appropriate lag adjustment factor shown in **Table 5** was assigned to each subcatchment. For urbanised sections of the catchment with no formal watercourse, most of the flow would be carried via stormwater pipes and/or along the gutter. This was considered to be analogous to a concrete lined channel.

Table 4 Adopted stream lag factors

Watercourse Description	Lag Factor
Natural	1.00
Excavated	0.50
Concrete lined	0.33
Urban/gutter	0.33

The stream lag factors that were adopted for each subcatchment are provided in **Appendix B**.

3.2.4 Rainfall Loss Model

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

To account for rainfall 'losses' of this nature, the hydrologic model incorporates a rainfall loss model. For this study, the 'Initial-Continuing' loss model was adopted, which is recommended in *'Australian Rainfall & Runoff - A Guide to Flood Estimation'* (Ball et al, 2019). This loss model assumes that a specified amount of rainfall is lost during the initial saturation/wetting of the catchment (referred to as the 'Initial Loss'). Further losses are applied at a constant rate to simulate infiltration/interception once the catchment is saturated (referred to as the 'Continuing Loss Rate'). The initial and continuing losses are deducted from the total rainfall over the catchment, leaving the residual rainfall to be distributed across the catchment as runoff.

The specific rainfall losses that were employed in the WBNM model were determined as part of the model calibration process. Further details on the model calibration are provided in Chapter 4.

3.2.5 Detention Areas

The catchment includes a number of major depression areas and water bodies. Most of these areas are associated with farm dams or quarry works, however a more formal basin is located in Cranebrook on the southern side of Vincent Road. Farm dams were assumed to be full at the start of the storm events and therefore would not provide any attenuation. However, the quarry works and basin on Vincent Road, Cranebrook have the potential to attenuate downstream flows by temporarily storing runoff. Therefore, it was considered important to include a representation of these flood detention areas in the WBNM model.

The representation of detention areas in WBNM requires the storage and basin outlet characteristics of the basin to be defined. The storage characteristics were defined using a water level versus storage volume relationship that was extracted from CatchmentSIM based upon the LiDAR DEM. For detention areas with a permanent water body (e.g., Hi-Quality Group quarry tailings dams), it was assumed that storage volume was only provided above the permanent water level.

The outflow characteristics were specified by defining the elevation and dimensions of the overflow weir as well as the details of any outlet culverts/pipes (e.g., invert elevation, size and number of conduits).

A total of six (6) detention areas were included in the WBNM model. The location of all detention areas that were represented is shown in **Figure 7**.

3.3 TUFLOW Model Development

3.3.1 Model Extent

A 2-dimensional hydraulic computer model of the catchment was developed using the TUFLOW Highly Parallelised Computer (HPC) software (version 2020-10-AD). TUFLOW is a fully dynamic, 1D/2D finite difference model developed by BMT WBM (2012). It is used extensively across Australia to assist in defining flood behaviour.

The extent of the TUFLOW model area is shown in **Figure 8**. As shown in **Figure 8**, the TUFLOW model extends across the full extent of the catchment, extending downstream of Blacktown Road (the Penrith LGA boundary). Areas of the Hawkesbury LGA that contribute runoff to the Rickabys Creek catchment were also included. The model covers an area of 93km^2.

3.3.2 Grid Size and Topography

The TUFLOW software uses a grid to define the spatial variation in topography and hydraulic properties (e.g., ground elevations and surface roughness) across the model area. As a result, 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 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 detail and simulation time.

In addition, a TUFLOW feature called sub-grid sampling (SGS) was employed as part of the model setup. When SGS is employed, TUFLOW will calculate water level versus storage volume relationships based on a more detailed underlying terrain representation rather than relying on a single elevation at the centre of the grid cell. Similarly, TUFLOW will calculate water level versus discharge relationships across each cell side based on the more detailed terrain rather than relying on the elevation at the midpoint of each cell to control when water moves from one cell to the next. This feature allows storage and conveyance to be represented in more detail than would have otherwise been allowed. The 1 metre DEM derived from the LiDAR described in Section 2.4.1 was used for this purpose.

3.3.3 Roughness Values

The TUFLOW software uses land use information to define the hydraulic roughness for each grid cell in the model. The remote sensing information described in Section 2.7 was used as the basis for defining the variation in land use across the TUFLOW model (refer **Figure 5**). This land use information, in turn, was used as the basis for assigning the variation in hydraulic roughness values across the model area. The roughness coefficient values adopted within the TUFLOW model for the various land use types are listed in **Table 6**.

Table 5 TUFLOW Roughness Coefficients

Land Use Description	Roughness
Buildings	1.000
Waterbodies	0.040
Creeks	0.060
Trees	0.080
Long Grass	0.050
Short Grass	0.035
Roads	0.022
Gravel	0.018

3.3.4 Building Representation

The Rickabys Creek catchment incorporates some localised urban areas. This urbanisation creates many overland flow obstructions. One of the most significant impediments 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 watertight (Smith et al, 2012). Accordingly, it was considered necessary to include a representation of the obstruction afforded by buildings in the computer model.

The lower part (i.e., the area between the ground surface and the floor level) of each residential building was represented as a complete flow obstruction. A 0.3 metre height was selected as being representative for most residential dwellings (reflecting two steps up to the front door typical of older style homes across the wider catchment).

Once the water level exceeds the floor level of each building, it was allowed to “enter” the building. A high Manning’s “n” value of 1.0 was adopted to reflect the significant impediment to flow afforded by the many flow obstructions contained with a typical house (e.g., walls, doors, furniture etc).

3.3.5 Watercourses

During floods, most of the flow is conveyed within each of the major watercourses/creeks. Therefore, it is important to provide a reliable representation of the conveyance provided along each of these watercourses.

To achieve this, TUFLOW ‘gully’ lines were included within the model to enforce the watercourse alignment in areas where the LiDAR derived DEM did not provide a sufficiently detailed representation. This was achieved by digitising ‘gully’ lines along watercourses, with elevations assigned along each line based on elevations extracted from the survey cross-sections and plans. This was supplemented with LiDAR ground points where the vegetation canopy did not obstruct the creek. The extent of the watercourse/gully lines in shown in **Figure 8**.

3.3.6 Culverts

All known/major culverts within the study area were represented within the TUFLOW model as 1D hydraulic structures. The location of culverts that were included within the TUFLOW model is shown in **Figure 8**.

The dimensions and invert elevations of the culverts were included directly in the TUFLOW model. The culvert information was populated based on the following sources (in order of priority):

- Culvert survey (refer Section 2.8.3)
- work-as-executed plans provided by Council (refer Section 2.6)
- *‘Penrith CBD Detailed Overland Flow Flood Study’ (2015)*
- *‘Torkington Creek, Londonderry - Flood Investigations’ (2013)*
- ‘*Hawkesbury Nepean Floodplain Management – Emergency Flood Evacuation Route Options for Richmond and Londonderry’ (1994)*
- *‘Trunk Drainage Londonderry’ (1985)*
- Assumed dimension (discussed below).

Details from the survey, plans or previous studies outlined accounted for 181 culverts throughout the catchment, with the remaining 200 culverts having no known details (note that these were typically minor culverts away from major watercourses). For these culverts, an assumption regarding the culvert size and number of lines was based on the context of the culvert location (i.e.: size of a nearby known upstream/downstream culvert, or designation with a standard culvert size based on peak preliminary 1% AEP flood depths at the crossing).

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

Blockage

During a typical flood, sediment, vegetation, and urban debris (e.g., litter, fence palings, bins) from the catchment can become mobilised leading to blockage of downstream culverts. Consequently, 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 were calculated for all culverts. The blockage factors were calculated based on guidelines contained in *'Australian Rainfall and Runoff: A Guide to Flood Estimation'* (Ball et al, 2019). The blockage calculations are summarised in **Appendix C** for each culvert located within the TUFLOW model area.

3.3.7 Stormwater System

The stormwater system has the potential to convey a significant proportion of runoff across the urbanised portion of the study area during more frequent rainfall events. Therefore, it was important to incorporate the stormwater system in the TUFLOW model to ensure the interaction between piped stormwater and overland flows was reliably represented across the urbanised sections of the catchment.

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

The properties of the stormwater system (e.g., pit types/sizes/inverts, pipe diameters/inverts) were fully defined from the detailed stormwater survey collected as part of this study (refer Section 2.8.4). The extent of the stormwater system included within the TUFLOW model is shown in **Figure 8**.

Inlet capacity curves were then 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 D**. A total of 155 different pit inlet capacity curves were developed.

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

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

Blockage

Like culverts, stormwater inlets can become partially blocked during significant rainfall events because of urban debris being mobilised by runoff. To account for the reduced inlet capacity that is afforded by blockage, blockage factors were applied to all stormwater pits in the TUFLOW model. The stormwater pit blockage factors were assigned based on Council’s current pit blockage policy, which are summarised in **Table 7.** These blockage factors 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 6 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%

3.3.8 Detention Areas

As discussed in Section 3.2.5, the Rickabys Creek catchment incorporates a number of major depression areas and water bodies that may attenuate downstream flows during rainfall events. Therefore, a representation of each storage was included in the TUFLOW model.

The absence of any water level monitoring gauges within each waterbody 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. If an outlet structure was present from the waterbody/dam, the water level was assigned to the invert of the outlet structure.

4 COMPUTER MODEL CALIBRATION AND VALIDATION

4.1 Overview

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

Calibration is typically completed by routing recorded rainfall from historic floods through the hydrologic model and comparing simulated flows against recorded flows at stream gauge locations. Unfortunately, there are no stream gauges located within the catchment. Therefore, it was not possible to complete a full calibration of the hydrologic model developed for this study.

However, as discussed in Section 2.2.2, surveyed flood marks were collected after the February 2012 flood. Moreover, there are several rainfall gauges located within close proximity to the catchment. Therefore, it was possible to jointly calibrate the computer models for the February 2012 event by routing recorded rainfall from the nearby gauges through the hydrologic model. The flows from the hydrologic model can then be applied to the hydraulic model and simulated floodwater levels can be compared against the surveyed flood levels.

Descriptions of flood behaviour as well as photographs for the February 2020, March 2021 and March 2022 events were also provide by the community as part of the community consultation. As such, the computer models could then be validated for the February 2020, March 2021 and March 2022 events using the information provided by the community.

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

4.2 February 2012 Flood

4.2.1 WBNM Modelling

Rainfall

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

Accumulated daily rainfall totals for each rainfall gauge that was operational during the February 2012 event were used to develop a rainfall isohyet (i.e., rainfall depth contour) map for the event, which is shown in **Figure 9**. The rainfall totals for two 'backyard' gauges were also included due to their ability to capture the localised nature of rainfall that occurred within the western parts of the catchment. Details of the location and rainfall depths from these gauges were extracted from the *'Torkington Creek, Londonderry - Flood Investigations'* (Molino Stewart, 2013).

The isohyet map indicates that there was some significant spatial variation in rainfall across the catchment during the February 2012 event. It indicates that over 180mm fell within the western portion of the study area, however, less than 60mm fell across the north-east of the study area near The Driftway. In recognition of the variation in rainfall across the catchment during this event, all rainfall gauges that were active during the event were incorporated into the WBNM model. The total rainfall depths that were applied to each subcatchment as part of the simulation is summarised in **Appendix F**.

The temporal (i.e., time-varying) distribution of rainfall for the non-continuous gauges (i.e.: Castlereagh Road, and the two 'backyard' gauges) was determined based on the closest continuous rainfall gauge that provided reliable recorded rainfall information for the event. This was determined to be the Penrith AWS (Gauge #67113) which is located to the south-west of the catchment.

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information for the catchment. This information is presented as **Figure E1** in **Appendix E** and indicates that the February 2012 event produced rainfall that varied from between a 5% AEP event, to greater than a 1% AEP event during its most intense period. However, it should be noted that the rainfall across the overall catchment varied greatly, and this rainfall intensity was isolated to the south-western corner of the catchment only with a much less severe rainfall intensity occurring across the north-eastern portion of the catchment.

Rainfall Losses

As discussed in Section 3.2.4, the initial-continuing loss model was employed to represent rainfall losses across the catchment. The following rainfall losses were adopted for pervious sections of the catchment for the February 2012 simulation:

- Initial loss = 10mm (based on conservative estimate of initial losses from Australian Rainfall and Runoff 2019)
- Continuing loss rate = 1.36 mm/hour (based on the continuing loss rate of 3.40mm/hr downloaded from ARR2019 Data Hub with NSW jurisdictional advice adjustment applied i.e., 0.4 multiplier).

For effective impervious surfaces, an initial loss of 0mm was adopted along with a continuing loss rate of 0 mm/hour (i.e., impervious surfaces fully contributed to runoff).

Results

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

The hydrographs generated by the WBNM model were subsequently applied to the TUFLOW model. Further discussion on the TUFLOW model simulation is provided below.

4.2.2 TUFLOW Modelling

Changes to Reflect Historic Conditions

As noted earlier in this report, some culverts and stormwater infrastructure across the catchment has been upgraded over the past decade. As a result, details of these hydraulic structures were reverted to their dimensions at the time of the February 2012 event if sufficient details were available from the engineering plan sets provided.

Inflow Boundary Conditions

Discharge hydrographs generated by the WBNM hydrologic model were used to define inflows to the TUFLOW model. The location of these are shown on **Figure 8**.

Downstream Boundary Conditions

Hydraulic computer models also require the adoption of a suitable downstream boundary condition 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 Rickabys Creek and the Hawkesbury River. Accordingly, the water level across the downstream reaches of the catchment will be driven by the prevailing water level along the Hawkesbury River at the time of the flood.

There is a stream gauge located on the Hawkesbury River, near to the confluence of Rickabys Creek (SCA Gauge 212903). Therefore, water level information from this stream gauge was extracted for the February 2012 event and was used to define the downstream boundary condition for the historic flood simulation. This was applied as a water level versus time relationship and reflected the change in water level within the Hawkesbury River during the 2012 flood. The peak level recorded at the gauge was 2.19m AHD which falls well below the Rickabys Creek channel invert at the downstream end of the study area (~4.5m AHD) and indicates that flood behaviour throughout the study area was driven by local catchment runoff only.

Blockage

As noted in Chapter 3, there is potential for drainage infrastructure (e.g., culvert, stormwater pits) to become partly blocked by debris during a flood. Consequently, these drainage structures will typically not operate at full efficiency during floods.

As discussed in Section 3.3.6, blockage factors were calculated for all culverts based upon *'Australian Rainfall and Runoff: A Guide to Flood Estimation'* (Ball et al, 2019). The blockage calculations are summarised in **Appendix C.**

As shown in **Appendix C**, the blockage factors are assigned based on the severity of the flood with larger floods being assigned a higher blockage factor due to the potential for larger floods to increase the mobilisation of debris. Although the February 2012 rainfall peaked with an intensity more severe than a 1% AEP event, this was only in the south-western corner of the catchment and was not representative across the broader catchment, which experienced less severe rainfall intensity. Therefore, the design blockage factors were assigned based on the medium blockage scenario (i.e., blockage factors extracted from the second last column in **Appendix C**).

For stormwater pits, the blockage factors described in Section 3.3.7 were retained.

Results

Calibration of the TUFLOW computer model was attempted based upon the 29 flood marks documented in the *'Torkington Creek, Londonderry - Flood Investigations'* (Molino Stewart, 2013). It should be noted that the flood marks were all collected within the western portion of the overall Rickabys Creek catchment only (i.e.: within the Torkington Creek catchment). This was due to the higher intensity rainfall that occurred within this portion of the overall Rickabys Creek catchment and therefore resulted in more severe inundation.

Peak floodwater depths were extracted from the results of the February 2012 flood simulation and are included on **Figure 10**. A comparison between the peak flood levels generated by the TUFLOW model and the flood marks for the February 2012 flood is also provided in **Figure 10** as well as **Table 8**.

The flood mark comparison shows that the TUFLOW model produces peak flood levels that are most commonly within 0.15 metres of the flood mark, with the average absolute difference being 0.07 metres. Larger discrepancies occur at some locations, however, as noted in the 'Comments' column in **Table 8,** these appear to be less reliable flood marks (i.e., not consistent with nearby marks) and/or are based on photographs that were not provided for independent review within the *'Torkington Creek, Londonderry - Flood Investigations'* (Molino Stewart, 2013). Many of the surveyed flood marks are also located at buildings, and any uncertainty in the shape, size, or obstruction provided by the buildings can influence flood behaviour at the building.

The timing of the peak flood levels was also analysed within the locations of the surveyed flood marks and indicated that peak flood levels would have occurred at 9pm within the upper reaches of the catchment (Hinxman Road), and at 11:30pm around the Londonderry Road crossing of Torkington Creek on 09/02/2012. This agrees with anecdotal evidence provided by Council that the peak occurred at night on 09/02/2012.

A comparison between the peak flood depths generated by the TUFLOW model and one anecdotal flood depth provided by the community, and well as three flood photos provided by Council for the February 2012 flood is provided in **Appendix G** as **Table G1**. **Table G1** indicates that a variation in reported/photographed depth of up to 0.1 m is predicted, which indicates a good correlation between simulated model results and the anecdotal report and photos of flooding.

Table 7 Comparison between simulated and reported flood levels for the February 2012 flood

ID	Reported/Surveyed Flood Level (mAHD)	Simulated Flood Level (mAHD)	Difference (m)	Source	Comment
1	33.00	N/A	N/A	Photo	No inundation predicted in this location. Photo unable to be reviewed as not provided within

ID	Reported/Surveyed Flood Level (mAHD)	Simulated Flood Level (mAHD)	Difference (m)	Source	Comment
					previous study. However, ground elevation at location is 33mAHD so any inundation would have been very shallow.
2	32.80	32.83	0.03	Photo	
3	30.55	30.47	-0.08	Survey	
4	30.52	29.57	-0.95	Survey	Surveyed flood mark is ~0.5m higher than nearby marks. Flood mark may be less reliable at this location.
5	30.51	30.30	-0.21	Survey	
6	30.42	30.16	-0.26	Survey	
7	30.33	30.20	-0.13	Survey	
8	30.15	30.23	0.08	Photo	
9	29.97	29.65	-0.32	Survey	
10	29.73	29.41	-0.32	Survey	
11	19.24	19.17	-0.07	Survey	
12	19.15	19.14	-0.01	Survey	
13	19.15	19.11	-0.04	Survey	
14	19.11	19.03	-0.08	Survey	
15	19.05	19.02	-0.03	Survey	
16	19.04	18.99	-0.05	Survey	
17	19.03	19.01	-0.02	Survey	
18	18.99	18.94	-0.05	Survey	
19	18.85	18.77	-0.08	Survey	
20	18.13	18.19	0.06	Survey	
21	18.10	18.19	0.09	Survey	
22	18.08	18.19	0.11	Survey	
23	18.06	18.18	0.12	Survey	
24	18.05	18.18	0.13	Survey	
25	18.04	18.18	0.14	Survey	
26	18.04	18.18	0.14	Survey	
27	17.50	17.58	0.08	Photo	
28	17.50	17.59	0.09	Photo	
29	28.30	27.90	-0.40	Photo	Photo unable to be reviewed as not provided in previous study.

4.3 February 2020 Flood

4.3.1 WBNM Modelling

Rainfall

The February 2020 flood occurred as a result of rain falling over a 5 day period commencing on 7th February. Accumulated rainfall totals for each active rain gauge over this 5 day period were used to prepare the isohyet map shown in **Figure 11**. As shown in **Figure 11**, total rainfall depths across the catchment during this event varied between 320 and 230 mm, with higher depths within the western portion of the study area relative to the eastern portion.

All rainfall gauges that were active during the event were incorporated into the WBNM model to represent the spatial variation in rainfall event, while the temporal distribution for the non-continuous gauges (i.e., Castlereagh Road gauge) was assigned based on the Penrith AWS (Gauge #67113).

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Figure E1** in **Appendix E** and indicates that the February 2020 event produced rainfall that was between a 20% AEP and 5% AEP design rainfall event within the south-western portion of the catchment.

Rainfall Losses

An initial loss of 10 mm and a continuing loss rate of 1.36 mm/hour was adopted for pervious surfaces as per the February 2012 simulation. An initial loss of 0 mm was adopted for effective impervious surfaces with a continuing loss rate of 0 mm/hour.

Results

The WBNM model was used to simulate rainfall-runoff behaviour for the February 2020 flood based upon the rainfall and rainfall loss information presented in the preceding sections. Peak discharges for each WBNM model subcatchment for the February 2020 flood are included in **Appendix F.**

The hydrographs generated by the WBNM model were subsequently applied to the TUFLOW model. Further discussion on the TUFLOW model simulation is provided below.

4.3.2 TUFLOW Modelling

Inflow Boundary Conditions

Discharge hydrographs generated by the WBNM hydrologic model were used to define inflows to the TUFLOW model.

Downstream Boundary Conditions

The Hawkesbury River stream gauge located near to the confluence Rickabys Creek (SCA Gauge 212903) was used to describe the time variation in water level at the downstream TUFLOW model boundary for the February 2020 event. The peak level recorded at the gauge was 9.40m AHD which was sufficient to produces some backwater inundation within the lower reaches of the Rickabys Creek channel within the study area (up to roughly Kenmare Road, Londonderry).

Blockage

As the February 2020 rainfall was generally between a 20% AEP and 5% AEP event, the design blockage factors were assigned based on this frequency (i.e., low blockage scenario from the third last column in **Appendix C**).

For stormwater pits, the blockage factors described in Section 3.3.6 were retained.

Results

Validation of the TUFLOW computer model was attempted based upon thirteen (13) anecdotal reports of flood behaviour for the February 2020 event. In general, the anecdotal reports of flooding describe floodwater depths at discrete locations across the study area. However, some floodwater depths were also estimated based upon photographs of the February 2020 flood (refer **Appendix G**).

Peak floodwater depths were extracted from the results of the February 2020 flood simulation and are included on **Figure 12**. A comparison between the peak flood depths generated by the TUFLOW model and the flood depths reported by the community for the February 2020 flood is provided in **Table G2** of **Appendix G**.

The flood depth comparison provided in **Table G2** shows that the TUFLOW model generally provides a reasonable reproduction of reported floodwater depths. In most cases, the TUFLOW model produces peak depths that are within 0.1 metres of reported depths. The only significant difference occurs at response #117 where the flood photo shows a significantly lower depth than the simulated peak depth, which is likely due to the photograph not being captured at the peak of the flood at this location.

It is noted that many respondents commonly provide general comments of depth within their property, often with a broad range (e.g., 0.3-0.5 metres deep in the back of the property), without a specific location within what are typically large, rural lots. A best guess on the location has been made based on the reported flood depth (using the simulated depths as a guide) but the generalised nature of the reports can contribute to the variations between reported and simulated depths identified.

Overall, it is considered that the TUFLOW model is providing a reasonable reproduction of reported flood depths for the February 2020 flood.

4.4 March 2021 Flood

4.4.1 WBNM Modelling

Rainfall

The March 2021 flood occurred as a result of rain falling over a week long period commencing on 18th March. Accumulated rainfall totals for each active rain gauge over this period were used to prepare the isohyet map shown in **Figure 13**. As shown in **Figure 13**, total rainfall depths across the catchment during this event varied from 340mm in the north, 380mm in the south, and 280mm within the western portion of the study area.

All rainfall gauges that were active during the event were incorporated into the WBNM model to represent the spatial variation in rainfall event, while the temporal distribution for the non-continuous gauges (i.e.: Castlereagh Road gauge) was determined based on the Penrith AWS (Gauge #67113).

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Figure E1** in **Appendix E** and indicates that the March 2021 event produced rainfall that was less severe than a 20% AEP event for durations up to 40 hours. For longer durations, the rainfall intensity fell between a 20% AEP and 5% AEP design rainfall event.

Rainfall Losses

An initial loss of 10 mm and a continuing loss rate of 1.36 mm/hour was adopted for pervious surfaces as per the previous simulations. An initial loss of 0 mm was adopted for effective impervious surfaces with a continuing loss rate of 0 mm/hour.

Results

The WBNM model was used to simulate rainfall-runoff behaviour for the March 2021 flood based upon the rainfall and rainfall loss information presented in the preceding sections. Peak discharges for each WBNM model subcatchment for the March 2021 flood are included in **Appendix F.**

The hydrographs generated by the WBNM model were subsequently routed through the TUFLOW model. Further discussion on the TUFLOW model simulation is provided below.

4.4.2 TUFLOW Modelling

Inflow Boundary Conditions

Discharge hydrographs generated by the WBNM hydrologic model were used to define inflows to the TUFLOW model.

Downstream Boundary Conditions

The Hawkesbury River stream gauge located near to the confluence Rickabys Creek (SCA Gauge 212903) was used to describe the time variation in water level at the downstream TUFLOW model boundary for the March 2021 event. The peak level recorded at the gauge was 12.91m AHD which was sufficient to produces some backwater inundation within the lower reaches of the Rickabys Creek channel within the study area (up to roughly Studley Street, Londonderry).

Blockage

As the March 2021 rainfall was between a 20% AEP and 5% AEP event, the design blockage factors were assigned based on this frequency (i.e., low blockage from the third last column in **Appendix C**).

For stormwater pits, the blockage factors described in Section 3.3.6 were again retained.

Results

Validation of the TUFLOW computer model was attempted based upon fifteen (15) reports of flood behaviour for the March 2021 event. In general, the anecdotal reports of flooding describe floodwater depths at discrete locations across the study area. However, some floodwater depths were also estimated based upon photographs of the March 2021 flood (refer **Appendix G**).

Peak floodwater depths were extracted from the results of the March 2021 flood simulation and are included on **Figure 14**. A comparison between the peak flood depths generated by the TUFLOW model and the flood depths reported by the community is provided in **Appendix G** as **Table G3**.

The flood depths comparison provided in **Table G3** shows that the TUFLOW model generally provides a reasonable reproduction of reported floodwater depths. In most cases, the TUFLOW model produces peak depths that are generally within 0.1 metres of reported depths. More significant differences occur at two locations, response #155 where the respondent has stated a depth of “6 foot” for all flood events (and is, therefore, not an accurate record of the flood depths for a specific flood), and response #157 where a description of “over 1.5 metres” was used, which again, is not a very specific record of the depth.

Like the February 2020 event, many respondents provide general comments on depths and locations, and it was necessary to estimate where this information was reported, owning to the large size of many lots. As a result, this can contribute to the differences between reported and simulated depths identified.

Overall, it is considered that the TUFLOW model is providing a reasonable reproduction of reported flood levels for the March 2021 flood.

4.5 March 2022 Flood

4.5.1 WBNM Modelling

Rainfall

The March 2022 flood occurred as a result of rain falling over a 9 day period commencing on 1st March. Accumulated rainfall totals for each active rain gauge over this period were used to prepare the isohyet map shown in **Figure 15**. As shown in **Figure 15**, total rainfall depths across the catchment during this event varied between 517mm in the south of the study area to 420 mm in the north.

All rainfall gauges that were active during the event were incorporated into the WBNM model to represent the spatial variation in rainfall event, while the temporal distribution for the non-continuous gauges (i.e.: Castlereagh Road gauge) was determined based on the Penrith AWS (Gauge #67113).

The continuous rainfall information was also analysed relative to design rainfall-intensity-duration information. This information is presented in **Figure E1** in **Appendix E** and indicates that the March 2022 event produced rainfall that peaked at an intensity sufficient to be considered a 5% AEP event.

Rainfall Losses

An initial loss of 10 mm and a continuing loss rate of 1.36 mm/hour was adopted for pervious surfaces. An initial loss of 0 mm was adopted for effective impervious surfaces with a continuing loss rate of 0 mm/hour.

Results

The WBNM model was used to simulate rainfall-runoff behaviour for the March 202 flood based upon the rainfall and rainfall loss information presented in the preceding sections. Peak discharges for each WBNM model subcatchment for the March 2022 flood are included in **Appendix F.**

The hydrographs generated by the WBNM model were subsequently applied to the TUFLOW model. Further discussion on the TUFLOW model simulation is provided below.

4.5.2 TUFLOW Modelling

Inflow Boundary Conditions

Discharge hydrographs generated by the WBNM hydrologic model were used to define inflows to the TUFLOW model.

Downstream Boundary Conditions

The Hawkesbury River stream gauge located near to the confluence Rickabys Creek (SCA Gauge 212903) was used to describe the time variation in water level at the downstream TUFLOW model boundary for the March 2022 event. The peak level recorded at the gauge was 13.71m AHD which was sufficient to produces some backwater inundation within the lower reaches of the Rickabys Creek channel within the study area (up to roughly halfway between Studley Street and Cherrybrook Chase, Londonderry).

Blockage

As the March 2022 rainfall peaked at the 5% AEP event intensity, the design blockage factors were assigned based on this approximate frequency (i.e., low blockage from the third last column in **Appendix C**).

For stormwater pits, the blockage factors described in Section 3.3.6 were retained.

Results

Validation of the TUFLOW computer model was attempted based upon thirty-three (33) reports of flood behaviour for the March 2022 event. As per the 2020 and 2021 events, the flood reports included a combination of anecdotal flood information as well as flood photographs of the 2022 flood.

Peak floodwater depths were extracted from the results of the March 2022 flood simulation and are included on **Figure 16**. A comparison between the peak flood depths generated by the TUFLOW model and the flood depths reported by the community and shown in flood photos is provided in **Appendix G** as **Table G4**.

The flood level comparison provided in **Table G4** shows that the TUFLOW model generally provides a reasonable reproduction of recorded floodwater depths. In most cases, the TUFLOW model produces peak depths that are within 0.1 metres of recorded depths.

However, more significant differences do occur at several locations. However, many of these differences can be explained:

- Response #119 which states a depth of overtopping of Smeeton Road by 0.3 metres in all flood events. It is, therefore, not an accurate record of the specific depths that might have occurred during the 2022 event (however, the simulated depths match relatively well for the February 2020 and March 2021 events).
- Response #120 which states that 3 metres depth of water formed across the front of the block, while simulated depths only exceed 2 metres. A second description of flooding on the same property by this respondent matches well with simulated depths, indicating the 3-metre depth estimate may have been over-estimated by the respondent.
- Response #131 which states a depth of 0.4 metres. However, the location of this depth is not specified, and simulated depths across the property are not severe. The report may refer to depths on an adjoining local roadway, in which case the simulate depths compare well.
- Response #136 which reported a depth of 1 metre across the front of the property (the simulated depths can exceed 1.6 metres). Photographs of the same flood have also been provided for this property which are better reproduced by the model. This may indicate that the depth may have been a "ballpark" estimate only.
- Response #155 where the respondent has stated a depth of "6 foot" for all flood events. Therefore, not an accurate record of the flood depths for the 2022 flood.
- Response #157 where a description of "over 2 metres" was used, which again, is not a very accurate record of the depth.

Despite the differences listed above, the majority of the anecdotal flood reports and flood photographs are well produced by the model. Therefore, when the above uncertainties are taken into consideration, the TUFLOW model is providing a reasonable reproduction of observed flood behaviour for the March 2022 flood.

5 DESIGN FLOOD ESTIMATION

5.1 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, level or depth at a particular location is the probability that the flood flow, 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 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 Rickabys Creek catchment are outlined in the following sections.

5.2 Hydrology

5.2.1 Rainfall

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

- North of Cranebrook (WBNM subcatchment 63): 33.6984°S, 150.7249°E
- East of Castlereagh (WBNM subcatchment 301): 33.6630°S, 150.7050°E
- North-east of Cranebrook (WBNM subcatchment 984): 33.6738°S, 150.7448°E
- West of John Morony Correction Complex (WBNM subcatchment 433): 33.6520°S, 150.7679°E
- West of RAAF Base Londonderry (WBNM subcatchment 591): 33.6386°S, 150.7205°E

A copy of the design rainfall information downloaded from the Bureau of Meteorology's is included in **Appendix H**. For simplicity, the design rainfall information is also summarised in **Table 9** at the centroid of the catchment (33.6529°S, 150.7388°E). However, within the WBNM model, the IFD location that was located closest to each subcatchment was applied to that subcatchment. The location of the five IFD locations used are shown on **Plate 6**.

Table 8 Point Design Rainfall Depths at the centroid of the catchment.

Storm Duration	Rainfall Depth (mm)								
	0.5EY	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMP
5 min	8.47	10.6	12.7	14.7	17.5	19.7	21.3	24.2	-
10 min	13.6	17.2	20.7	24.2	28.8	32.5	34.9	39.5	-
15 min	16.9	21.5	25.9	30.3	36.1	40.7	43.8	49.6	120
20 min	19.4	24.6	29.6	34.6	41.2	46.4	50	56.6	-
25 min	21.3	27	32.4	37.8	45.1	50.7	54.7	62	-
30 min	22.9	28.9	34.6	40.3	48.1	54.1	58.5	66.3	170
45 min	26.3	32.9	39.4	45.8	54.5	61.4	66.5	75.4	220
1 hour	28.8	35.8	42.7	49.6	59.1	66.6	72.1	81.8	260
1.5 hour	32.6	40.1	47.7	55.3	65.9	74.4	80.5	91.3	340
2 hour	35.5	43.5	51.7	59.9	71.5	80.7	87.3	98.9	390
3 hour	40.4	49.2	58.4	67.8	81.1	91.8	98.8	112	480
4.5 hour	46.4	56.4	67.1	78.2	93.7	106	114	129	-
6 hour	51.5	62.9	75	87.6	105	120	128	144	630
9 hour	60.4	74.3	89.1	105	126	143	153	173	-
12 hour	68	84.4	102	120	145	165	176	199	-
18 hour	80.7	102	124	146	177	201	216	244	-
24 hour	91	116	142	169	204	231	250	283	-
30 hour	99.7	129	158	188	226	257	288	330	-
36 hour	107	139	171	204	246	279	317	366	-
48 hour	119	156	193	231	277	313	357	412	-

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 WBNM model. The PMP is defined as the greatest depth of rainfall that is meteorologically possible at a specific location.

PMP depths were derived for a range of storm durations up to and including the 6-hour event based on procedures set out in the Bureau of Meteorology's *'Generalised Short Duration Method'* (GSDM) (Bureau of Meteorology, 2003). The GSDM PMP calculations are included in **Appendix H** and the calculated rainfall depths are summarised in **Table 9**.

5.2.2 Areal Reduction Factors

The design rainfall intensities presented in the preceding section (as well as **Appendix H)** are only applicable for catchment areas of up to 1 km^2. Therefore, ARR2019 includes areal reduction factors that recognise that there is unlikely to be a uniformly high rainfall intensity across all sections of large catchments.

The primary input variable to calculate the areal reduction factors is the contributing catchment area. One of the main difficulties in applying the areal reductions factors for a flood study such as this is the fact that the contributing catchment area varies considerably across the study area. For example, the contributing catchment areas vary from less than 1 km^2 at the upstream end of each major subcatchment (and smaller tributaries) up to 86.9 km^2 at the downstream end of the catchment. Therefore, to fully apply the correct areal reductions factors, it would be necessary to calculate the catchment area draining to the outlet of each subcatchment, determine the reduction factor for each subcatchment then adjust the point rainfall intensities individually for each subcatchment. This would result in a significant increase in the number of design storms that need to be simulated with associated increases in simulation times and processing effort. Therefore, the selection of areal reduction factors was restricted to six focus locations across the catchment.

Focus Locations

The focus locations were chosen to represent the various sections of the overall catchment and are shown on **Plate 6**. More specifically, the focus locations were identified as follows:

Focus Location 1: Within the Cranebrook urbanised portion of the catchment to represent the critical flood behaviour within the urbanised portion of the catchment.

Focus Location 2: Just upstream of the Londonderry Road crossing of Rickabys Creek to represent the Rickabys Creek catchment at this major roadway crossing.

Focus Location 3: Just upstream of the Londonderry Road crossing of Torkington Creek to represent the Torkington Creek catchment at this major roadway crossing.

Focus Location 4: Just upstream of the Carrington Road crossing of Rickabys Creek to represent the combined Torkington and Rickabys Creek catchment at this major roadway crossing (this is located downstream of the confluence of Rickabys and Torkington Creek and so captures the interaction of the two creeks).

Focus Location 5: Just upstream of The Northern Road crossing of a smaller tributary within the Rickabys Creek catchment to represent the many natural catchment areas contributing to the main Rickabys/Torkington Creeks.

Focus Location 6: Just upstream of the Hinxman Road crossing of a smaller tributary within the Torkington Creek catchment to represent the many rural-residential catchment areas contributing to the main Rickabys/Torkington Creeks.

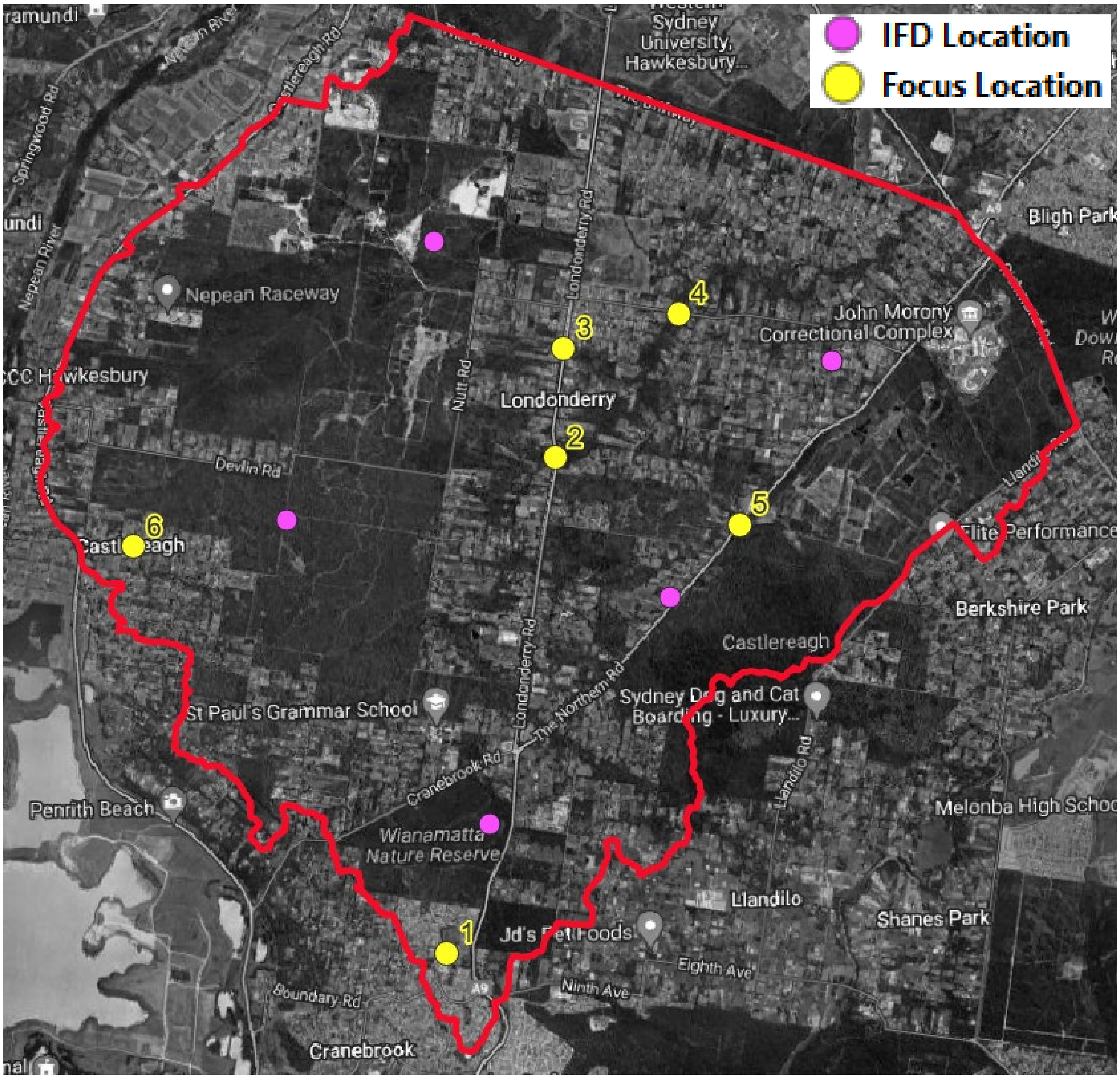

Plate 6 Location of IFD extraction locations and focus locations.

5.2.3 Rainfall Losses

As discussed, the WBNM model was developed to include the "initial-continuing loss" model to account for rainfall that is intercepted/infiltrated and does not contribute to runoff.

ARR2019 recommends a hierarchical approach for determining the most appropriate initial (i.e., burst) loss and continuing loss values to apply as part of design flood simulations. In instances where insufficient information is available to fully calibrate rainfall losses (such as this study where no stream gauge was available), the ARR2019 jurisdictional advice for NSW recommends the adoption of rainfall loss information from the ARR2019 data hub. Further information on the adopted burst losses and continuing loss rates adopted for the study are provided below.

Burst Losses

Probability neutral "burst" losses were downloaded from the ARR2019 data hub and are summarised in **Table 10**. For the Rickabys Creek catchment, this resulted in burst losses that vary between 8.4 mm and 48.3 mm.

Table 9 Adopted Burst Rainfall Losses for Pervious Surfaces

Duration	Burst Loss (mm)					
	0.5EY	20% AEP	10% AEP	5% AEP	2% AEP	≥1% AEP
<1 hour	26	18.6	16.5	16.8	15.6	12.6
1 hour	26	18.6	16.5	16.8	15.6	12.6
1.50 hour	29.4	17.5	15.1	15.0	14.8	12.9
2 hours	32.1	20.9	16.1	14.6	12.6	10.5
3 hours	35.4	19.6	15.7	15.2	14.0	10.4
6 hours	33.5	19.5	15.9	14.8	12.4	8.4
12 hours	34.4	23.4	21.5	21.0	18.5	9.7
18 hours	35.3	25.5	24.5	23.1	20.7	11.6
24 hours	37.5	29.2	28.8	28.6	25.9	17.4
36 hours	40.9	34.0	32.9	33.4	29.7	15.3
48 hours	46.2	40.6	41.5	45.8	35.0	19.0
72 hours	48.3	41.7	42.3	47.1	38.9	27.4

For impervious areas, Section 3.5.3.1.2 of Book 5 of ARR2019 recommends a storm initial loss of 1 mm. However, the storm loss of 1 mm needs to be adjusted to account for pre-burst rainfall. This yielded an impervious burst loss of 0 mm for all storm durations.

A burst/initial loss of 0 mm was adopted for all surfaces/catchment areas for the PMP simulations in line with recommendations in Chapter 4 of Book 8 of ARR2019.

Continuing Loss Rates

The ARR2019 jurisdictional advice for NSW recommends that the continuing loss rate documented on the Data Hub be reduced by 60% before application to pervious surfaces. The "raw" loss rate documented on the data hub is 3.4 mm/hr. Therefore, the adjusted loss rate is 1.36 mm/hr and was adopted for application to pervious surfaces. Note that this is the identical continuing loss applied to pervious surfaces in the calibration/validation flood events.

For impervious areas, Section 3.5.3.1.2 of Book 5 of ARR2019 recommends a continuing loss rate of 0 mm/hr. Therefore, the continuing loss rate of 0 mm/hr was adopted directly in the WBNM model.

5.2.4 Temporal Patterns

ARR2019 employs 10 different temporal patterns for each AEP/storm duration to define the time variation in rainfall during each storm. The use of a variety of different temporal patterns is intended to reflect the natural variability of a typical rainfall event (i.e., no two storms will be the same).

The temporal patterns for the study area were downloaded from the ARR data hub and were used to simulate the temporal distribution of rainfall for each design storm. Although the overall catchment has a contributing catchment area of 86.9km^2, none of the focus locations exceeds 75km^2. In accordance with ARR2019 for catchments with an area less than 75 km^2, the "point" temporal patterns rather than "areal" temporal patterns were selected to describe the temporal variation in rainfall.

ARR2019 groups the temporal patterns into "frequent", "intermediate" and "rare" bins, which were applied to each design storm as follows:

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

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

5.2.5 Results

The WBNM model was used to simulate rainfall runoff processes for the complete suite of design storms. The design 0.5EY, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and 0.2% AEP storms were simulated in addition to the PMP.

As discussed, a suite of ten temporal patterns were used to represent the temporal variation in rainfall for each design flood frequency up to and including the 0.2% AEP event. The peak discharges from the full suite of temporal patterns for each design event were reviewed to determine the critical storm duration at each focus location. The critical storm duration was defined by calculating the average discharge for each storm duration for each subcatchment (based on all 10 temporal patterns). The storm duration that generated the highest average discharge was selected as the critical duration for that subcatchment. The resulting critical storm durations for each subcatchment are presented in **Appendix I**.

Once the critical duration was determined for each AEP, a representative temporal pattern was selected for that duration. The temporal pattern that generated the peak discharge immediately above the average discharge was selected as the most representative temporal pattern for each subcatchment, and the adopted storm duration and temporal pattern for each focus location, for each flood event is shown in **Table 11**. The peak discharge generated by the representative temporal pattern is provided in **Table 12** for all focus locations, and within **Appendix I** for all subcatchments.

Table 10 Critical Durations and Temporal Patterns at Focus Locations.

Focus Location	Description	Critical Storms								
		0.5EY	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
1	Corner Cassar Cres and Andromeda Dr, Cranebrook (Urban Area)	10min (TP4388)	60min (TP4578)	20min (TP4444)	20min (TP4444)	20min (TP4371)	20min (TP4367)	20min (TP4367)	20min (TP4404)	15min
2	US Rickabys Ck crossing of Londonderry Rd	1080min (TP4846)	270min (TP4711)	180min (TP4663)	180min (TP4663)	120min (TP4617)	120min (TP4611)	120min (TP4611)	120min (TP4617)	90min
3	US Torkington Ck crossing of Londonderry Rd	1440min (TP4877)	360min (TP4737)	360min (TP4696)	360min (TP4591)	720min (TP4785)	720min (TP4751)	720min (TP4751)	720min (TP4751)	120min
4	US Carrington Rd (DS confluence of Tork and Rickabys Ck)	1440min (TP4877)	360min (TP4737)	360min (TP4696)	360min (TP4660)	720min (TP4785)	720min (TP4751)	720min (TP4751)	720min (TP4751)	120min
5	US The Northern Rd (Rickabys Ck) - tributary crossing	540min (TP4770)	180min (TP4677)	90min (TP4594)	90min (TP4594)	90min (TP4532)	60min (TP4559)	60min (TP4559)	60min (TP4559)	90min
6	US Hinxman Rd (Torkington Ck upper reaches)	540min (TP4770)	90min (TP4605)	90min (TP4593)	90min (TP4593)	60min (TP4405)	45min (TP4528)	45min (TP4534)	45min (TP4534)	45min

Table 11 Peak Design Discharges and Key Locations.

Location		WBNM ID	Peak Discharge (m³/s)								
			0.5EY	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.2%AEP	PMP
Rickabys Creek	Vincent Rd (northern end of Cranebrook) - Rickabys Creek	39	2.63	5.47	6.62	9.37	15.2	21.3	23.18	30.2	68.5
	Smeeton Rd - Rickabys Creek	980	13.8	23.3	33.7	42.6	53.7	66.8	75.55	90.8	393
	Londonderry Rd - Rickabys Creek	314	31.2	49.8	66.1	83.2	125	143	155	179	1056
	Londonderry Rd - Xing of SE Tributary of Rickabys Ck	203	10.3	16.2	22.3	28.7	41.6	47.7	51.5	59.2	341
	An eastern tributary of Rickabys Creek (near Sutherland Rd) - Xing of The Northern Road	238	4.58	8.24	12.4	16.0	19.8	24.1	27.8	33.3	141
	An eastern tributary of Rickabys Creek (near Carrington Rd) - Xing of The Northern Road	466	7.2	13.2	20.0	25.2	32.4	40.1	44.7	53.3	213
	Rickabys Ck crossing of Carrington Rd	1021	97.1	132	191	222	280.3	412	445	513	2916
	Rickabys Ck crossing of Blacktown Rd	1	138	177	253	329	416	480	516	704	4170
	Downstream Londonderry Rd crossing of north-western tributary of Rickabys Creek	637	4.48	7.28	10.2	13.3	17.8	20.3	22.4	27.0	134
	Blacktown Rd crossing of north-western tributary of Rickabys Creek (inflows from Hawkesbury LGA)	999	21.9	36.6	52.3	67.2	86.1	102	115	139	614
	The Driftway crossing of northern tributary of Rickabys Creek (inflows from Hawkesbury LGA)	695	9.88	17.1	26.4	33.4	40.9	52.6	58.7	69.5	252
Torkington Creek	Downstream Smeeton Rd - Torkington Ck	163	5.81	10.3	15.5	19.9	24.1	30.7	34.1	41.2	178
	Devlin Rd crossing of Torkington ck	302	11.9	18.8	26.2	34.0	47.3	54.1	58.4	69.0	402
	Devlin Rd crossing of south-western tributary of Torkington Ck	315	6.1	12.1	17.2	21.5	27.4	35.0	39.4	47.4	171
	Torkington Rd crossing of Torkington Creek	520	43.2	68.2	94.7	111	145	206	222	257	1383
	Londonderry Rd crossing of Torkington Ck	1026	51.3	71.3	101	117	148	219	237	274	1500

The results of the hydrologic analysis shown in **Table 11** indicate that the critical duration across the study area can vary greatly, with shorter duration events (between 10 minutes and 60 minutes) being critical at focus location 1 which represents the urbanised portion of the catchment, and storm durations of up to 1440 minutes being critical for focus locations 3 and 4 which represent Torkington Creek at Londonderry Road, and the Rickabys Creek crossing of Carrington Road (which incorporates the combined Torkington and Rickabys Creek catchments). Therefore, worst case flooding across the catchment can vary greatly depending on the location within the catchment.

The PMP durations at each focus location are also included within **Table 11**, however, it is noted that no ARF is applied to PMP depths (although the GSDM ellipses are applied to vary rainfall spatially across the study area). Therefore, the focus locations are used to determine the critical PMP durations only (only one temporal pattern exists per duration).

5.2.6 Comparison with Alternate Modelling Approaches

Regional flood Frequency Estimation (RFFE)

Project 5 of the Australian Rainfall & Runoff revision process included the development of a regional flood frequency estimation (RFFE) approach that enables peak design discharges to be estimated for ungauged catchments. Accordingly, peak discharges were established using the RFFE approach at the 6 focus locations based on the procedures set out by Engineers Australia (2015). The 5% and 1% AEP RFFE discharge estimates are provided in **Table 13**. It is noted that the RFFE method suggests a lower limit on catchment size of 0.5 km^2, and so the results at focus location 1 may be less reliable than other locations.

The regional flood frequency approach acknowledges that there is uncertainty associated with regional approaches. Accordingly, the approach also provides confidence intervals so that an appreciation of the uncertainty in the discharge estimates can be gained. The confidence limits are also included in **Table 13**.

The comparison provided in **Table 13** shows that the WBNM model produces lower discharges relative to the RFFE approach. This difference is most likely associated with the catchment being partially cleared (the RFFE notes that results will be less reliable where there has been land clearing and/or drainage modifications, such as roadside swales). However, for focus locations 1 through 5, the WBNM peak discharges fall well within the RFFE confidence limits at each location. At focus location 6, the WBNM flows fall below the confidence limits, however, the RFFE is not recommended to be used for catchment areas less than 1km^2 (focus location 6 has a 0.6km^2 contributing area) and therefore the RFFE results are not considered reliable at this location.

5.3 Hydraulics

5.3.1 Boundary Conditions

Inflows

As discussed in the previous section, a WBNM hydrologic model was used to simulate the transformation of rainfall into runoff and generate discharge hydrographs throughout the catchment for each design storm. The discharge hydrographs generated by the WBNM model was used to define inflow boundary conditions for the TUFLOW model.

Table 12 Flow comparison between RFFE and WBNM

Focus Location	Description	WBNM ID	Contributing Catchment Area (km2)	WBNM	RFFE			WBNM	RFFE		
				5% AEP	5% AEP	5% AEP Lower Confidence Limit	5% AEP Upper Confidence Limit	1%AEP	1% AEP	1% AEP Lower Confidence Limit	1% AEP Upper Confidence Limit
1	Corner Cassar Cres and Andromeda Dr, Cranebrook (Urban Area)	RbyCk888	0.28	7.06	10.8	4.58	25.8	11.2	22.1	8.93	55.4
2	US Rickabys Ck crossing of Londonderry Rd	RbyCk314	17.01	83.3	143	60.4	342	144	293	118	735
3	US Torkington Ck crossing of Londonderry Rd	RbyCk1026	29.1	118	180	76.0	430	220	368	148	923
4	US Carrington Rd (DS confluence of Tork and Rickabys Ck)	RbyCk1021	54.7	223	295	124	704	413	602	243	1510
5	US The Northern Rd (Rickabys Ck) - tributary crossing	RbyCk232	1.94	15.0	37.6	15.9	89.1	23.5	76.7	31.1	191
6	US Hinxman Rd (Torkington Ck upper reaches)	RbyCk946	0.60	5.20	14.4	6.08	34.4	8.54	29.4	11.9	73.7

Hawkesbury River Boundary Conditions

The Rickabys Creek catchment drains into the Hawkesbury River at Windsor. Accordingly, the prevailing water level within the Hawkesbury River can have a significant impact on flood behaviour across the downstream catchment area. Therefore, it is important to define a reliable Hawkesbury River boundary condition as part of the design flood simulations. At the same time, it was also considered important to note that the goal of the current study is to define flood behaviour for the Rickabys Creek catchment and not re-define flood behaviour for the Hawkesbury River (which has been completed as part of the 'Hawkesbury-Nepean River Flood Study' - NSW Reconstruction Authority, 2024).

It is unlikely that floods of equivalent frequency will occur simultaneously in the Hawkesbury River and Rickabys Creek catchment due to the different characteristics of each catchment. As part of past studies within the Penrith LGA that also drain to the Hawkesbury-Nepean River (e.g., College, Orth & Werrington Creeks catchment), it was assumed that floods of equivalent severity were occurring in the local catchment and receiving watercourse during all events up to and including the 5% AEP event. For local catchment events rarer than the 5% AEP, the 5% AEP tailwater level was retained. This is intended to reflect the reduced probability of very rare/large floods occurring in both watercourses at the same time. Accordingly, it was considered appropriate to apply a consistent methodology for the Rickabys Creek catchment.

In this regard, peak flood levels at the downstream study area boundary were extracted from the 'Hawkesbury-Nepean River Flood Study' (NSW Reconstruction Authority, 2024). The combination of local catchment and Hawkesbury River floods that were adopted for each design flood is summarised in **Table 14**.

Table 13 Adopted Hawkesbury River Boundary Conditions

Design Flood Event	Rickabys Creek Flood Event	Hawkesbury River Flood Event	Hawkesbury River Flood Level (mAHD)
0.5EY	0.5EY	0.5EY	5.38
20% AEP	20% AEP	20% AEP	9.59
10% AEP	10% AEP	10% AEP	11.96
5% AEP	5% AEP	5% AEP	13.80
2% AEP	2% AEP	5% AEP	13.80
1% AEP	1% AEP	5% AEP	13.80
0.5% AEP	0.5% AEP	5% AEP	13.80
0.2% AEP	0.2% AEP	5% AEP	13.80
PMF	PMF	5% AEP	13.80

5.3.2 Blockage

Culvert Blockage

'Base' blockage factors for each bridge and culvert in the study were estimated based upon recommendations in Chapter 6 of Book 6 of ARR2019. 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 C** and is also summarised below:

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

The impact of alternate culvert blockage scenarios was assessed as part of the sensitivity analysis (refer Section 6.3.3).

Stormwater Blockage

Blockage factors were also assigned to stormwater pits based on Council's current pit blockage policy, which is summarised in **Table 15**. The impact of no blockage as well as complete blockage of stormwater pits was assessed as part of the model sensitivity analysis.

Table 14 Adopted Stormwater Pit Blockage Factors

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

The impact of alternate stormwater blockage scenarios was assessed as part of the sensitivity analysis (refer Section 6.3.4).

5.4 Results

5.4.1 Design Flood Envelope

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

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

5.4.2 Presentation of Results

The results of the flood simulations were reviewed, and it was noted that significant areas of shallow 'sheet' flow were present across the catchment, predominantly across very flat terrain such as within the middle reaches of the Torkington Creek catchment (Agnes Banks Nature Reserve). Areas of shallow/isolated inundation are also produced by the application of sub grid sampling where the calculated water level exceeds the median elevation within each model grid cell (i.e., adjoining model cells might have a median elevation that is slightly above versus slightly below the calculated water level, resulting in discontinuous results).

Therefore, it was considered necessary for the results of the computer simulations to be "filtered" to distinguish between areas of significant inundation depth / flood hazard and those areas subject to negligible inundation.

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, to ensure flow paths with higher hazard (even at low depths) were captured, results were reinstated where the velocity-depth product exceeded $0.1m^2/s$. It was noted that the application of the before-mentioned criteria 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 (i.e.: away from other puddles that would form a continuous flow path if no filter criteria was implemented).

5.4.3 Peak Depths, Levels and Velocities

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

- Floodwater Depths: **Figures 17** to **25**
- Floodwater Levels: **Figures 26** to **34**
- Floodwater Velocities: **Figures 35** to **43**

Peak flood levels were also extracted at thirty-four discrete locations across the study area and are provided in **Table 16**. The locations where the results were extracted are shown in **Plate** 7.

Stage hydrographs at key roadway crossings for all flood events have been prepared and are presented in **Appendix J**, and include the crossings of:

- Rickabys Creek at Londonderry Road
- Torkington Creek at Londonderry Road
- Torkington Creek at Torkington Road
- Rickabys Creek at Carrington Road (Hawkesbury River tailwater impacted)
- Rickabys Creek at Cranebrook Road
- Tributary of Rickabys Creek at The Northern Road (adjacent Sutherland Road)

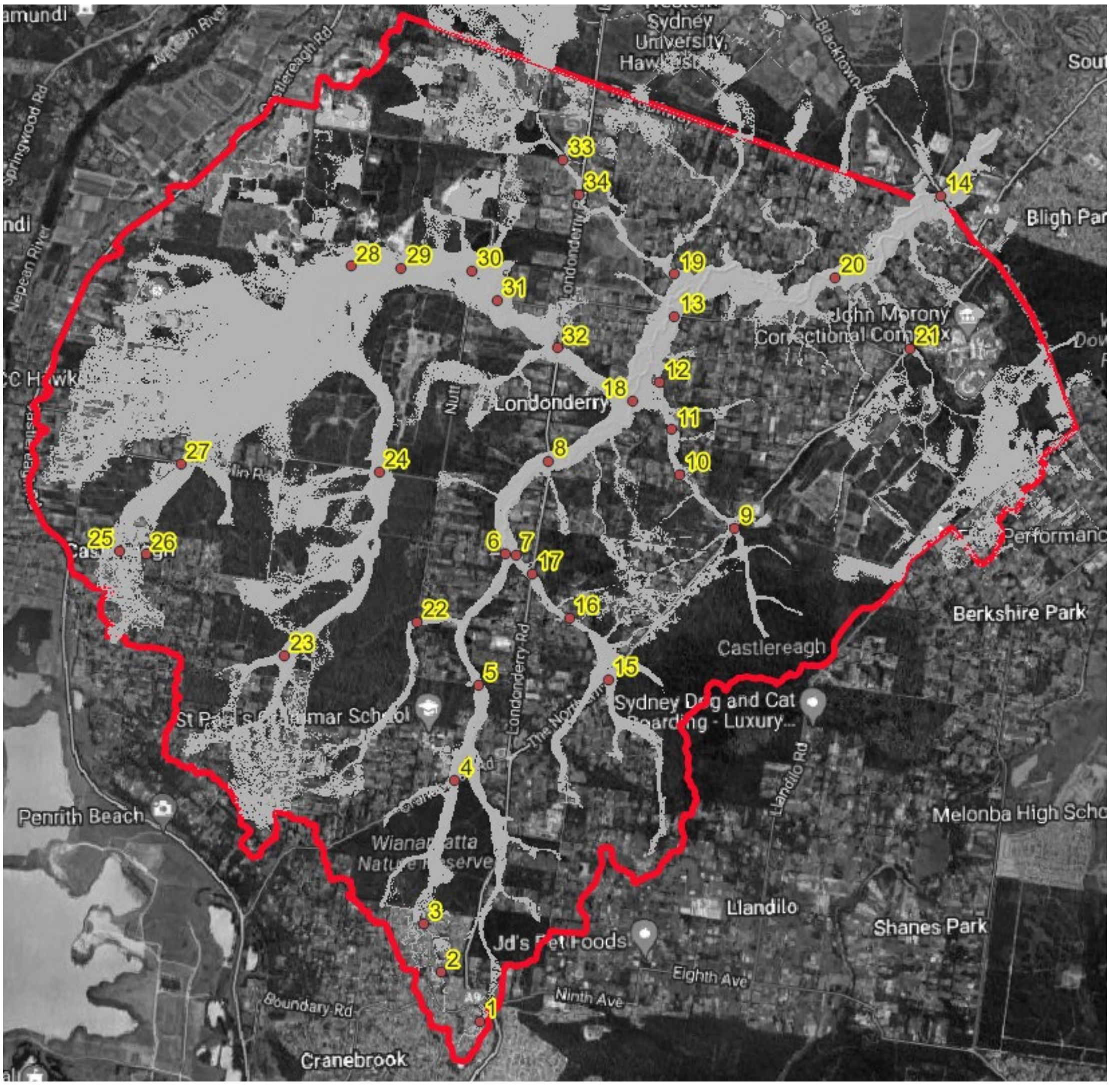

Plate 7 Tabulated flood level result locations.

These stage hydrographs indicate that Londonderry Road is not predicted to overtopped in any flood event other than the PMF at either the Rickabys or Torkington Creek crossings. However, Torkington Road at Torkington Creek is predicted to be overtopped in all flood events, as is Carrington Road at Rickabys Creek. It should be noted that Carrington Road falls within the Hawkesbury River backwater area, and therefore is heavily influenced by flood behaviour within the Hawkesbury River.

5.4.4 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 the identification of where stormwater capacity constraints may exist across the catchment.

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

Location (refer to Plate 7)	Peak Flood Level (m AHD)								
	0.5EY	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
1	57.46	57.72	57.77	57.85	57.90	57.99	58.03	58.07	58.39
2	51.94	52.03	52.09	52.15	52.17	52.25	52.28	52.32	52.66
3	44.94	45.26	45.33	45.44	45.50	45.66	45.71	45.75	46.20
4	35.42	35.54	35.63	35.70	35.74	35.80	35.83	35.90	36.44
5	29.83	29.94	30.03	30.12	30.20	30.33	30.37	30.47	31.72
6	21.87	22.16	22.39	22.57	22.72	22.93	22.98	23.14	25.19
7	22.51	22.64	22.71	22.81	22.87	23.03	23.06	23.19	25.16
8	17.96	18.16	18.34	18.45	18.61	19.04	19.13	19.34	20.96
9	26.93	27.13	27.25	27.33	27.39	27.50	27.54	27.64	28.40
10	20.58	20.73	20.85	20.90	20.94	21.01	21.05	21.10	21.98
11	17.53	18.04	18.17	18.30	18.36	18.44	18.48	18.61	19.49
12	14.64	15.16	15.25	15.35	15.38	15.42	15.43	15.52	17.30
13	12.44	12.62	12.87	13.90	14.02	14.14	14.18	14.28	16.46
14	7.48	9.61	11.97	13.81	13.81	13.82	13.82	13.83	14.41
15	31.12	31.18	31.25	31.30	31.35	31.39	31.41	31.46	31.89
16	26.92	27.02	27.13	27.24	27.34	27.46	27.50	27.62	28.94
17	24.02	24.17	24.30	24.44	24.59	24.81	24.88	25.08	26.31
18	14.06	14.44	14.66	14.77	14.95	15.19	15.25	15.38	17.56
19	12.02	12.13	12.46	13.86	13.93	14.01	14.05	14.12	16.17
20	9.40	9.92	12.00	13.82	13.84	13.86	13.87	13.89	15.15
21	20.31	20.63	21.05	22.40	22.78	22.89	22.93	23.04	23.62
22	31.27	31.38	31.43	31.49	31.53	31.59	31.60	31.67	32.23
23	36.91	36.94	36.97	37.00	37.02	37.04	37.05	37.08	37.45
24	26.99	27.06	27.11	27.13	27.18	27.26	27.28	27.32	28.32
25	30.14	30.19	30.21	30.23	30.25	30.27	30.28	30.30	30.55
26	30.68	30.78	30.83	30.88	30.90	30.92	30.94	30.98	31.09
27	26.65	26.72	26.77	26.83	26.87	26.93	26.94	26.99	27.46
28	20.49	20.59	20.70	20.77	20.91	21.02	21.06	21.14	22.51
29	20.27	20.35	20.43	20.49	20.59	20.68	20.71	20.78	22.01
30	18.59	18.66	18.75	18.81	18.95	19.03	19.07	19.15	20.78
31	18.06	18.15	18.28	18.38	18.58	18.69	18.75	18.86	20.61
32	16.74	16.82	16.97	17.09	17.34	17.46	17.52	17.70	19.58
33	20.20	20.25	20.28	20.32	20.42	20.47	20.48	20.52	21.03
34	18.09	18.21	18.30	18.35	18.57	18.70	18.75	18.84	19.78

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”.

The resulting stormwater capacity maps are presented in **Figure 44**. As shown in **Figure 44**, the pipe capacities are colour-coded based on the nominal capacity that was calculated.

The information presented in **Figure 44** shows that the capacity of the system varies considerably across the urbanised portion of the study area. The upper reaches of the stormwater network are generally able to convey 1% AEP flows or rarer, with the lower reaches generally limited to a capacity of a 0.5EY flood. However, the lower reaches of the stormwater network are typically ‘low flow’ pipes that are located beneath a channel/swale (such as the swale between Andromeda Drive and Vincent Road, Cranebrook), although there are pipes located away from these areas with capacity as low as the 0.5EY event.

Overall, the majority of the stormwater system (i.e., >50%) is predicted to have a capacity of no greater than the 5% AEP event.

5.4.5 Flood Hazard

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

For this study, the variation in flood hazard across the study was defined using flood hazard vulnerability curves presented in the NSW Government’s *‘Flood Risk Management Guideline FB03 - Flood Hazard’* (2023). The hazard curves are reproduced in **Plate 8**.

As shown in **Plate 8**, the hazard curves assess the potential vulnerability of people, cars and structures based on the depth and velocity of floodwaters at a particular location. Therefore, peak depth, velocity and velocity-depth product outputs generated by the TUFLOW model were used to map the variation in flood hazard across the catchment based on the hazard criteria shown in **Plate 8** for each design flood. The resulting hazard category maps are shown in **Figures 45** to **48** for the 5% AEP, 1% AEP, 0.5% AEP floods and the PMF.

The hazard mapping for the 1% AEP event shown in Figure 46 indicates that the highest hazard conditions occur within the watercourse and main drainage lines. The remainder of the 1% AEP inundation, including the main urbanised portions of the study area, is typically only classified as H1 or H2 hazard.

Figure 28 indicates that during the PMF, H3 hazard and higher become predominant across large portions of the study area, with many roads in urbanised portions of the study area exposed to a hazard of H4 or above. This includes sections of Goldmark Crescent, Vincent Road, Andromeda Drive, Arcturus Cl, Londonderry Road, Torkington Road, Carrington Road, and Bowman Road. This indicates that cars and people in these areas would be exposed to a significant flood risk.

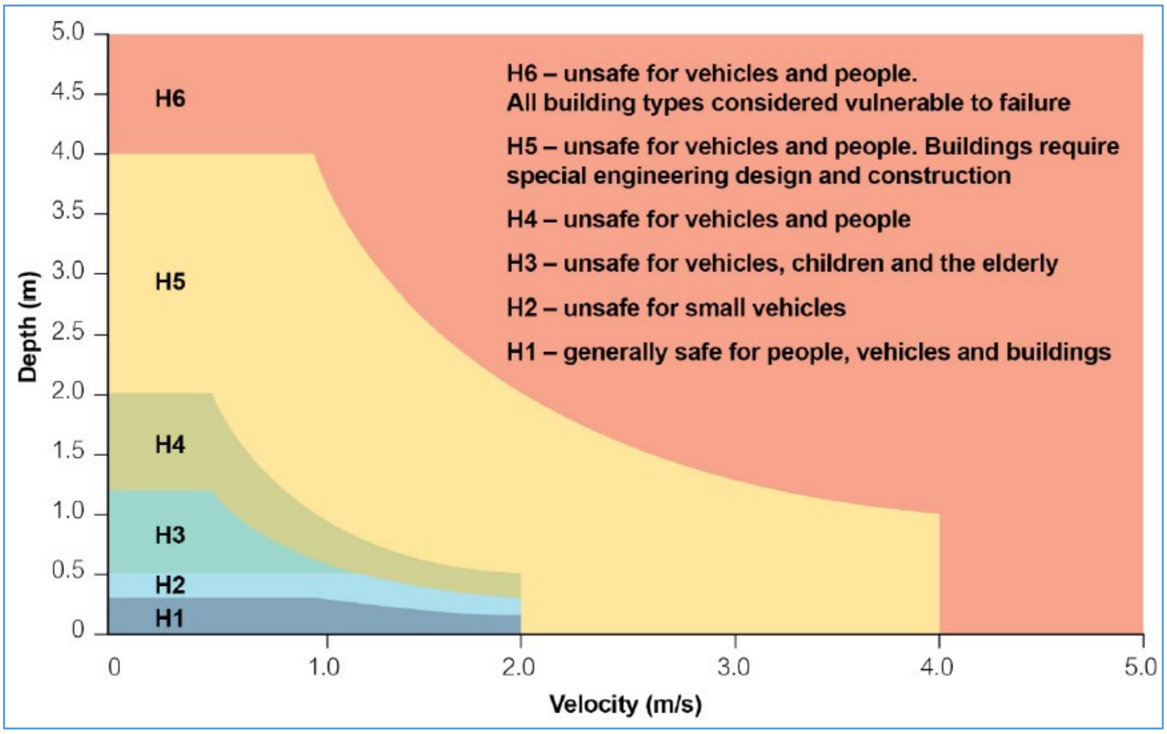

Plate 8 Flood hazard vulnerability curves (Ball et al, 2019)

5.4.6 Flood Emergency Response Classifications

In an effort to understand the potential emergency response requirements across different sections of the study area, flood emergency response precinct (ERP) classifications were prepared in accordance with the flow chart shown in **Plate 9** (NSW Government, 2023). The ERP classifications can be used to provide an indication of areas that may be inundated or isolated during floods. This information, in turn, can be used to quantify the type of emergency response that may be required across different sections of the study area during future floods. This information can be useful in emergency response planning.

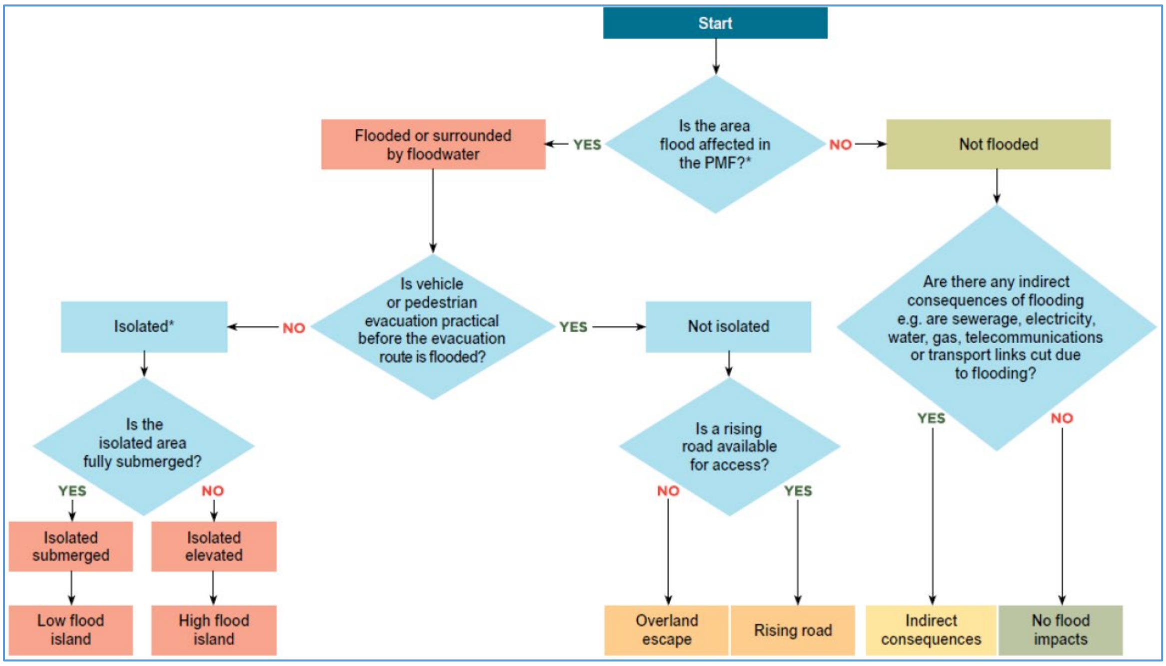

Plate 9 Flow Chart for Determining Flood Emergency Response Classifications.

Each lot within the study area was classified based upon the ERP flow chart for the 20%, 5%, 1%, 0.5% AEP floods and the PMF. This was completed using the TUFLOW model results, digital elevation model and a road network GIS layer in conjunction with proprietary software that considered the following factors:

- Whether evacuation routes get "cut off" by the depth of inundation (a 0.2 m depth threshold was used to define a "cut" road).
- Whether evacuation routes continuously rise out of the floodplain. This criterion is applied to the nearest cross street or road, assuming it is located outside of the PMF extent (i.e., floodplain).
- Whether properties become flooded. A property is considered "flooded" if more than 5% of the property is inundated by floodwaters. When a property is inundated by less than 5% of the total property area, it was considered to be "elevated".
- Indirect consequences are identified when the property is located completely outside of the flood extent. However, it is impacted by other external factors, such as roadways being cut by water, which would prevent access.

The resulting ERP classifications for the 20%, 5%, 1%, 0.5% AEP floods and the PMF are presented on **Figure 49** through **Figure 53**. A range of other datasets were also generated as part of the emergency response classification process to assist Council and the SES. This includes roadway overtopping locations which are also included on **Figure 49** through **Figure 53**. The ID attached to each road overtopping point can be cross-referenced against the results in **Appendix K** to determine the maximum water depth and velocity at the location within the design flood, as well as time the crossing is first cut (from the start of rainfall) and duration that the crossing is cut.

The emergency response classifications indicate that:

- In the 20% AEP event, many properties adjacent Torkington Creek from Nutt Road, and those adjacent to Rickabys Creeks between from Londonderry Road and Blacktown Road are classified as a 'high flood island', and a large number of properties adjacent the upper reaches of the creeks and smaller watercourses are classified as having 'rising road access' meaning that there is some inundation on the lot, but that access is not cut. Many other properties scattered across the study area are also classified as having 'indirect consequences' which indicates properties are not flooded but access to or from the properties is cut and there is potential for utilities to be disrupted.
- In the 5% AEP event, a similar pattern is observed, however, a larger number of properties adjacent to the main creeks in the lower catchment are classified as a 'high flood island' and almost all properties in the northern portion of the study area are classified as either having 'rising road access' or 'indirect consequences'. A large number of properties within Cranebrook are also classified as having 'indirect consequences'.
- In the 1% AEP and 0.5% AEP events, almost all properties adjacent to Torkington Creek from Nutt Road, and those adjacent to Rickabys Creeks (and its smaller tributaries) from Smeeton Road to Blacktown Road are classified as a 'high flood island'. Almost all properties in the northern portion of the study area are classified as having 'rising road access', 'overland escape routes' or 'indirect consequences'. Properties on Bellatrix

Street, Aldebaran Street and Mozart Place within Cranebrook are classified as a 'high flood island'.

- In the PMF, a similar pattern to the 1% AEP and 0.5% AEP event is predicted, however, additional properties in the northern portion of the study area are classified as a 'high flood island' and much of the remainder of the study area is classified as having an 'overland escape route', 'rising road access' or 'indirect consequences'.

Appendix K indicates that the depth of flooding would be sufficient to cut 228 individual road segments during the 20% AEP flood, 298 in the 5% AEP event, 349 in the 1% AEP event, 359 in the 0.5% AEP event and 503 in the PMF.

5.4.7 Flood Function

The NSW Government's *'Flood Risk Management Manual'* (2023) subdivides flood prone areas according to the flood function categories presented in **Table 17**. The flood 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 or storage of floodwaters.

Guidance for establishing flood function categories is provided in the NSW Government's *'Flood Risk Management Guideline FB02 - Flood Function'* (2023). However, explicit quantitative criteria for defining each category are not provided. This is because the extent of floodway, flood storage and flood fringe areas are typically specific to a particular catchment. Therefore, it was necessary to review the modelling results and use this information as a basis for developing criteria to describe each flood function category.

For this study, the following approach was employed.

- Floodways:
  - Initial estimates of the floodway extent were determined based on the depth and velocity and velocity-depth product (VxD) outputs from the flood modelling in conjunction with criteria set out in Howells et al (2003).
  - The use of depth, velocity and VxD outputs resulted in discontinuities, so the conveyance approach recommended in the NSW Government's *'Flood Risk Management Guideline FB02 - Flood Function'* (2023) was incorporated.
  - Manual adjustments of the floodways were then undertaken to ensure full continuity, where required.
  - Given the differing nature of flood behaviour within the main Torkington and Rickabys Creeks (and their major tributaries) compared to the more minor creeks and overland flow areas, separate criteria were adopted for floodways.
  - The final adopted criteria for floodways are provided in **Table 17**
- Flood Fringe:
  - Floodways were removed.
  - A water depth threshold was then used to identify potential flood fringe areas.
  - The suitability of the flood fringe was tested by blocking out all flood fringe areas and re-running the design flood and confirming if this produced an unacceptable flood impact (as noted in **Table 17**, removing flood fringe areas should not have a

significant impact on flood behaviour). For this study, an "unacceptable impact" was quantified as a flood level increase of 0.1 metres.

- The depth threshold was adjusted iteratively until the flood level impacts were contained below 0.1 metres.
- The final adopted criteria for flood fringe are provided in **Table 17.**
- Note that as filtering of the results was undertaken (see Section 5.4.2), much of the flood fringe area has also been removed from the presentation of results.

- Flood Storage:
  - Remaining areas after floodway and flood fringe areas were removed.

Table 16 Qualitative and Quantitative Criteria for Hydraulic Categories

Hydraulic Category	Flood Risk Management Manual Definition	Adopted Criteria	
		Major Creeks	Minor Creeks/ Overland Flow
Floodway	Are generally areas which convey a significant portion of water during floods and are particularly sensitive to changes that impact flow conveyance. They often align with naturally defined channels	• $VxD \geq 0.5$ m^2/s AND $D \geq 3$ m AND $V \geq 0.5$ m/s • Conveyance of 80% of flow	• $VxD \geq 0.4$ m^2/s AND $D \geq 1$ m AND $V \geq 0.4$ m/s • Conveyance of 80% of flow
Flood Fringe	Are areas within the extent of flooding for the event but which are outside floodways and flood storage areas. Flood fringe areas are not sensitive to changes in either flow conveyance or storage.	• Not floodway AND • $D \leq 0.2$ m OR • $V \leq 0.2$ m/s	
Flood Storage	Are areas outside of floodways, are generally areas that store a significant proportion of the volume of water and where flood behaviour is sensitive to changes that impact on the storage of water during a flood	• Remaining Areas	

NOTES: V = Velocity, D = Depth

The velocity and depth results produced by the TUFLOW model for each design flood were combined with the criteria detailed in **Table 17** to produce hydraulic category maps. The resulting maps are shown in **Figures 54** to **57** for the 5% AEP, 1% AEP, 0.5% AEP floods and the PMF.

Verification of the hydraulic categories was also completed, and the outcomes of the verification are provided in **Appendix L**.

5.4.8 Inundated Properties

The number of properties inundated during each design flood was also determined. This information is summarised in **Table 18** (there are 3,335 properties contained within the study area).

Table 17 Number of Inundated Properties

Event	Number of Inundated Properties	Percentage of Total Number of Properties
0.5EY	876	26%
20% AEP	962	29%
10% AEP	1023	31%
5% AEP	1095	33%
2% AEP	1155	35%
1% AEP	1225	37%
0.5% AEP	1254	38%
0.2% AEP	1294	39%
PMF	1671	50%

The information presented in **Table 18** indicates that 37% of properties located within the catchment will be at least partly inundated to a depth of at least 0.15 metres at the peak of the 1% AEP flood. This is predicted to increase to 50% during the PMF. Accordingly, major flooding has the potential to impact a significant number of properties within the catchment.

5.5 Discussion on Flood Behaviour

The flood mapping shows that during more frequent floods (e.g., up to and including the 10% AEP event), flooding is generally restricted to the main creeks, water courses and roadside swales, including across urbanised portions of the study area. However, roadways within the more rural portion of the catchment can become inundated (depths > 0.3 metres which would typically be sufficient to prevent vehicular movement) in events as frequent as the 20% AEP. This includes:

- Boscobel Road (near Londonderry Road)
- Nutt Road (near Torkington Road)
- Torkington Road (near Nutt Road)
- Carrington road (near Milford Road)
- Milford Road (near Carrington Road)
- Smeeton Road (near Londonderry Road)

During moderate to larger floods (i.e., up to and including the 1% AEP event), water depths across roads become more significant. In addition to many roadways within the more rural portion of the catchment, the following major roadways also become inundated and impassable during the 1% AEP flood:

- The Northern Road (near Carrington Road, near Sutherland Road)
- Devlin Road (near Nutt Road)
- Vincent Road (near Andromeda Drive)
- Taylor road

During the PMF, almost all major roadways become inundated, such as:

- Londonderry Road (at numerous locations)
- Cranebrook Road (near Londonderry Road)
- The Driftway (near Blacktown Road)
- Blacktown Road (near The Northern Road and The Driftway)

There are a number of regional (Hawkesbury-Nepean) emergency evacuation routes that pass through the study area, including The Northern Road, Llandilo Road, Londonderry Road and Castlereagh Road. At the peak of the events up to and including the 0.5% AEP, these all remain trafficable apart from The Northern Road at Sutherland Road. However, in the PMF, The Northern Road and Londonderry Road are cut at multiple locations.

Given that the Rickabys Creek watercourse upstream of Londonderry Road and the upper and lower reaches of the Torkington Creek catchment fall within private property, there are a large number of private properties impacted by flooding in all severity events.

5.6 Results Verification

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

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

It should be noted that the 2019 revision of Australian Rainfall & Runoff has been used in the current study to define catchment hydrology. Previous studies have used the 1987 version of Australian Rainfall & Runoff. Accordingly, it is not possible to complete an “apples with apples” comparison as part of this verification due to the different hydrologic approaches. Nevertheless, it is still considered possible to ensure the models are producing realistic estimates of design flood behaviour using this verification approach.

5.6.1 Comparison with Past Studies

The Penrith Overland Flow Flood “Overview Study” (Cardno, 2006) provides the most comprehensive flood level information across the catchment. Peak 1% AEP levels were extracted from the overview study and were compared against peak 1% AEP flood levels produced by the TUFLOW model developed for the current study at various locations across the catchment. The water level comparisons are provided in **Table 19**.

Table 18 Verification of 1% AEP water levels

Location (refer to Plate 7)	**Peak Water Level (mAHD)** Penrith Overland Flow Flood "Overview Study" (2006)	**Peak Water Level (mAHD)** Current Study (TUFLOW)	**Difference**	**Comment**
1	58.07	57.99	-0.08	
2	52.06	52.25	0.19	
3	45.89	45.66	-0.23	
4	35.56	35.80	0.23	
5	30.27	30.33	0.06	
6	22.46	22.93	0.47	Located upstream of culvert crossing (design blockage not applied to 2006 study) or incorrect culvert size used (2006 study)
7	22.52	23.03	0.50	
8	18.67	19.04	0.37	
9	N/A	27.50	N/A	
10	20.91	21.01	0.10	
11	18.12	18.44	0.32	Located upstream of culvert crossing (design blockage not applied to 2006 study) or incorrect culvert size used (2006 study)
12	15.19	15.42	0.23	
13	13.12	14.14	1.02	Located in Hawkesbury River backwater area, with different TWL parameters to 2006 study
14	N/A	13.82	N/A	
15	31.30	31.39	0.09	
16	27.09	27.46	0.37	Located upstream of culvert crossing (design blockage not applied to 2006 study) or incorrect culvert size used (2006 study)
17	25.36	24.81	-0.55	
18	14.85	15.19	0.34	Located in Hawkesbury River backwater area, with different TWL parameters to 2006 study
19	12.64	14.01	1.37	Located upstream of culvert crossing (design blockage not applied to 2006 study) or incorrect culvert size used (2006 study)
20	11.29	13.86	2.57	Located in Hawkesbury River backwater area, with different TWL parameters to 2006 study
21	N/A	22.89	N/A	
22	31.33	31.59	0.26	
23	36.88	37.04	0.16	

Location (refer to Plate 7)	Peak Water Level (mAHD)		Difference	Comment
	Penrith Overland Flow Flood "Overview Study" (2006)	Current Study (TUFLOW)		
24	27.40	27.26	-0.14	
25	30.19	30.27	0.08	
26	N/A	30.92	N/A	
27	26.95	26.93	-0.02	
28	21.01	21.02	0.01	
29	20.62	20.68	0.06	
30	18.83	19.03	0.21	
31	18.44	18.69	0.25	
32	17.08	17.46	0.38	Located upstream of culvert crossing (design blockage not applied to 2006 study) or incorrect culvert size used (2006 study)
33	20.26	20.47	0.21	
34	19.51	18.70	-0.81	Located upstream of culvert crossing (design blockage not applied to 2006 study) or incorrect culvert size used (2006 study)

The comparison provided in **Table 19** indicates that both studies generate similar peak 1% AEP flood levels (i.e., levels generally agree to better than 0.3 metres). The current study produces higher peak flood levels. This is considered to be associated with the current study being a predominantly mainstream flood model (with flows applied to the main flow areas) as opposed to the 'direct rainfall' approach adopted as part of the 2006 study which includes passive (and potentially unrealistic) storage outside of mainstream areas. The current study also applies a different Hawkesbury River tailwater level, has incorporated surveyed culvert details (size and inverts), and uses a 2 metre grid size (instead of the 9 metre grid used in the "Overview Study"). There are some locations where the TUFLOW model developed for the current study is predicting lower peak design flood levels relative to the 2006 study. These are typically located upstream of roadway crossings and are likely associated with some of the major cross-drainage structures not being represented, or incorrectly represented in the 2006 study, resulting in lower upstream flood levels in these locations within the current study.

It should also be noted that flood level comparison is not available from the 2006 study in some locations due to the 'filtering' criteria used within the study removing results in areas of shallow inundation.

5.6.2 Comparison with Alternate Modelling Approaches

Direct Rainfall Model

As discussed, two separate computer models have been developed as part of the study to define flood behaviour across the Rickabys Creek catchment. It was considered important to undertake additional validation of these models by comparing the results generated by the WBNM and TUFLOW models against an alternate computer modelling approach.

"Direct rainfall" (DR) TUFLOW models have been employed in a number of recent flood studies across the Penrith City Council LGA. The direct rainfall models involve application of rainfall directly to the hydraulic model (i.e., the hydrology and hydraulics are combined into a single model). Unfortunately, the long run times associated with the DR TUFLOW models means that they cannot be efficiently used to simulate the thousands of storms required under ARR2019. However, it was still considered that a DR version of the Rickabys Creek catchment could be developed and used to verify the results of the WBNM/TUFLOW model based upon the reduced set of 1% AEP ARR2019 storms discussed in the preceding sections (see **Table 11**).

Flow hydrographs were extracted from the direct rainfall model at four locations and were compared against flow hydrographs extracted from the TUFLOW model with hydrology defined by the WBNM model. The locations where the hydrographs were extracted included:

- Rickabys Creek at Senta Road (reflecting the headwaters of Rickabys Creek with the influence of the urbanised portion of Cranebrook);
- Rickabys Creek at Londonderry Road (reflecting mainstream Rickabys Creek flows);
- Torkington Creek at Londonderry Road (reflecting mainstream Torkington Creek flows); and,
- Rickabys Creek near Carrington Road (reflecting combined Rickabys/Torkington Creek flows).

The flow hydrographs are presented on **Plate 10** through **Plate 13**. The comparison indicates that the overall shape and timing of the hydrographs at all locations are very similar. The hydrographs extracted from TUFLOW using WBNM flows tend to have some higher flows earlier in the 1% AEP event. This is most likely associated with minor depression storage in the direct rainfall version of the model that is not reflected in the WBNM version of the model. At the peak of the 1% AEP flood and during the receding limb, the flows are almost identical. Overall, the WBNM/TUFLOW version and direct rainfall version of the model are generating very similar results.

The peak flood levels generated by the direct rainfall model were also compared against peak flood levels generated by the WBNM/TUFLOW models for the 5%, 1% and 0.5% AEP events. This is presented in the form of flood level difference mapping and is included in **Appendix M**. Greens/blues indicate the direct rainfall version of the model is generating lower levels while yellows/oranges indicate the direct rainfall version is generating higher levels. Note that, as the direct rainfall version of the model produces flood level information for every cell within the model domain, a 0.15 metre 'cutoff' depth has been applied prior to the comparison to remove areas of shallow inundation and allow a clearer comparison between model results to be undertaken.

This difference maps shows that, in general, the WBNM version and direct rainfall versions of the model are producing peak flood levels that are generally within +- 0.05 metres of each other. In the 5% AEP event, some larger reductions of over 0.1 metres are shown to occur along the main Torkington and Rickabys Creeks and suggest that the direct rainfall version of the model is allowing some additional passive storage and/or flow attenuation that is reducing peak flood levels compared to the WBNM version of the model. Overall, the results of the direct rainfall verification shows that the WBNM and TUFLOW models are providing realistic descriptions of design flood behaviour.

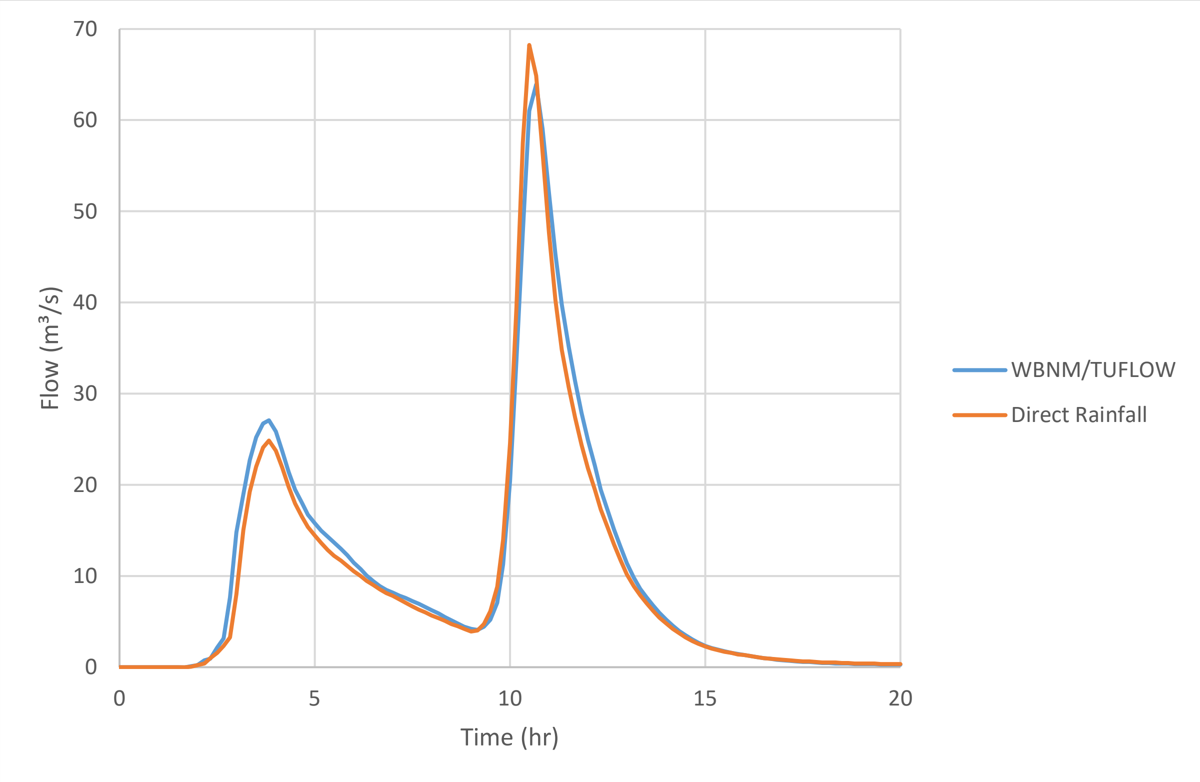

Plate 10 1% AEP Direct Rainfall and WBNM/TUFLOW Comparison on Rickabys Creek at Senta Road

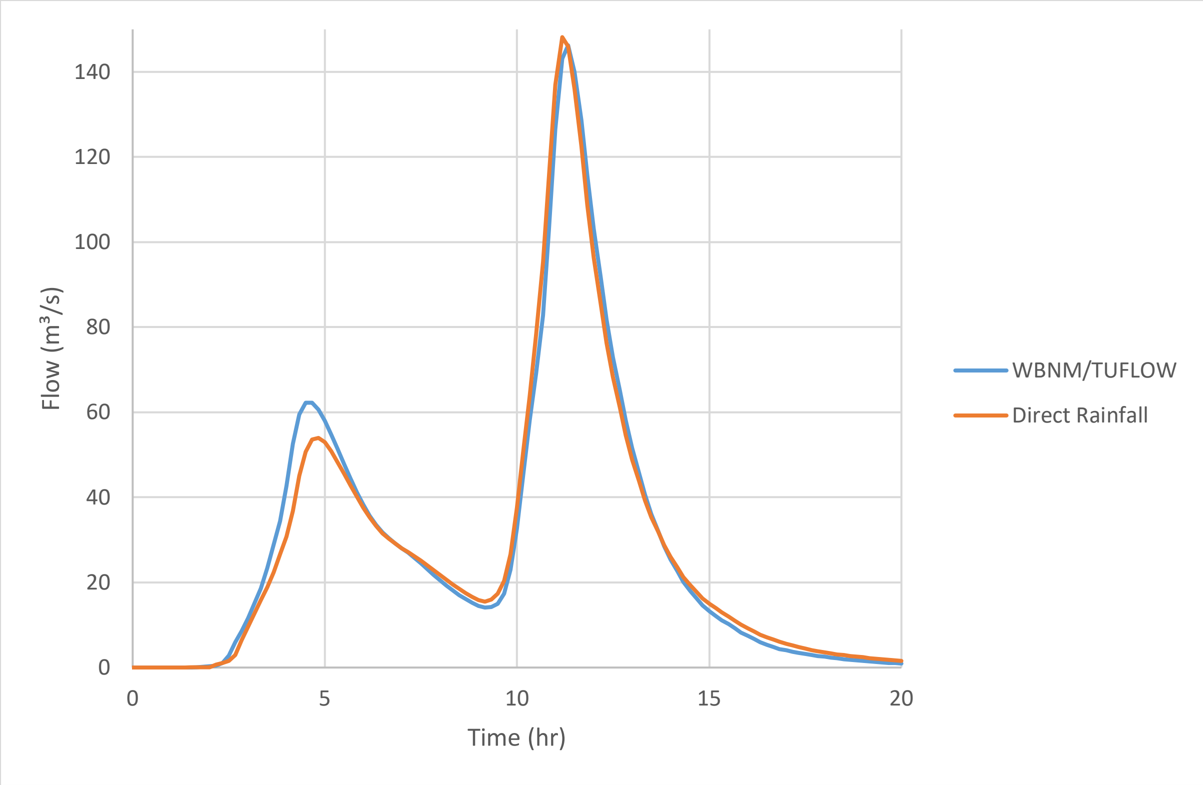

Plate 11 1% AEP Direct Rainfall and WBNM/TUFLOW Comparison on Rickabys Creek at Londonderry Road

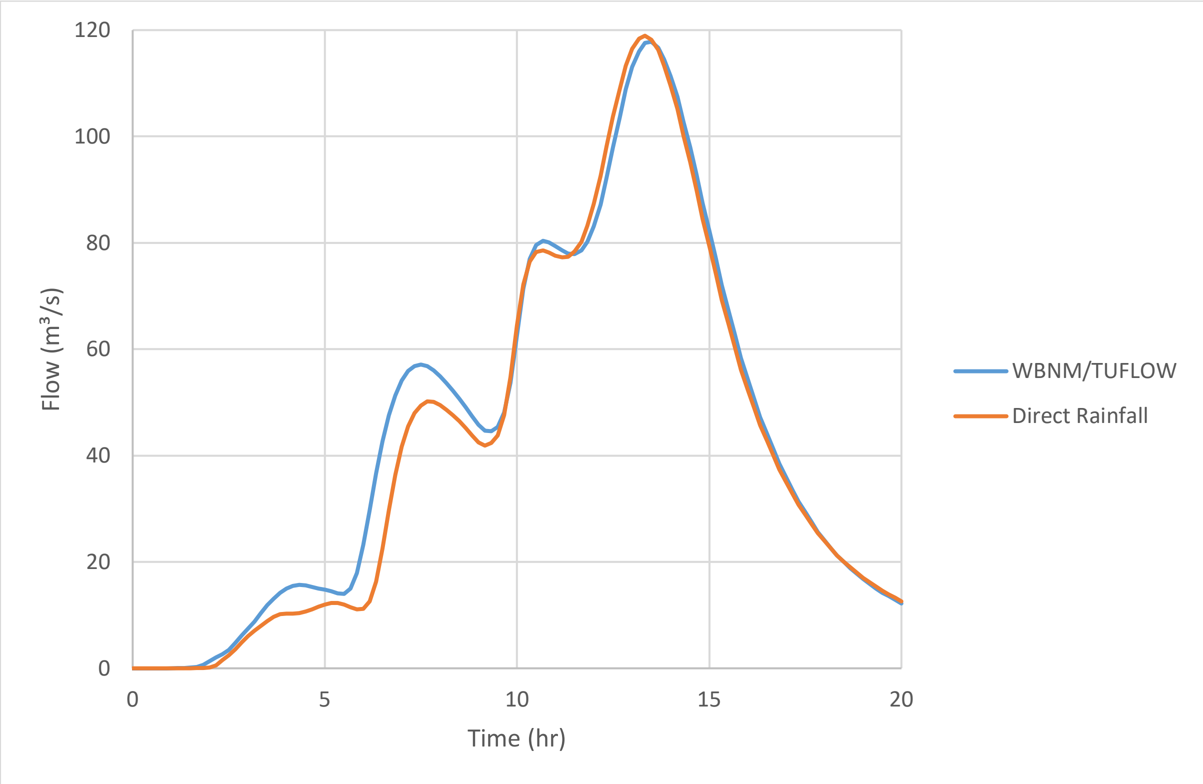

Plate 12 1% AEP Direct Rainfall and WBNM/TUFLOW Comparison on Torkington Creek at Londonderry Road

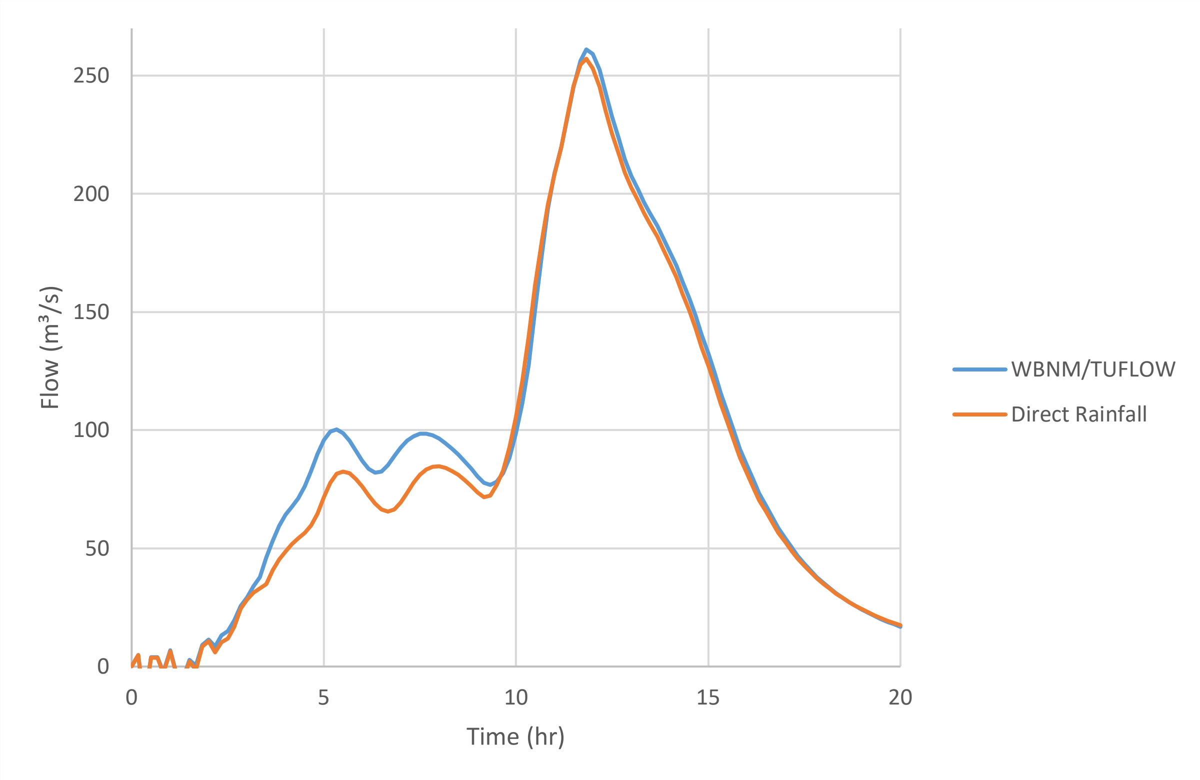

Plate 13 1% AEP Direct Rainfall and WBNM/TUFLOW Comparison on Rickabys Creek at Carrington Road

5.6.3 Field Verification

Preliminary floodwater depth maps were prepared for the 1% AEP flood based upon the 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, "field verification" was completed to confirm the veracity of the modelling results. The verification involved undertaking a field review of locations where there was any uncertainty in the modelled flood behaviour or that required confirmation of the preliminary mapping results. The field verification generally focussed on areas where overland flow was predicted to impact on buildings, major roadways were predicted to be cut, or flood hazard around buildings was high. This field verification process aimed to confirm whether the modelling results were realistic and whether the model would require any adjustments to improve the representation of flood behaviour.

A total of 35 locations were visited in the field and confirmed that the flood behaviour shown on the mapping was representative of conditions observed on the ground. **Appendix M** presents the outcomes of the field verification at these locations.

5.6.4 Peer Review

As previously discussed, the WBNM and TUFLOW computer models provided a good reproduction of historic flood information and also compare favourably against other flood-related studies. However, to further ensure that the models were appropriately setup and parameterised, an additional internal peer review of both the WBNM hydrologic and TUFLOW hydraulic models was completed. The review also included interrogation of the TUFLOW 'messages' layer which identifies any unexpected or inconsistent input data.

The review identified no major issues with model schematisation and that the TUFLOW messages largely refers to check/warning messages for such things as different inverts for connected pipes/pits (which is realistic based on the detailed survey) and interpolated "z point" elevations on gully lines (which is also reasonable).

6 SENSITIVITY AND CLIMATE CHANGE ANALYSIS

6.1 Overview

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

As outlined in Chapter 4, 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 Chapter 4 and Section 5.6, the TUFLOW model was validated against recorded and observed flood information for four historic events and was further verified against results documented in past studies. In general, this information confirmed that the models were providing realistic descriptions of flood behaviour across the catchment.

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

6.2 Model Parameter Sensitivity

6.2.1 Initial / Storm Loss

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

The variation in antecedent wetness conditions was represented by altering the "burst" rainfall loss in the WBNM model. This involved altering the probability neutral rainfall losses discussed by ±20% and re-running the 5%, 1% and 0.5% AEP storms. The revised discharges were extracted from the results of the modelling and are included in **Appendix N**. Peak discharges for the "base" conditions are also included in **Appendix N** for comparison.

The peak discharge comparison indicates that increasing the storm loss (reflecting a dryer catchment) decreases peak design discharges at most locations. More specifically, peak discharges are predicted to reduce, on average, by 3% in the 5% and 1% AEP events, and up to 7% in the 0.5% AEP event.

Reducing the storm loss (reflecting a wetter catchment) is predicted to generate increases in peak design discharges at most locations. Peak discharges are predicted to increase, on average, by between 3 and 4% in the 5% and 1% AEP events, and up to 6% in the 0.5% AEP event, although localised increases of more than 10% are predicted at a small number of locations.

Overall, the peak discharges do not appear to be particularly sensitive to the adopted storm losses.

The revised discharge hydrographs were then applied to the TUFLOW model and the TUFLOW model was used to re-simulate the 5%, 1% and 0.5% AEP floods with the modified storm losses. Peak water levels were extracted from the results of the modelling and were compared against peak flood levels for "base" design conditions. This allowed water level difference mapping to be prepared showing the magnitude of any change in water levels associated with the change in initial loss values. The difference mapping is presented in **Appendix O**. Decreases in "design" flood levels associated with the parameter change are shown in shades of green/blue and increases in flood levels are shown in shades of yellow/red.

The difference mapping shows that in the 5% AEP event, a 20% increase in burst loss will generate only small decreases in peak flood level (generally no more than 0.02 metres). A 20% decrease in burst loss is shown to produce increases of generally no more than 0.03 metres, although increases of 0.02 metres are more common.

In the 1% and 0.5% AEP events, the flood level impacts are negligible (i.e.: < 0.01 metres). Therefore, the hydraulic model results do not appear to be sensitive to the adopted initial/burst loss.

6.2.2 Continuing Loss Rate

An analysis was also undertaken to assess the sensitivity of the results generated by the WBNM model to variations in the adopted continuing loss rates. Accordingly, the continuing loss rates within the WBNM model was changed from the "design" values of 1.36 mm/hr (pervious areas) and 0 mm/hr (impervious areas) to:

- Increased Continuing Loss Rates: 1.86 mm/hr for pervious areas and 0.5mm/hr for impervious areas.
- Decreased Continuing Loss Rates: 0.86 mm/hr for pervious areas and 0mm/hr for impervious areas.

The modified continuing loss rates were applied to the WBNM model and were used to re-simulate the 5%, 1% and 0.5% AEP storms. The revised discharges were extracted from the results of the modelling and are included in **Appendix N**.

The peak discharge comparison indicates that changing the continuing loss rate generally produces only small changes in peak discharges. More specifically, peak discharges are predicted to change, on average, by less than ±1% in the 5% and 1% AEP events, and up to ±1.5% in the 0.5% AEP event. Accordingly, the hydrologic model does not appear to be particularly sensitive to changes in the adopted continuing loss rate.

The revised discharge hydrographs were then applied to the TUFLOW model and the TUFLOW model was used to re-simulate the 5%, 1% and 0.5% AEP floods with the modified continuing loss rates. Peak water levels were extracted from the results of the modelling and were used to prepare flood level difference mapping which are presented in **Appendix O**.

The results of the sensitivity analysis show that the TUFLOW model is relatively insensitive to changes in continuing loss rates. More specifically, **Appendix O** shows that only small (i.e., generally no greater than ±0.02m) localised changes in 5%, 1% and 0.5% AEP flood levels are predicted with the modified continuing loss rates.

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

6.2.3 Temporal Pattern

As discussed in Section 5.2.4, ten different temporal patterns were applied to each design storm frequency and duration. The first temporal pattern that provided a peak discharge above the mean was adopted as part of the 'base' design flood simulations. However, it was considered important to gain an understanding of how variations in the rainfall temporal pattern (i.e., how the rainfall is distributed over time) may impact on the results generated by the model. Therefore, additional simulations were completed by adopting a temporal pattern at the upper and lower end of the adopted critical durations.

Appendix N summarises the peak discharges at each subcatchment that are generated using the upper and lower temporal patterns within the critical duration for the 5%, 1% and 0.5% AEP events. The 'base' peak discharges are also listed for comparison.

The comparison presented in **Appendix N** shows that the temporal patterns that produce lower peak discharges will generate peak design discharges that are 16% lower (on average) for the 5% and 1% AEP events, and 26% lower for the 0.5% AEP event relative to the base peak discharges. Adopting a temporal pattern that produces high peak discharges is predicted to increase peak design discharges by 12%, on average for the 5% and 1% AEP events, and 20% higher for the 0.5% AEP event.

The revised discharge hydrographs were then applied to the TUFLOW model and the TUFLOW model was used to re-simulate the 5%, 1% and 0.5% AEP floods with the modified temporal patterns. Flood level difference mapping was prepared to display the impacts of the temporal pattern modifications and are presented in **Appendix O**.

The difference mapping shows that adopting a lower temporal pattern can reduce peak 5% AEP flood levels by up to 0.25 metres. In the 1% and 0.5% AEP events, levels can reduce by up to 0.3. By adopting a higher temporal pattern, peak 5% AEP flood levels can increase by up to 0.1 metres, and peak 1% and 0.5% AEP levels can increase by up to 0.15 metres.

It is noted that the changes in peak flood level in all flood events are greatest within the main creek channels with much more modest changes in peak flood level present within the minor tributaries and overland flow areas where changes are generally no greater then +- 0.05 metres. However, the peak design discharges and water level results appear to be somewhat sensitive to the adopted temporal pattern, although, it is considered that the adopted temporal patterns better maintain AEP neutrality (and conform to guidance within ARR2019).

6.2.4 Spatial Location of IFD Coefficients

As discussed in Section 5.2.1, IFD coefficients were extracted and applied at five locations around the study area, with the closest IFD location to each subcatchment applied as part of the design flood simulations. To test the impact of any variation in the applied IFD coefficients, additional 5%, 1% and 0.5% AEP simulations were undertaken with a single IFD located at the centroid of the catchment, with the aerial reduction factor for this location being applied (catchment area of 29.1 km2).

Appendix N summarises the peak discharges at each subcatchment that are generated using the IFD/ARF at the centroid of the catchment (see "Base ARR2019 Results (Centroid of Catchment)" columns within the initial loss and continuing loss tables).

Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping, which is presented in **Appendix O**. In general, the changes in flood levels associated with the application of the single IFD coefficient across the catchment produces decreases in peak flood level of less than 0.05 metres for the 5%, 1% and 0.5% AEP events. This is associated with the higher aerial reduction factor applied across the catchment due to the larger area being considered. Therefore, peak flood levels across the catchment does not appear to be sensitive to the adoption of the IFD and ARF at the centroid of the catchment. However, it is still considered better to use the adopted flows generated at the 6 focus locations to ensure flood behaviour at the local level is not underrepresented.

6.3 Hydraulic Model Inputs

6.3.1 Roughness Coefficients

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 roughness values). Therefore, additional analyses were completed to quantify the impact that any uncertainties associated with hydraulic 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 roughness values and additional 5%, 1% and 0.5% AEP simulations were completed with the modified "n" values (no changes to hydrology were completed as part of this assessment). Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping, which is presented in **Appendix O**.

In the 5% AEP event, the changes in flood levels associated with modifying roughness values are predicted to be less than ±0.03 metres, but localised flood level increases/decreases approaching ±0.08 metres are predicted at some locations (primarily along heavily vegetated watercourses). It is noted that reducing roughness values will typically reduce flood levels. However, there are some areas within the lower reaches of the catchment, where the more rapid response of rainfall from the upper catchment is predicted to generate localised increases in flood levels. Conversely, increasing roughness values will typically increase flood levels, but the additional attenuation will result in flood level reductions across some locations (again, within the lower reaches of the catchment).

In the 1% and 0.5% AEP events, a similar pattern is observed, albeit with a larger magnitude of variation, with typical changes of ±0.04 metres, and maximum changes of up to ±0.1 metres.

Although localised changes in flood level of up to ±0.1 metres are predicted at isolated locations (predominantly within the main watercourses), the change in flood levels across most of the study area, including across urban areas, are predicted to be much smaller. As a result, it is considered that the model is relatively insensitive to changes in roughness values.

6.3.2 Structure Losses

Loss coefficients were included for each culvert to represent hydraulic losses through each structure. To gain an understanding of how uncertainty in the loss coefficients may impact on the design flood results, revised simulations were completed with the loss coefficients increased and decreased by 20%.

The updated versions of the model were used to re-simulate the 5%, 1% and 0.5% AEP floods with the modified loss coefficients. Peak water level results were extracted and used to prepare flood level difference mapping which is enclosed in **Appendix O**.

The results of the simulations show that altering the loss coefficient by ±20% is predicted to produce small, localised impacts on peak flood levels, particularly upstream of large drainage infrastructure. In all cases the flood levels are not predicted to change by more than 0.02 metres. Therefore, any uncertainty in the representation of structure losses does not have a significant influence on the model results.

6.3.3 Hydraulic Structure Blockage

As discussed in Section 5.3.2, blockage factors were applied to all culverts 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 by adopting the ‘low’ blockage scenario (normally adopted for frequent flood events >5% AEP), and the ‘high’ blockage scenario (normally adopted for rare flood events <0.5% AEP).

Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping, which is presented in **Appendix O**.

The results of the simulations show that adopting a lower complete blockage will produce decreases of generally less than 0.02 metres in peak flood level on the upstream side of structures in all flood events. Increases on the downstream side of structures is also predicted, however, the magnitude of these is generally less than 0.01 metres.

Adopting a higher blockage factor is predicted to produce increases in flood levels on the upstream side of structures of between 0.02 metres in the 5% AEP event and 0.06 metres in the 1% and 0.5% AEP events. These increases can extend within the upstream channels and is associated with the "damming" effect provided by the embankment. Some small decreases in peak flood level are also predicted on the downstream side of some structures.

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 higher than expected blockage of structures occurs. This outcome emphasises the need to ensure key drainage infrastructure and bridges and culverts are well maintained (i.e., debris is removed on a regular basis).

6.3.4 Stormwater Blockage

As also discussed in Section 5.3.2, blockage factors were applied to all stormwater pit inlets as part of the design flood simulations based on Councils design standards. However, the actual degree of blockage in any flood event at any stormwater pit is not known and subject to variation.

As such, additional TUFLOW simulations were completed to determine the impact that alternate stormwater pit blockage would have on flood behaviour. Specifically, additional simulations were undertaken by adopting 0% and 100% blockage of all stormwater pits within the study area. Peak flood levels were extracted from the results of the modelling and were used to prepare flood level difference mapping, which is presented in **Appendix O**.

In general, the lower stormwater blockage scenario produces modest reductions (up to 0.03 metres) in peak flood level in isolated locations (generally trapped low points). However, the complete blockage of stormwater inlets can have a significant impact on flood levels within the urbanised portions of the study area. This includes increases that can exceed 0.8 metres in the 1% and 0.5% AEP events, and areas of 'now wet' forming in all events due to a redistribution of flow. Decreases in peak flood level are predicted downstream of urbanised areas where less flow is able to be discharged.

6.4 Australian Rainfall & Runoff 1987

Flood studies across the Penrith LGA over the last three decades have been prepared in accordance with *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (Engineers Australia, 1987) (ARR1987). The current study has been prepared in accordance with the updated ARR2019, which reflects application of more thorough hydrologic procedures as well as an additional 30 years of rainfall information. Nevertheless, it was considered important to understand how results produced based on ARR2019 may differ from those generated using ARR1987. Therefore, an additional sensitivity assessment was completed to confirm the impact that the revised hydrologic procedures may have on design flood behaviour across the study area.

This involved re-running the 5% AEP and 1% AEP storms based on ARR1987 through the WBNM model to confirm the critical storm durations and then running the critical storms through the TUFLOW model. Difference mapping was then prepared by subtracting the ARR2019 water level results from the ARR1987 water level results. The difference mapping is presented in **Plate 14** and **Plate 15**.

The difference mapping presented in **Plate 14** and **Plate 15** also shows that ARR1987 is predicted to produce higher flood levels relative to ARR2019. Along major watercourses, the ARR1987 levels are typically 0.1 metres higher than ARR2019 levels in the 5% AEP event, and 0.2 metres higher than ARR2019 levels in the 1% AEP event. Localised increases of more than 0.2 metres are predicted in the 5% AEP event, and up to 0.4 metres in the 1% AEP event in the vicinity of major hydraulic controls (e.g., roadway embankments/culverts such as the Londonderry Road crossing of Torkington Creek). Across the urbanised sections of the catchment, the flood level difference are most commonly less than 0.02 metres in the 5% AEP event and 0.04 metres in the 1% AEP event.

Accordingly, there are some notable differences between flood behaviour defined under ARR1987 versus ARR2019. However, ARR2019 takes advantage of a greater amount of historic rainfall information and employs that latest available research in defining the design flood estimates. Therefore, it is considered that the flood estimates defined under ARR2019 are reasonable and improve upon the flood estimates provided by ARR1987.

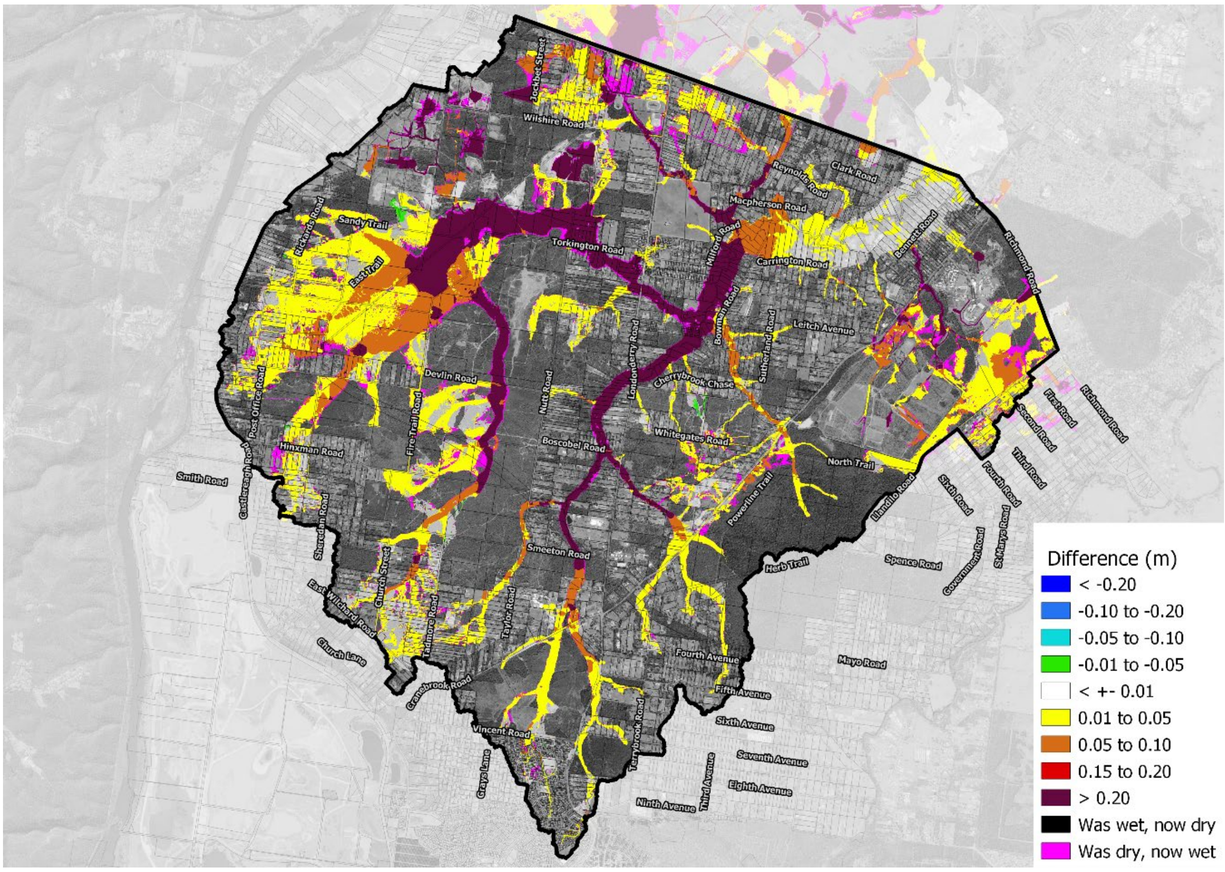

Plate 14 5% AEP Flood level difference comparing ARR1987 and ARR2019.

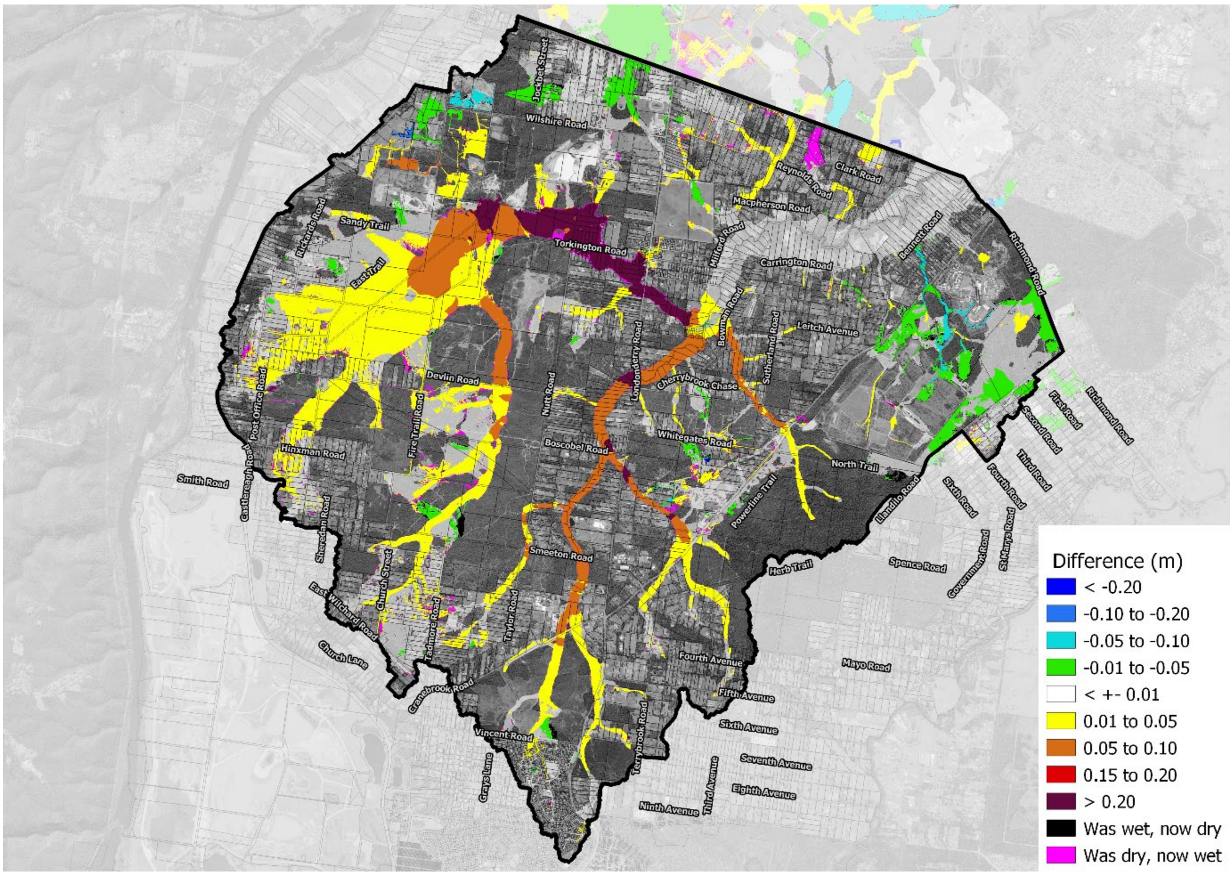

Plate 15 1% AEP Flood level difference comparing ARR1987 and ARR2019.

6.5 Future Catchment Development

The Rickabys Creek study area is largely undeveloped compared to other areas of the Penrith LGA. However, this is primarily a function of the land use zoning which is predominantly RU4 (primary production) as well as a number of environmental conservation zones.

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

Firstly, those areas that are currently underdeveloped and could be developed in the future were identified. This was completed by reviewing land use zoning information relative to contemporary aerial imagery. This review identified two areas where new development could likely occur in the future and consists of:

- Large lots on Perseus Close in Cranebrook (refer **Plate 16**), and
- A future major roadway (refer **Plate 17**).

As the future characteristics of these areas is not known, an assumption that the R2 residential lots would be increased to 65% impervious (in line with the future imperviousness of the remaining R2 lots under future catchment conditions) and that the SP2 Future Roadway would be 75% impervious (including allowance for paved surface and grassed nature strip).

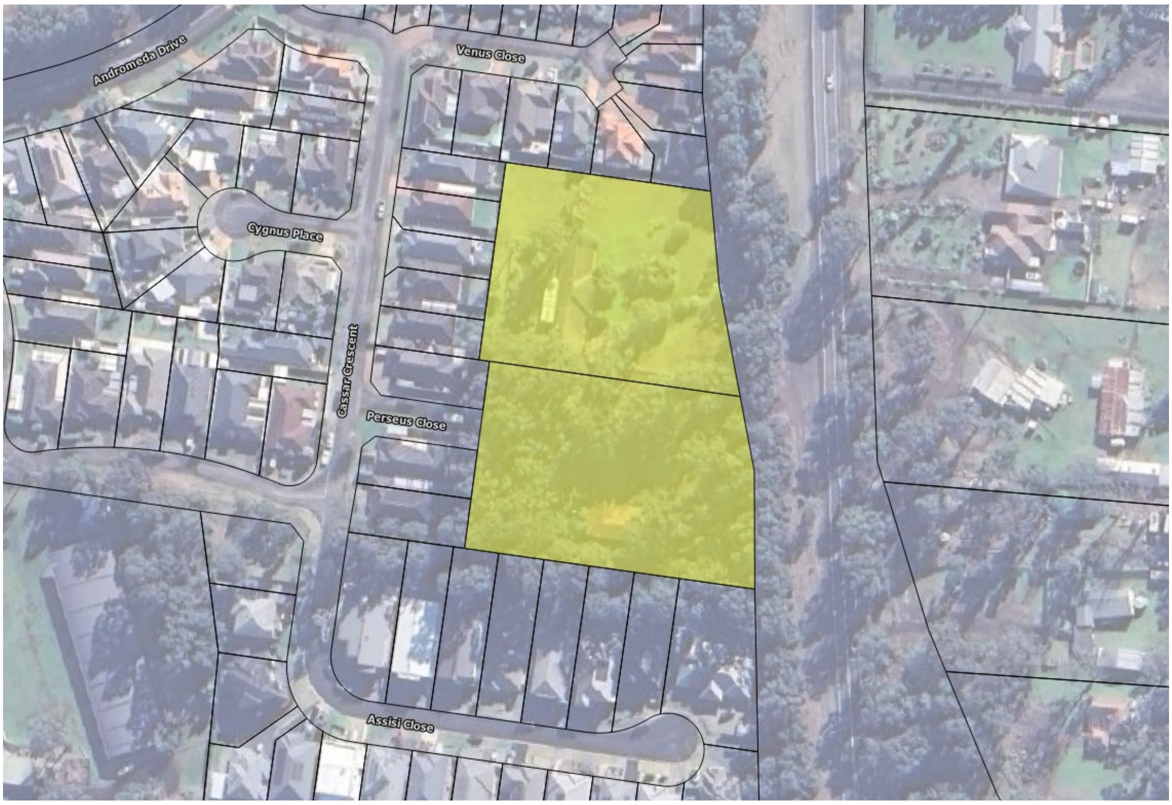

Plate 16 Area of Cranebrook that was assumed to be developed as part of future catchment development assessment

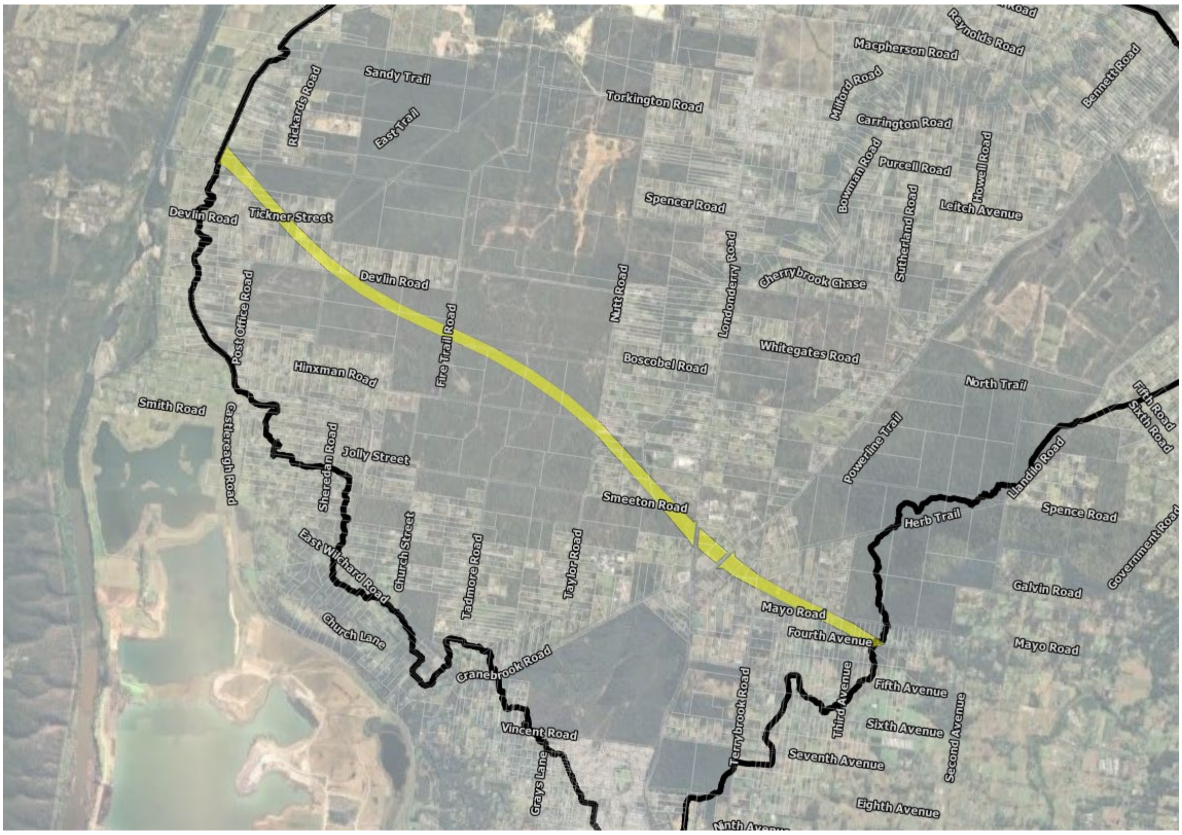

Plate 17 SP2 Future Road LEP zone that was assumed to be developed as part of future catchment development assessment

For the balance of the study area (i.e., those areas that are already developed but where intensification of development may occur in the future), it was assumed that the current impervious proportion would increase by 15% within R2 zoned areas, and 10% within RU4, RU5 and C4 zoned areas.

The updated total impervious areas for each subcatchment were subsequently adjusted to an effective impervious area using the same 0.70 adjustment factor that was adopted for the base design simulations.

The updated impervious values were applied to an updated "ultimate catchment development" version of the WBNM model. The updated model was used to re-simulate the 5% AEP, 1% AEP, 0.5% AEP storms as well as the PMF under potential future catchment development conditions.

Peak discharges extracted from the results of the revised hydrologic assessment are presented in **Appendix P**. Peak discharges for current catchment development conditions are also included in **Appendix P** for comparison.

The discharge comparison indicates that the future catchment scenario is generally predicted to generate small increases in peak discharges across all flood events, viz:

- An average increase in peak discharge of 3.6% in the 5% AEP event
- An average increase in peak discharge of 4.1% in the 1% AEP event
- An average increase in peak discharge of 4% in the 0.5% AEP event
- An average increase in peak discharge of 1.6% in the PMF event

However, some reductions in peak discharges are also predicted within subcatchments along some tributaries. This is a result of the quicker hydrologic response of some subcatchments with a higher impervious proportion, and changes in relative timing to other contributing subcatchments, allowing flows to run off quicker and avoid peak flows coinciding.

The future catchment development discharge hydrographs were also applied to the TUFLOW model to confirm the nature and extent of potential changes in flood behaviour associated with future development. Flood level difference mapping was prepared for the 1% AEP event and is presented in **Figure P1** in **Appendix P**.

The difference map shows that future development is predicted to generate typical increases in 1% AEP flood levels of between 0.02 and 0.03 metres along watercourses passing through, and downstream of RU4 zoned land, as well as in the vicinity of the SP2 Future Road. Some reductions in peak flood level of 0.02 metres are predicted along the channel passing through Cranebrook (from Chausson Place to Vincent Road) as well as within the main Rickabys creek channel between Smeeton Road and Carrington Road.

The results of the revised hydrologic and hydraulic modelling indicate that potential future development across the study area is only predicted to have relatively small impacts on existing flood behaviour, which is primarily a result of the limited future development potential based on the current land use zoning.

6.6 Climate Change Analysis

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

It was considered important to provide an assessment of the potential impact that climate change induced rainfall intensity increases may have on the current flood risk across the study area. In this regard, the results of the 0.5% AEP and 0.2% AEP flood were compared to the results from the 1% AEP flood to gain an appreciation of the impacts of the rainfall increases. The 0.5% AEP rainfall reflects an 8% average increase relative to current 1% AEP rainfall intensities, while the 0.2% AEP rainfall reflects a 23% increase relative to current 1% AEP rainfall intensities. Based on information contained in the NSW Government's Flood Risk Mangement Guideline: *Understanding and Managing Flood Risk* (2023), this is a little lower than the RCP4.5 2090 projection (12.7% increase in rainfall) and the RCP8.5 2090 rainfall projection (26% increase in rainfall). However, it provides an indicative understanding of the sensitivity of existing 1% AEP results to rainfall increases.

Flood level difference mapping was prepared to quantify the impacts that an 8% and 23% increase in rainfall would have on current 1% AEP flood level estimates. The difference mapping was prepared by subtracting the peak 1% AEP flood levels from the 0.5% and 0.2% AEP flood levels. The difference mapping is presented in **Plate 18** and **Plate 19**.

The difference maps in **Plate 18** and **Plate 19** show that the rainfall increases will increase current 1% AEP flood level estimates across most of the catchment. An 8% increase in rainfall is predicted to increase 1% AEP flood levels by at least 0.03 metres along most watercourses, including through the urbanised portions of the study area.

The 23% increase in rainfall is typically predicted to increase existing 1% AEP flood levels by more than 0.15 metres along main watercourses, and up to 0.07 metres along the main drainage flow paths through the urbanised portion of the study area.

Accordingly, the outcomes of the climate change assessment show that increases in rainfall associated with climate change have the potential to produce a notable increase in the severity of flooding across the study area. However, across most of the urbanise sections of the catchment, the impact is less significant.

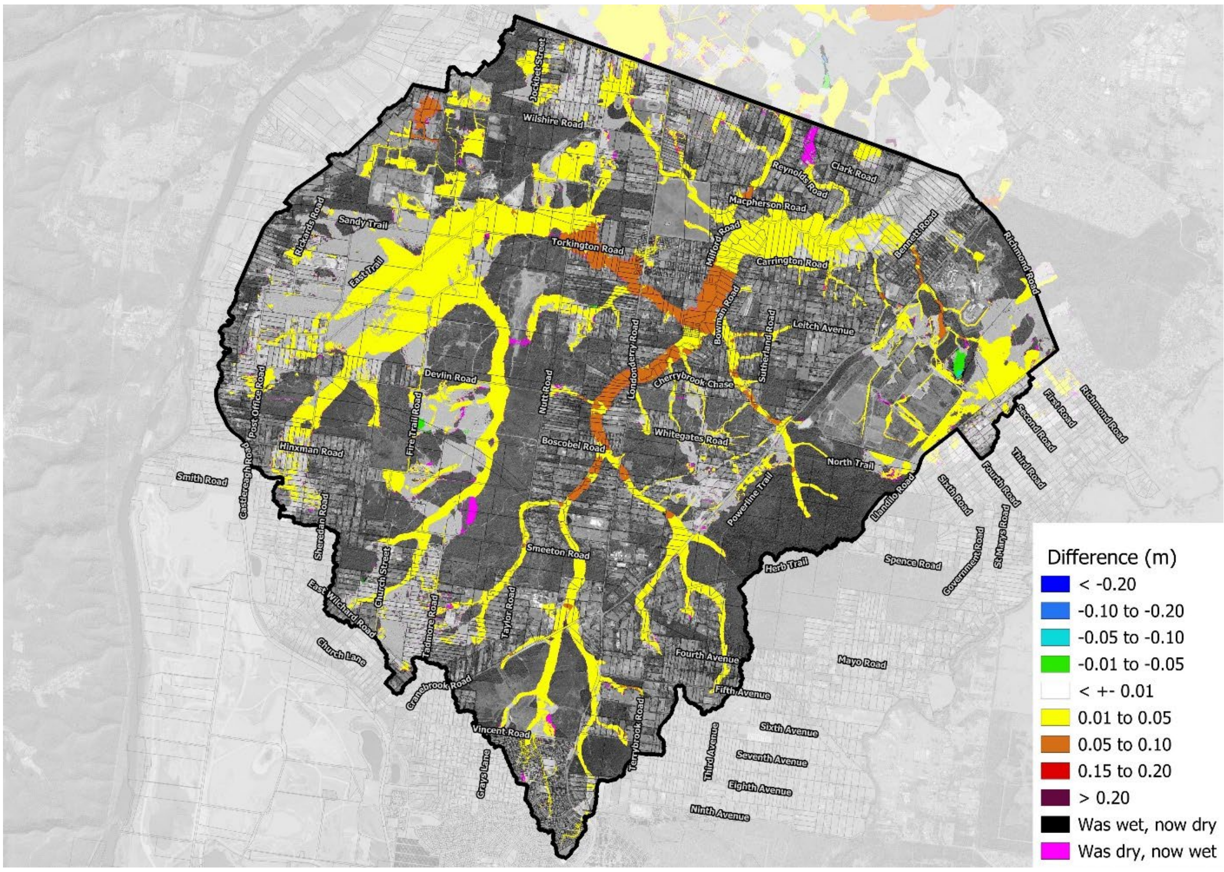

Plate 18 1% AEP Flood level difference map with 8% Increase in rainfall.

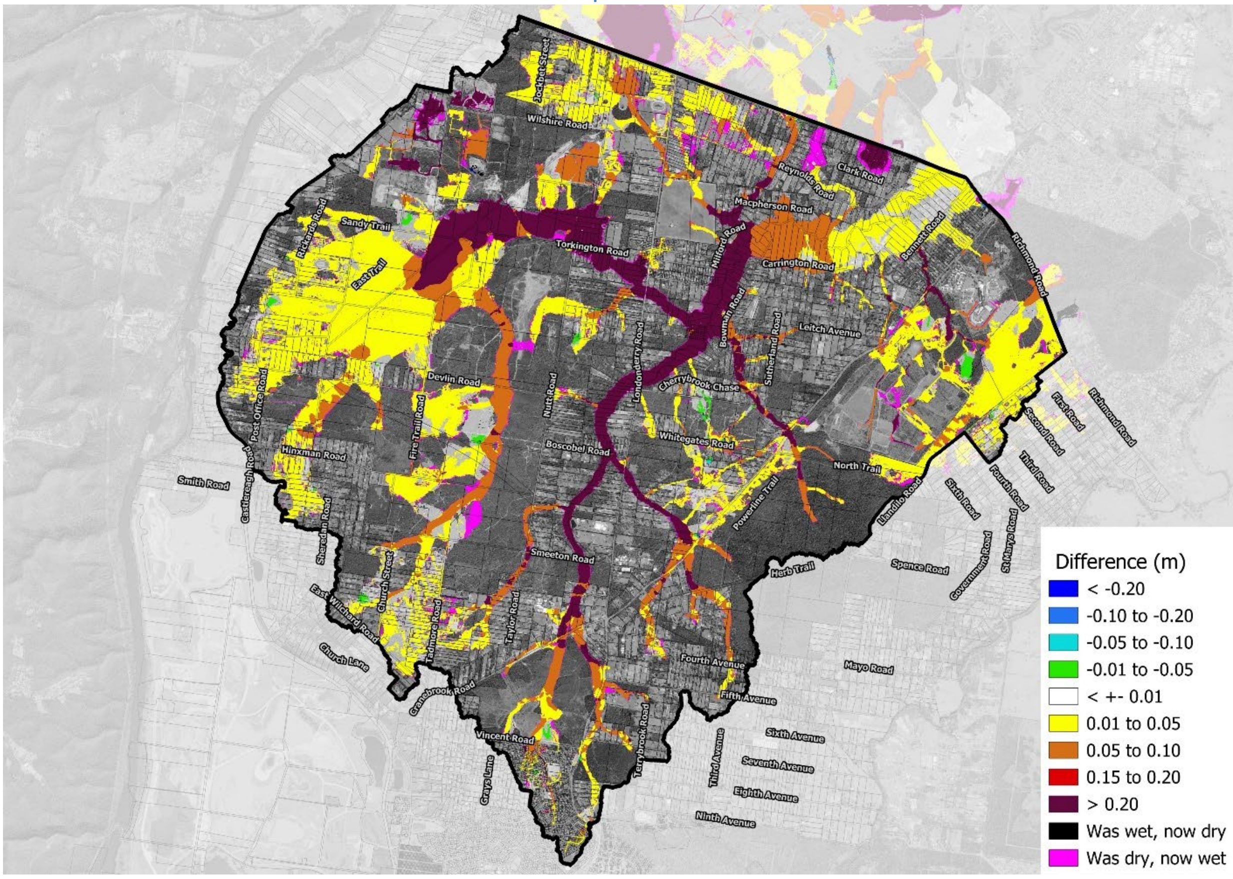

Plate 19 1% AEP Flood level difference map with 23% Increase in rainfall.

7 Flood Planning Information

7.1 Overview

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

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

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

7.2 1% AEP Flood Selection

As discussed, flood planning levels are derived by combining a "defined flood event" (DFE) with a "freeboard". Historically, Penrith City Council has defined the DFE as the 1% AEP flood and has applied a 0.5 metre freeboard to the 1% AEP flood level to define the FPL. However, the recently revised Clause 5.21 of the Penrith City Council LEP, no longer incorporates this strict definition of the FPL, instead only referring to the FPA. A discussion on deriving the FPA is provided in the 'Considering Flooding in Land Use Planning' (NSW Government, 2021) guideline. The guideline mentions the 'appropriateness' of adopting the 1% AEP flood event plus 0.5m freeboard as the 'starting point' for determining the FPA. However, it leaves open the possibility of Council adopting a different 'Defined Flood Event' (DFE) and/or freeboard for determining the FPA (including consideration of more extreme floods where there is a significant increase in flood risk/hazard).

A review of the design flood levels (refer to **Table 16**) shows that the flood risk does not escalate significantly during floods rarer than the 1% AEP event (i.e., peak flood levels typically increase by less than 0.15 metres during the 0.2% AEP flood). As a result, adopting a flood rarer than the 1% AEP event is not considered necessary, with the 1% AEP event adopted as the DFE. However, as discussed in Section 5.3.1, the downstream sections of the catchment also have the potential to be inundated during large Hawkesbury River floods. Therefore, from a flood planning perspective, it was considered important to not only define 1% AEP flood levels from local catchment flooding but also include 1% AEP flood levels as a result of Hawkesbury River flooding.

As outlined in Section 5.3.1, it was not considered appropriate to assume a 1% AEP flood was occurring in the Hawkesbury River at the same time as a 1% AEP local catchment flood due to the significant differing characteristics of the contributing catchments. To overcome this, an approach which creates a design flood “envelope” comprising the following events was adopted:

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

Accordingly, revised 1% AEP depth, level and velocity maps were prepared based upon the combined local catchment and Hawkesbury River results and are presented in **Figures 58**, **59** and **60** respectively. Flood hazard and hydraulic category mapping was also prepared for the combined 1% AEP results set and are provided in **Figures 61** and **62** respectively.

7.3 Flood Planning Area

Flood Planning Levels (FPLs) are an important tool in the management of flood risk and are derived by adding a freeboard to the “Defined Flood Event” (DFE). 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.

7.3.1 Defined Flood Event

Historically, Penrith Council has established the DFE as the 1% AEP flood and has applied a 0.5 metre freeboard to the 1% AEP flood level to define the FPL. However, Clause 5.21 of the LEP no longer incorporates a strict definition of the FPL, instead only referring to the FPA. A discussion on deriving the FPA is provided in the ‘Considering Flooding in Land Use Planning’ guideline. The guideline mentions the ‘appropriateness’ of adopting the 1% AEP flood event plus 0.5m freeboard as the ‘starting point’ for determining the FPA, however, it leaves open the possibility of Council adopting a different ‘Defined Flood Event’ (DFE) and/or freeboard for determining the FPA (and therefore the extent of application of flood related development controls).

A review of the 0.2% AEP flood levels across the catchment shows that they are generally no more than 0.2 metres higher than the peak 1% AEP flood levels and are most commonly no greater than 0.1 metres higher. Therefore, there does not appear to be a significant escalation of flood risk beyond the 1% AEP flood. As a result, the 1% AEP flood was considered suitable to serve as the DFE for mainstream flooding.

7.3.2 Freeboard

Freeboard is a factor of safety that is used to account for uncertainties in deriving the DFE levels. Freeboard is used to account for the following uncertainties:

- Model parameter uncertainty
- "Local" factors that can't be explicitly represented in the computer modelling (e.g., increases in flood levels around local features such as power poles), and
- Wave action (e.g., wind, boat or car induced waves).

As noted above, Council currently adopts a 0.5 metre freeboard for setting flood planning levels for mainstream/riverine flooding. To understand the suitability of the 0.5 metre freeboard to the Rickabys Creek catchment, the results of the sensitivity assessment were further reviewed to quantify the uncertainty associated with model parameters. This interrogation indicated that peak 1% AEP flood levels did not increase by more than 0.2 metres at most locations.

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 cars would typically be operating at low speeds. As shown in **Plate 20**, under these circumstances, the waves generated by cars are unlikely to exceed 0.15 metres and dissipate significantly in height by the time the wave reaches the edges of the road. Therefore, a wave action allowance of 0.15 metres is considerd to be sufficient.

Plate 20 Example of cars driving through flood waters and generating waves

Surges in water level around local features are likely be very localised and a 0.15 metre allowance was considered reasonable.

All of the freeboard components are included in **Table 20**. A "probability factor" was also included to reflect that the local water level surges and wave action are unlikely to occur at the same time and at the same location. **Table 20** provides the freeboard components and the total freeboard that was determined to be suitable for the study area. This determined that a mininum freeboard of 0.4 metres is suitable for the catchment.

Table 19 Adopted Freeboard

Freeboard Element	Allowance (m)	Probability	Freeboard Factor (m)
Uncertainties in DFE flood levels	0.25	1.0	0.25
Local water surge	0.15	0.5	0.08
Wave action	0.15	0.5	0.08
		Total Freeboard	0.40

As noted above, Penrith City Council currently implements a freeboard of 0.5 metres across the LGA. Therefore, to ensure consistency across other parts of the LGA and ease the application of freeboard across the LGA a freeboard of 0.5 metres has been adopted. Although the 0.5 metre freeboard exceeds the suggested freeboard in **Table 20**, it should be recognised that the 0.4 metre value is a minimum value and the 0.5 metre freeboard exceeds this value.

7.3.3 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 or located within a 1% AEP floodway, the flood planning level grid was projected laterally until the flood planning level encountered higher terrain.
  In areas where the FPA was not constrained by higher ground, the lateral extent of the FPA was restricted to the PMF extent, as it was not considered appropriate to extend the FPA beyond the limit of flood liable land.
- In areas where the 1% AEP inundation depths were less than 0.3 metres or a flood fringe area, the flood planning level grid was not projected laterally. This is intended to reflect the reduced flood risk and increased confidence in model results across areas of shallow inundation.

The resulting flood planning area is shown in **Figure 63**.

7.3.4 Flood Control Lots

A preliminary flood control lot layer was prepared by selecting all cadastral lots that were intersected by the flood planning area. That is, if the flood planning area extended across any part of a cadastral parcel it was selected as a flood control lot. This layer was provided directly to Council for internal use.

7.4 Flood Planning Constraint Categories

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

The flood planning constraint categories (FPCC) are summarised in **Table 21**. **Table 21** also summarises how the categories are defined along with the associated planning implication/considerations. In general, a FPCC categorisation of "1" implies a more flood constrained section of land relative to FPCC category "2", and so on.

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

The information contained in **Table 21** was used with the flood modelling outputs (most notably the flood hazard, hydraulic category and emergency response mapping) to prepare the FPCC map shown in **Figure 64**. Also included on **Figure 64** are the current land use zones to gain an appreciation of how the current zoning aligns with the FPCC. The proportion of each land zone that falls within each FPCC was also extracted and is presented in **Table 22**.

The FPCC categories presented in **Figure 64** show that current land use zones are broadly compatible with the level of flood exposure. More specifically, the more highly constrained land (i.e., FPCC 1) typically coincides with areas of open space or low development density (i.e., zones C1, C2, C3, RE1, RU1, as well as the various SP2 special use zones), which is considered to be more compatible land uses. The notable exception to this is the major overland flow path (with floodway/flood storage in the 1% AEP event) that extends from Mozart Place to Andromeda Drive, and Aldebaran Street to Bellatrix Street in Cranebrook. However, these flow paths are primarily located within road reserves (although the flow path does pass through the Corpus Christi School). This categorisation indicates that any flow obstructions have the potential to impact on flood behaviour. Therefore, care will need to be exercised if any new/redevelopment occurs in this area to ensure existing flow paths and storage areas are retained. For example, any filling or buildings that would serve to obstruct flow or remove storage volume should be avoided and the potential to install "open" fencing could be explored.

Table 20 Flood Planning Constraint Categories (AIDR, 2017)

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

FPCC	Sub-Category	Constraint	Implications	Consideration
	E	H6 hazard in events rarer than the DFE	Hazardous conditions may develop in an event rarer than the DFE, which may have implications for the development and its occupants	Consider the need for additional development conditions to reduce the effect of flooding on the development and its occupants
3	-	Outside FPCC 2 but generally below the DFE plus freeboard	Hazardous conditions may exist creating issues for vehicles and people. Structural damage to buildings that meet building standards unlikely because of flooding	Standard land-use and development controls aimed at reducing damage and the exposure of the development to flooding in the DFE are likely to be suitable. Consider the need for additional conditions for emergency response facilities, key community infrastructure and vulnerable users
4	-	Outside of FPCC 3 but within the PMF extent	Emergency response may rely on key community facilities such as emergency hospitals, emergency management headquarters and evacuation centres operating during an event. Recovery may rely on key utility services being able to be readily re-established after an event	Consider the need for conditions for emergency response facilities, key community infrastructure and land uses with vulnerable users

Table 21 Land use zones falling within each FPCC

Zone	FPCC								
	1		2					3	4
	A	B	A	B	C	D	E		
1(a)	2.3%	0.6%	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%	0.6%
1(b)	0.1%	2.2%	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%	1.7%
C1	3.0%	0.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.4%
C2	1.9%	1.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.4%
C4	3.5%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%
DM	2.0%	0.9%	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%	0.8%
R2	3.2%	0.1%	0.0%	0.0%	0.0%	0.0%	0.4%	0.0%	0.2%
R5	3.9%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
RE1	3.1%	0.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.3%
RU1	2.5%	0.9%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%
RU4	1.9%	0.6%	0.0%	0.0%	0.0%	0.2%	0.8%	0.0%	0.3%
RU5	2.5%	0.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	1.0%
SP2 Correctional Centre	3.5%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.3%
SP2 Emergency Services Facility	3.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
SP2 Future Road	2.4%	0.5%	0.0%	0.0%	0.0%	0.1%	0.1%	0.0%	0.7%
SP2 Sewerage System	0.0%	0.3%	0.0%	0.6%	0.0%	2.8%	0.0%	0.0%	0.0%
SP2 Water Supply System	4.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
SP2 Waste or Resource Management	3.0%	0.4%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.5%

Zone	FPCC								
	1		2					3	4
	A	B	A	B	C	D	E		
SP2 Classified Road	3.0%	0.3%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.4%
8	0.2%	2.0%	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%	1.6%

8 Hot Spots Investigation

8.1 General

A detailed analysis of flood behaviour was completed across a number of "hotspots" where a high impact on private properties was predicted. The outcome of this detailed analysis is summarised below and includes the following areas:

- Carrington Road to Torkington Road, Londonderry;
- Torkington Road to Warrina Place, Londonderry;
- Nutt Road to Torkington Road, Londonderry;
- Hinxman Road near Isaac Smith Road, Castlereagh; and
- Boscobel Road west of Londonderry Road, Londonderry.
- Mills and Studley St to Kenmare Rd, Londonderry
- Hercules Close, Arcturus Close to Bellatrix St, Cranebrook

A list of potential flood mitigation measures was also 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 "hot spots". Those mitigation measures could then be shortlisted for a more comprehensive analysis as part of the subsequent floodplain risk management study.

8.2 Flooding "Hot Spots" and Potential Mitigation Measures

8.2.1 Hot Spot 1 - Carrington Road to Torkington Road, Londonderry

As shown on **Figure 46.6** an overland flowpath with H1 hazard (depths of up to 0.3m and velocities of up to 0.4m/s) extends from Carrington Road (at Muscharry Road) in a south-westerly direction through properties to Londonderry Road in the 1% AEP event. The flowpath then continues through properties on the western side of Londonderry Road through to Torkington Road.

The source of flooding is best described as overland flow that overtops the kerb at trapped low points and moves through properties, following depressions in the local terrain. The stormwater capacity shown on **Figure 44.6** (and reproduced on **Plate 21**) indicates that the pipes in this area have a capacity as low as a 10% AEP event. Accordingly, during significant rainfall events, the stormwater capacity is exceeded resulting in the excess water "ponding" and then spilling through properties.

Plate 22 shows the 5%AEP and 1% AEP flood extents through the affected properties and indicates no less than 5 properties experience inundation in the 5% AEP event, and 12 properties are subject to inundation at the peak of the 1% AEP event. **Table 23** shows the typical depths of flow through these properties in the 20% AEP, 5% AEP, 1% AEP and the PMF which can vary from 0.2 metres in the 20% AEP flood to more than 2 metres in the PMF.

Table 22 Typical peak flood depth at hot spot locations

Hot Spot		Typical Flood Depth			
		20% AEP	5% AEP	1% AEP	PMF
1	Carrington Road to Torkington Road, Londonderry (within properties)	0.20	0.15-0.25	0.2-0.3	1.0-2.0
2	Torkington Road to Warrina Place, Londonderry (within properties)	0.4-0.7	0.6-1.0	0.8-1.5	2.0-3.0
3	Nutt Road to Torkington Road, Londonderry (within properties)	1.0-1.6	1.2-1.9	1.5-2.2	3.2-4.1
4	Hinxman Road near Isaac Smith Road, Castlereagh (near dwellings/sheds where over floor inundation has previously been experienced)	0.22	0.31	0.35	0.65
5	Boscobel Road west of Londonderry Road, Londonderry (on the road surface)	0.15-0.5	0.3-0.8	0.4-1.2	2.5-3.4
6	Mills and Studley St to Kenmare Rd, Londonderry	0.5–1.6	1.2–1.9	1.7-2.0	3.0-4.2
7	Hercules Close, Arcturus Close to Bellatrix St, Cranebrook	<0.1	<0.1	<0.1	0.05-0.15

Plate 21 Stormwater capacity in the vicinity of the Carrington Road to Torkington Road hotspot.

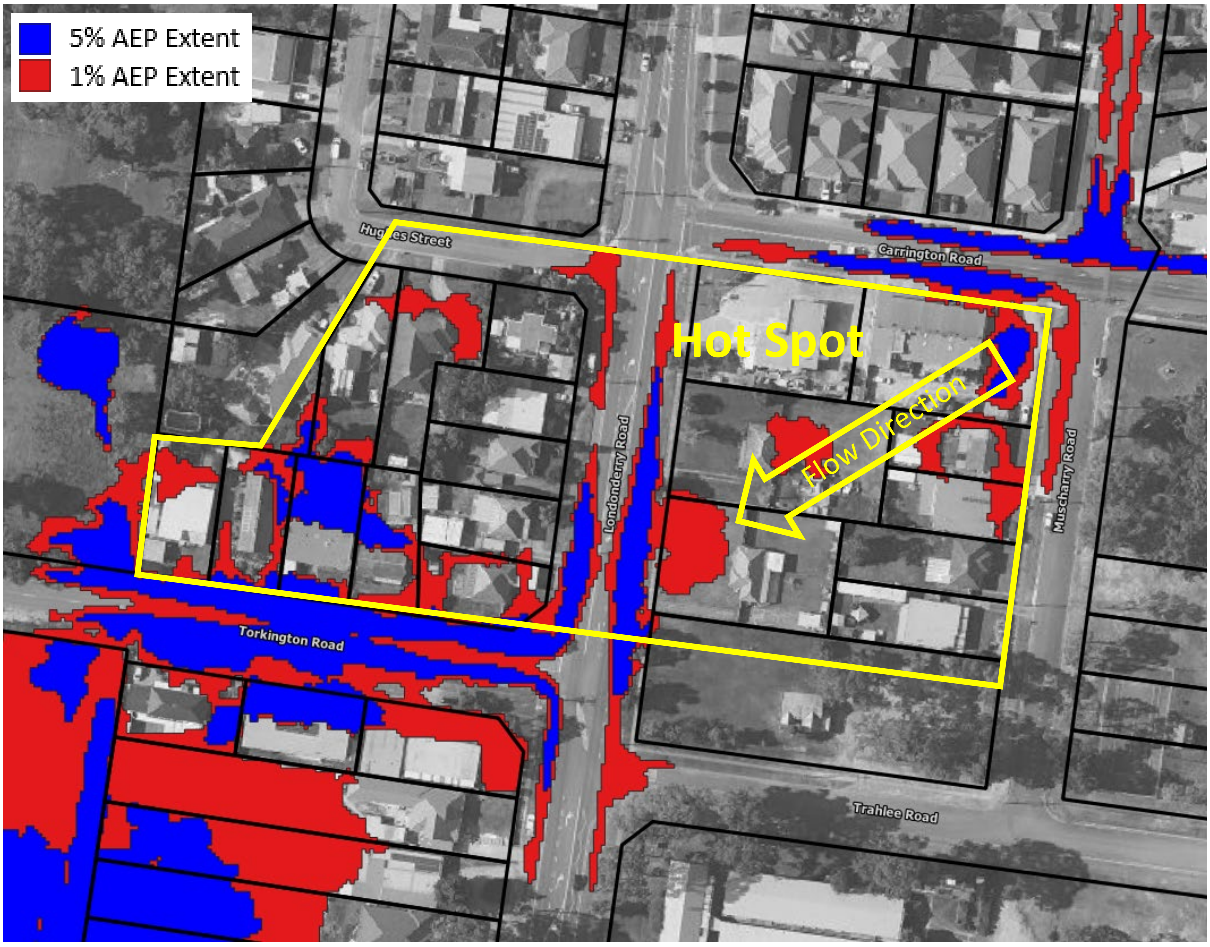

Plate 22 Flood extents in the vicinity of Hotspot 1.

Potential Mitigation Options

As discussed, the inundation problems across this area are primarily associated with the limited capacity of the existing drainage system to manage large flood events. Options available to rectify this limitation are primarily related to increasing 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). Construction of additional stormwater infrastructure is unlikely to produce any adverse impacts to adjoining properties, however, may produce some afflux at the discharge location of the stormwater network. Increasing the conveyance provided within the road reserve could also be explored (e.g., including shallow swales on the nature strip to direct flows away from properties).

8.2.2 Hotspot 2 - Torkington Road to Warrina Place, Londonderry

As shown on **Figure 46.6**, a small channel with a 1% AEP hazard of H4 runs along the rear of properties on Londonderry Road from the southern side of Torkington Road to Warrina Place. This location receives flow from the previously discussed “hot spot”. The channel is overtopped resulting in inundation of properties to the east where a hazard of H3 is predicted and is associated with depths of up to 1 metre and velocities of 0.4m/s. Inspection of the flood levels in this area indicates that this inundation is primarily associated with mainstream floodwater from Torkington Creek ‘backing up’ into the channel and adjacent properties.

Plate 23 shows the 5%AEP and 1% AEP flood extents through the affected properties and indicates 15 properties experience inundation in the 5% AEP event, and 19 properties are subject to inundation in the 1% AEP event. **Table 23** shows the typical depths of flow through these properties in the 20% AEP, 5% AEP, 1% AEP and the PMF which can vary from 0.4 metres in the 20% AEP to 3 metres in the PMF.

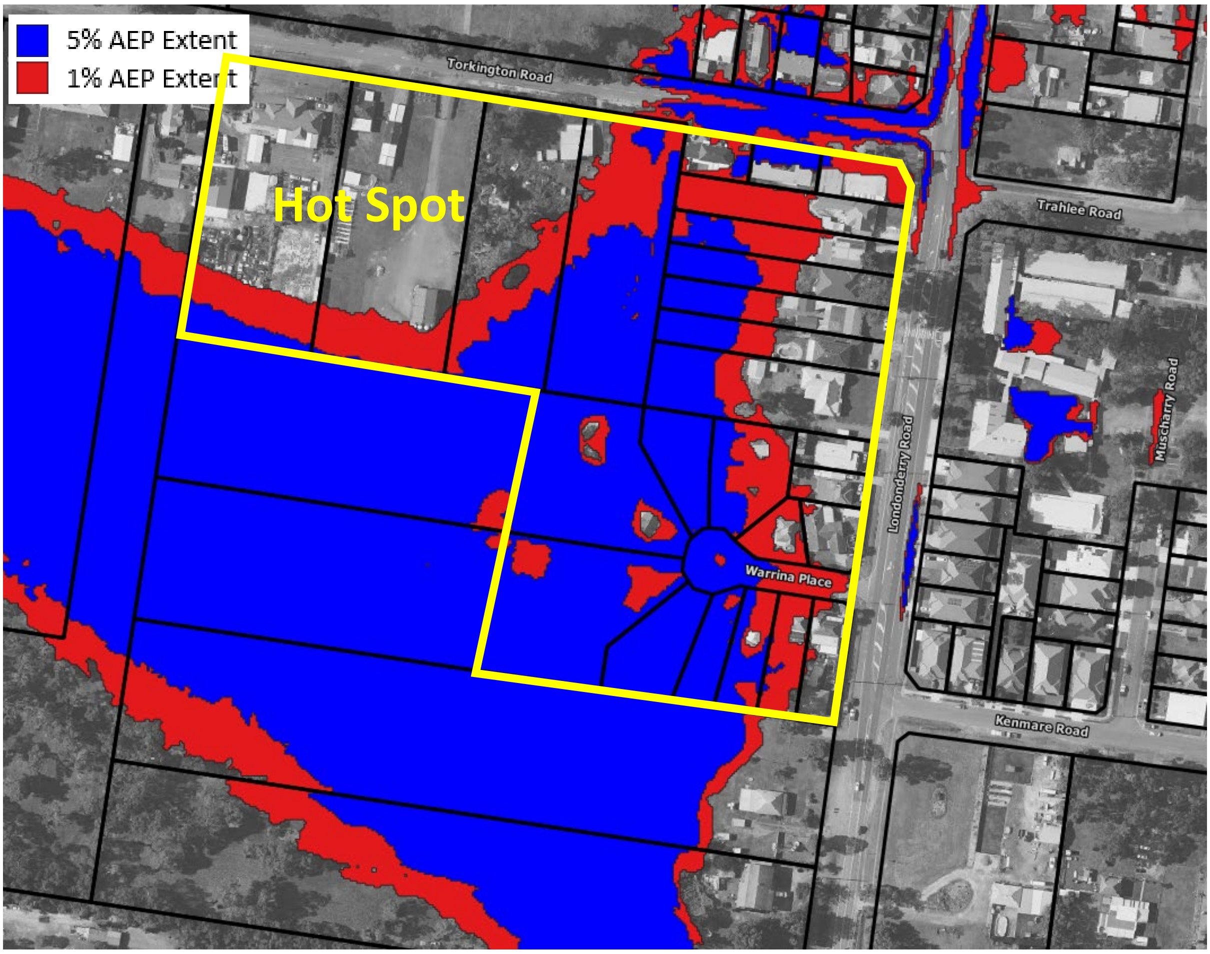

Plate 23 Flood extents in the vicinity of Hotspot 2.

Potential Mitigation Options

As discussed, the inundation is primarily a result of mainstream Torkington Creek floodwater. Two potential options may be able to rectify this limitation:

- Construct a flood storage detention basin within the catchment upstream of this area, such as to the west of Nutt Road; and
- Construction of a levee to protect from the water 'backing up' into properties on Warrina Place and Londonderry Road. An example of the potential alignment of the levee is shown on **Plate 24**. Although this will not protect all properties within the hot spot, it would be intended to protect the majority of properties, with the lowest likelihood of adverse impacts on other surrounding properties or the main Torkington Creek channel.

Both of these options will require works on private property and detailed hydraulic assessment to ensure no adverse impacts to adjacent properties, or the wider Torkington Creek floodplain.

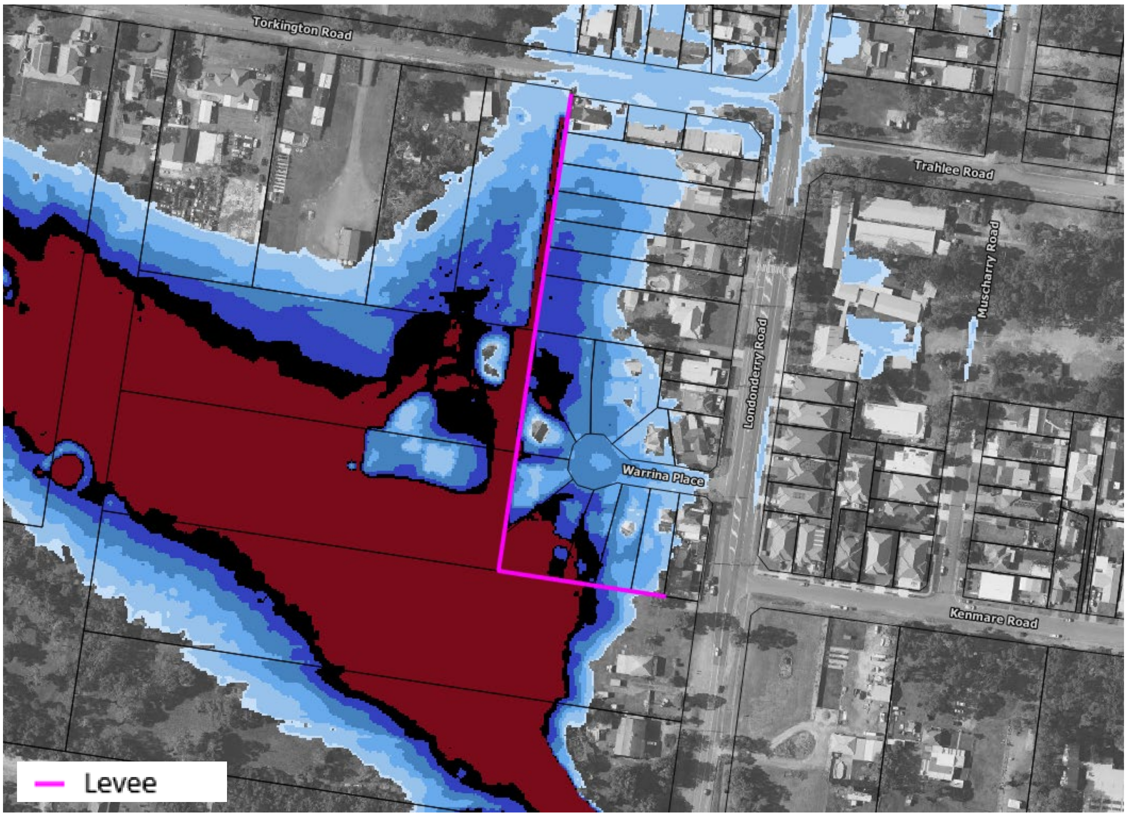

Plate 24 1% AEP depths south of Torkington Road and example of potential levee location.

8.2.3 Hotspot 3 - Nutt Road to Torkington Road, Londonderry

As shown in **Figure 46.5**, Torkington Creek extends through a number of private properties located between Nutt Road and Torkington Road with a peak 1% AEP hazard of up to H5 (although H3/H4 is more common). This high hazard area surrounds multiple dwellings and sheds and would isolate these properties. The Torkington Creek channel through this area is ill defined with only a relatively small, narrow channel which is significantly undersized for the potential flows that pass through this area (see **Plate 25**).

Plate 26 shows the 5%AEP and 1% AEP flood extents through the affected properties and shows 11 properties experience inundation in the 5% AEP and 1% AEP events. **Table 23** shows the typical depths of flow through these properties in the 20% AEP, 5% AEP, 1% AEP and the PMF which can vary from 1 metre in the 20% AEP to 4 metres in the PMF.

Potential Mitigation Options

Given the area to the west of this hot spot is natural bushland, a detention basin could be constructed to temporarily store floodwater and reduce the extent and level of inundation within downstream properties with little adverse impact on any private property or local roadways. This would also ensure Nutt Road and Torkington Road would be trafficable for longer before inundation and floodwaters would likely recede from the roadway surface earlier after a flood event (with appropriate outlet design). Excavation and formalisation of a larger creek channel could also be investigated to increase the in-bank flow carrying capacity and reduce the overbank flows that occur within adjacent properties. An intensive flood awareness campaign could also be focussed on the residents of the heavily impacted properties to raise awareness of the flood risk and appropriate actions to take in the event of a flood (e.g., sandbagging). Strictly enforced development controls should also be maintained.

Plate 25 Torkington Creek channel at Nutt Road (looking east). Photo courtesy of Google Streetview©

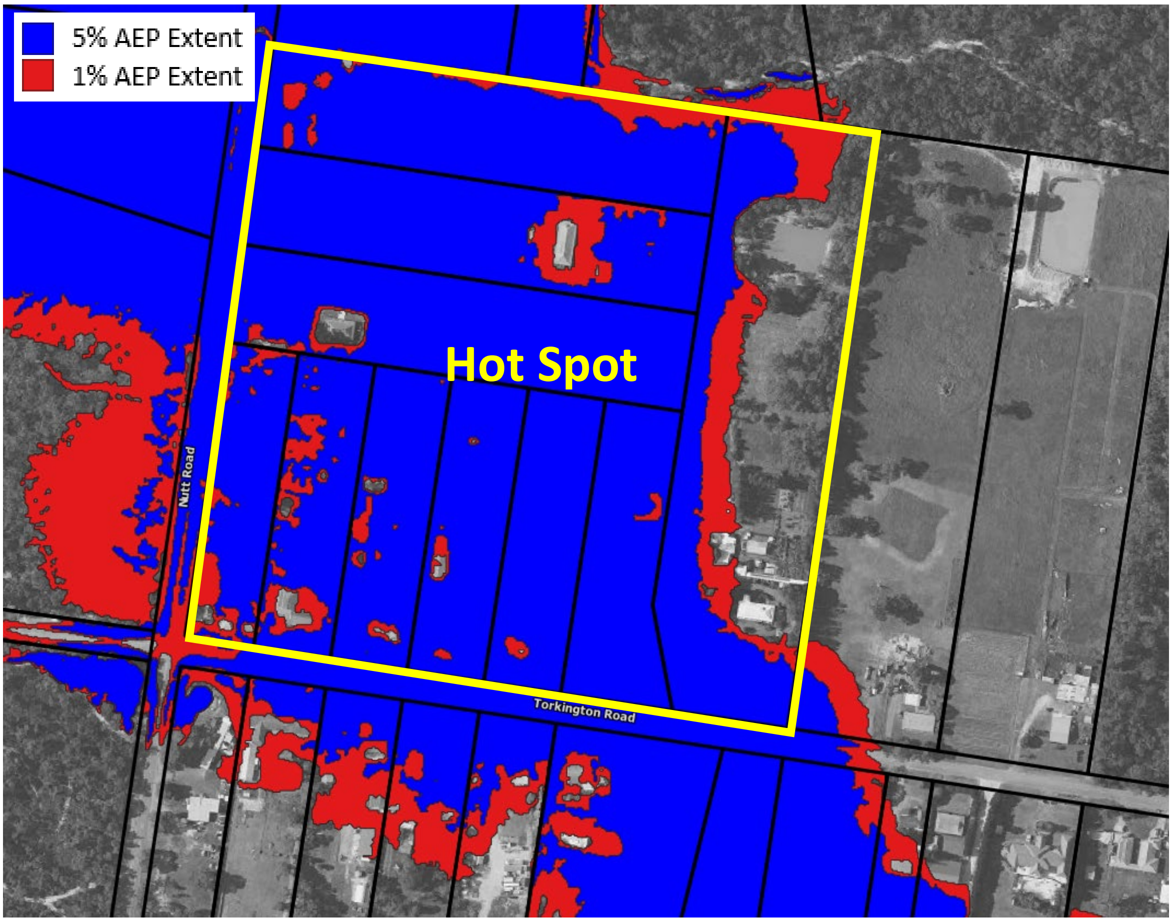

Plate 26 Flood extents in the vicinity of Hotspot 3.

8.2.4 Hotspot 4 - Hinxman Road near Isaac Smith Road, Castlereagh

As shown in **Figure 46.8**, a hazard of up to H2 is predicted within properties on the northern side of Hinxman Road, together with inundation across Hinxman Road. This inundation surrounds a number of dwellings and sheds and over floor inundation was reported in this area in the 2012 flood based on community survey responses collected as part of the *'Torkington Creek, Londonderry - Flood Investigations'* (Molino Stewart, 2013). The inundation is produced by flow exceeding the capacity of the culverts crossing Hinxman Road, causing water to spill across the roadway surface, as well as laterally along Hinxman Road, before spilling through the properties on the northern side of Hinxman Road.

Plate 27 shows the 5%AEP and 1% AEP flood extents in the vicinity of Hinxman Road and indicates that inundation is predicted across the front of five properties, including around buildings and sheds. **Table 23** shows the typical depths of flow for the 20% AEP, 5% AEP, 1% AEP and the PMF in the vicinity of dwellings/sheds that have previously recorded over flood inundation. This shows that depths can vary from greater than 0.2 metres in the 20% AEP to 0.65 metres in the PMF.

Plate 27 Flood extents in the vicinity of Hotspot 4.

Potential Mitigation Options

Given the local flood behaviour, two primary options are available to rectify this inundation:

- Increase the capacity of the existing culverts across Hinxman Road; and,
- Constructing a formalised roadside swale/channel on the northern side of Hinxman Road to convey flow to the drainage swale heading north near Isaac Smith Road (there is currently built-up ground at the upstream end of this drainage swale – see **Plate 28**).

Both of these options could be undertaken within the road reserve and are unlikely to produce any significant adverse impacts to properties, and little to no impact to the flood levels/behaviour within the drainage swale would be anticipated.

Plate 28 Location of built up ground near the upstream end of the drainages wale. Photo courtesy of Google Streetview ©

8.2.5 Hotspot 5 - Boscobel Road west of Londonderry Road, Londonderry

As shown in **Figure 46.9**, Boscobel Road becomes inundated with peak 1% AEP floodwater producing a widespread hazard of H5 across two adjacent locations (corresponding to the confluence of Rickabys Creek and a slightly smaller tributary). The creek crossings of Boscobel Road are located roughly 200 and 300 metres west of Londonderry Road respectively. Although the risk posed by this hazard on road users is significant, there is also a high risk to a dwelling located directly adjacent to the high hazard area, and an additional two dwellings which are surrounded by the high hazard area and would have no means to evacuate at the peak of the flood.

Plate 29 shows the 5%AEP and 1% AEP flood extents through the affected properties and indicates no less than 5 properties experience inundation in the 5% AEP and 1% AEP events, as well as significant inundation of the Boscobel Road surface along the frontage of these properties. **Table 23** shows the typical depths of flow through these properties in the 20% AEP, 5% AEP, 1% AEP and the PMF which can vary from 0.15 metres in the 20% AEP to 3.4 metres in the PMF.

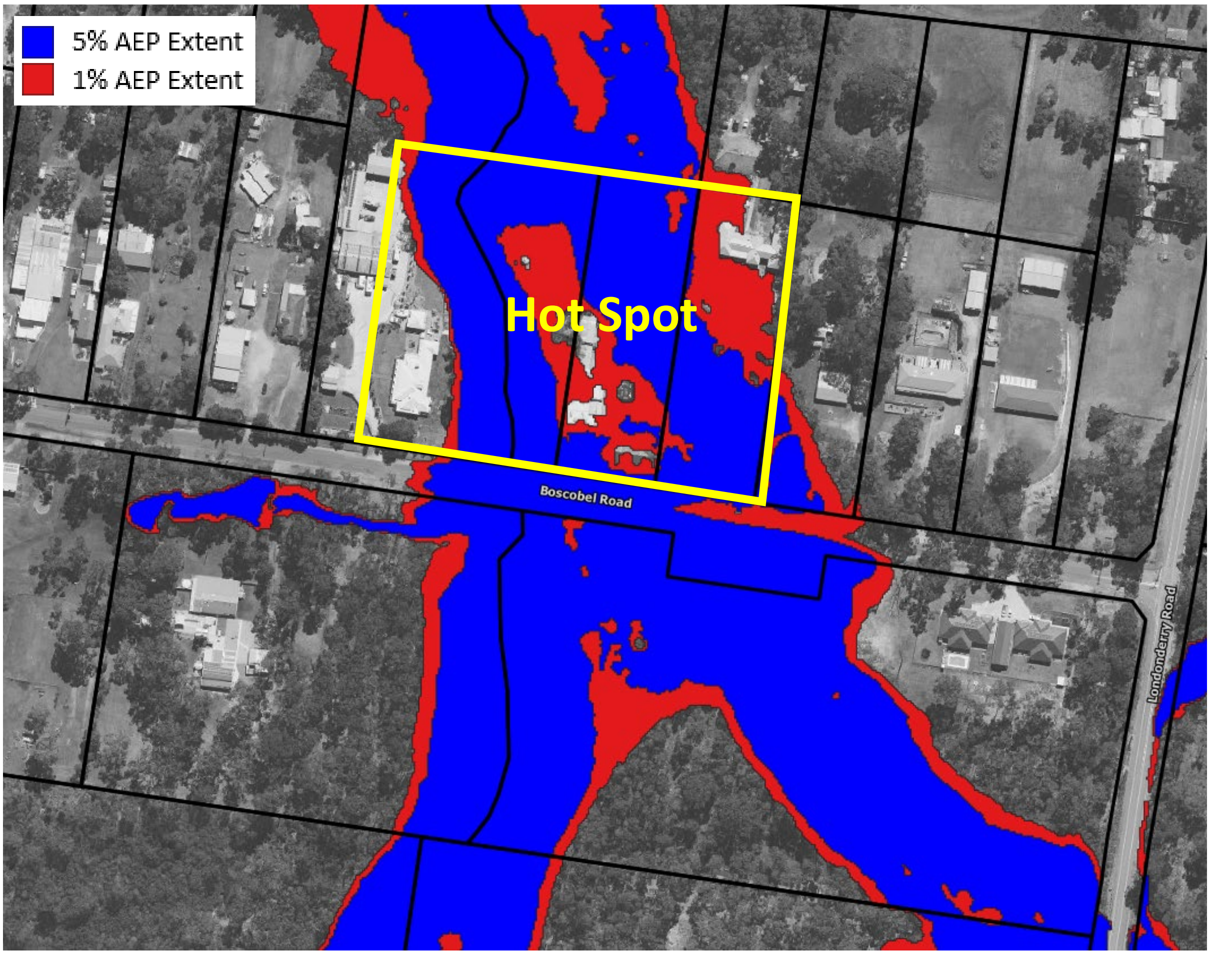

Plate 29 Flood extents in the vicinity of Hotspot 5.

Potential Mitigation Options

Given the high flows that are predicted at this location along both watercourses, there would be limited options available to modify flood behaviour in the immediate vicinity of Boscobel Road. Therefore, risk management would need to focus on preventing access along Boscobel Road at these locations by means of warning signs (activated when water levels reach a pre-defined level) or an automatically operated flood gate/barrier to prevent vehicular access. An intensive flood awareness campaign could also be focussed on the residents of the heavily impacted properties to raise awareness of the flood risk and appropriate actions to take in the event of a flood. Strictly enforced development controls should also be maintained. The subject properties may also be eligible for voluntary house purchase.

8.2.6 Hotspot 6 - Mills and Studley St to Kenmare Rd, Londonderry

As shown in **Figure 46.6**, a number of properties within this area experience a 1% AEP hazard of up to H5, including sections of Mills Road and Kenmare Road. The inundation within the hotspot is produced by the main Rickabys Creek watercourse, as well as three smaller tributaries that drain into Rickabys Creek. The inundation during the 1% AEP events is predicted to isolate multiple properties, with 11 dwellings in the area impacted by a hazard of H3 or higher at the peak of the 1% AEP event. This hazard categorisation suggests that there is a potential risk to life for occupants and no means to evacuate at the peak of the 1% AEP flood.

Plate 30 shows the 5%AEP and 1% AEP flood extents through the hotspot locality and indicates no less than 12 rural residential properties are predicted to experience significant inundation. Inundation is also predicted to extend across a number of smaller allotments within the south-western portion of the hotspot, however these are currently undeveloped. **Table 23** shows the typical water depths through these properties can vary from 0.5 metres in the 20% AEP to more than 4 metres in the PMF.

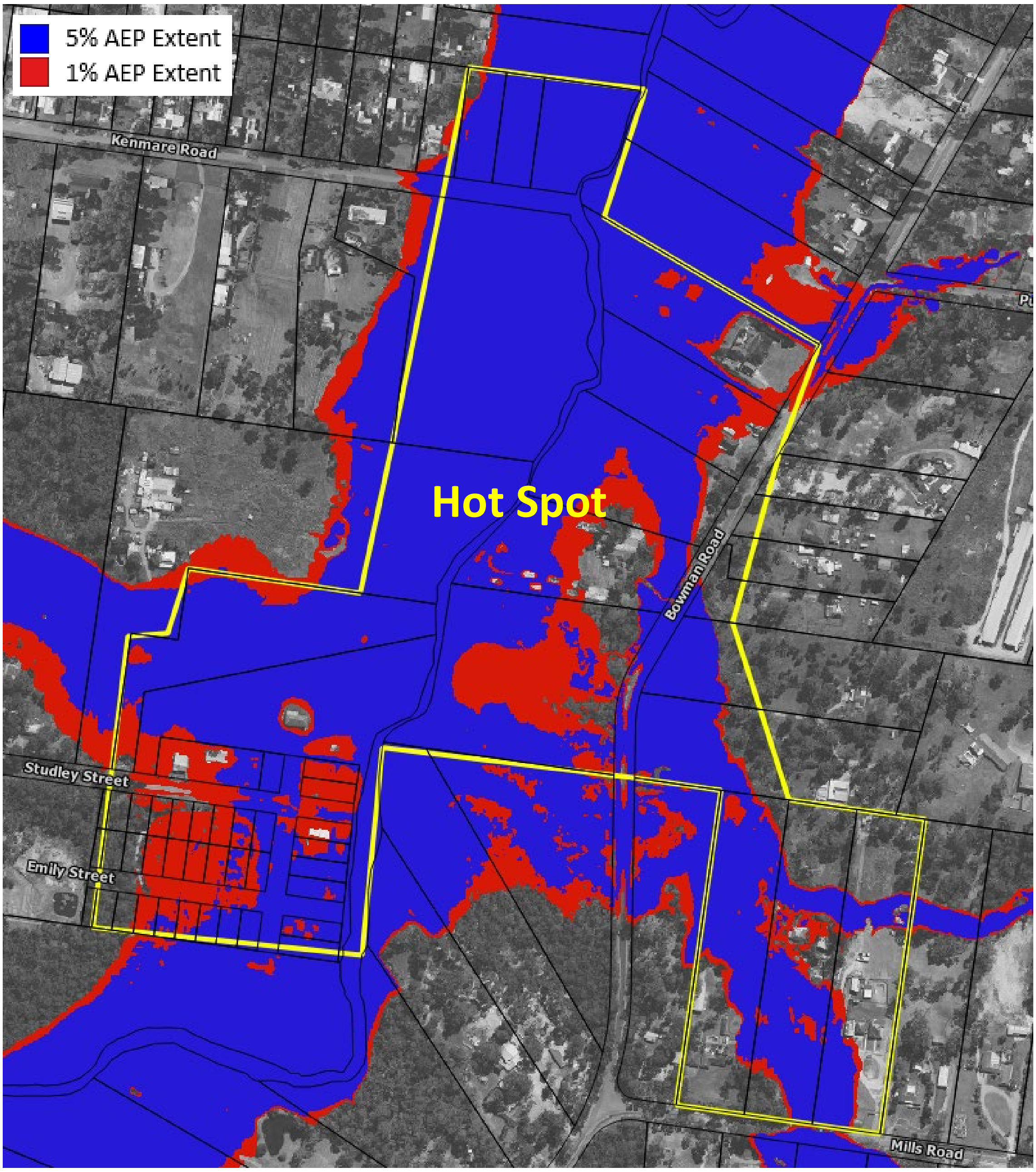

Plate 30 Flood extents in the vicinity of Hotspot 6.

Potential Mitigation Options

Given that Rickabys Creek as well as several tributaries influence flooding across this hot spot, developing structural mitigation measures to significantly reduce this flooding is challenging. However, as Mills Road experiences a minimum hazard of H4 in the 1% AEP event, elevating the roadway, together with larger culverts may serve to improve emergency response outcomes. Local levee systems could also be explored for those properties that are predicted to experience higher flood hazard.

A flood awareness campaign could also be focussed on the residents of the heavily impacted dwellings to raise awareness of the flood risk and appropriate actions to take in the event of a flood. Strictly enforced development controls should also be maintained moving forward. Voluntary house purchase could also be explored for the higher flood hazard properties.

8.2.7 Hotspot 7 - Hercules Close, Arcturus Close to Bellatrix St, Cranebrook

As shown in **Figure 46.14**, trapped low points are present on Hercules Close and Arcturus Close at Cranebrook. Although not explicitly shown on **Figure 46.14** (due to the filter criteria adopted as part of the results presentation), overflow from these trapped low points commonly occurs (in events as frequent as the 20% AEP) and moves through properties towards Bellatrix Street. Although the depths and velocity of this floodwater is not high, it produces nuisance flooding to a number of residential properties (e.g.: during the March 2022 event, a resident was required to place sandbags across their driveways to prevent inundation of the garage – see **Appendix G**).

Plate 31 shows the 5%AEP and 1% AEP (unfiltered) flood extents through the affected properties and indicates no less than 17 properties are predicted to experience shallow inundation around dwellings in the 5% AEP and 1% AEP events. The shallow nature of inundation is confirmed by reference to **Table 23** which shows the typical depths of flow through these properties varies from less than 0.1 metres in events up to and including the 1% AEP to a maximum of 0.15 metres in the PMF.

Potential Mitigation Options

Given the flood behaviour is driven by floodwater spilling from trapped low points within roadways, the most effective option to mitigate the flooding would be to improve the efficiency and/or capacity of the existing stormwater system. This could include the installation of additional stormwater pits, measures to reduce blockage of the existing stormwater infrastructure (e.g., regular maintenance) or upgrading the stormwater pipes. Some local regrading within the nature strips, and/or along the pedestrian walkway between Arcturus Close and Scorpius Place could also be used to reduce the spill from the trapped low points, particularly in the more frequent flood events.

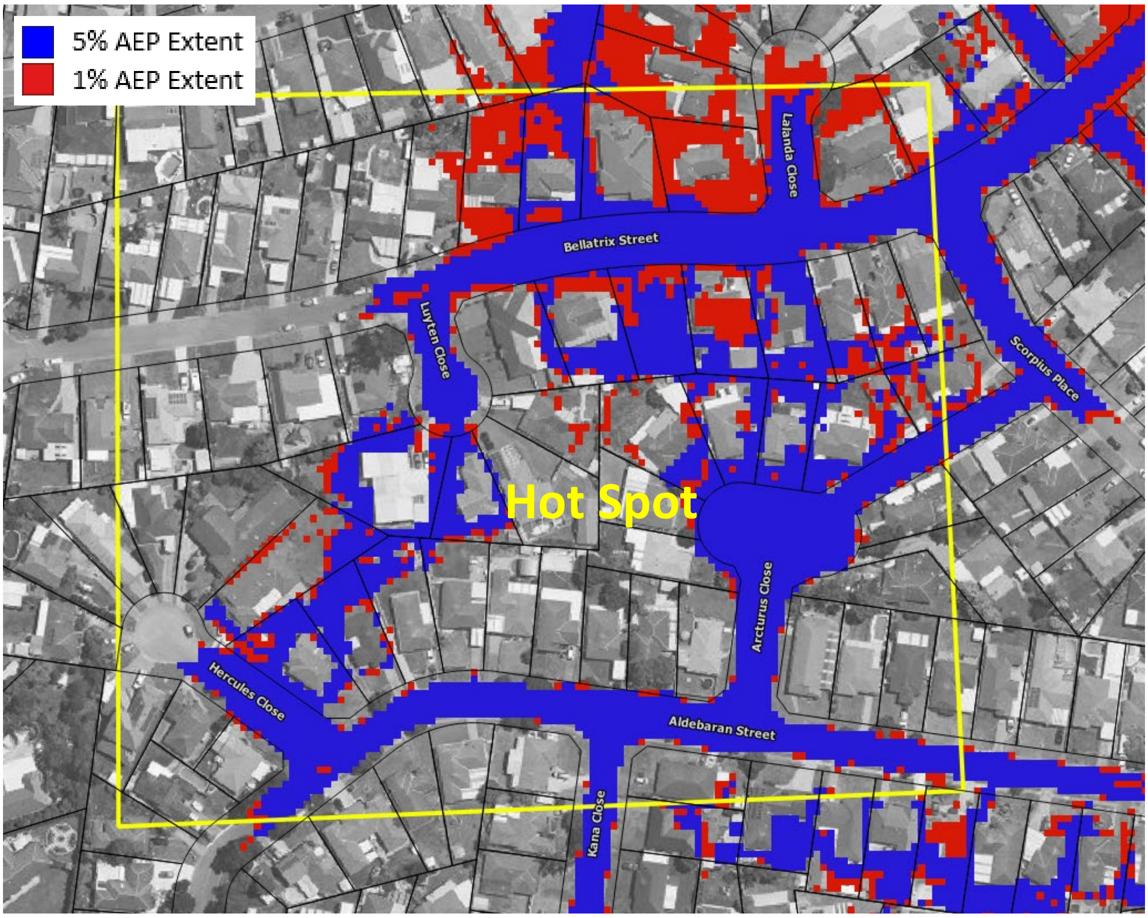

Plate 31 Flood extents in the vicinity of Hotspot 7.

9 Conclusion

This report documents the outcomes of investigations completed to quantify overland and mainstream flood behaviour across the Rickabys Creek 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 WBNM hydrologic model to define rainfall-runoff process and a TUFLOW hydraulic computer flood model that was used to simulate the movement of floodwaters. The TUFLOW model included a full representation of the stormwater drainage system and hydraulic structures (e.g., culverts).

The computer models were validated using historic rainfall and reported descriptions of flood behaviour that were provided by the community for floods that occurred in 2012, 2020, 2021 and 2022. The model was also verified against alternate modelling techniques as well as results presented in other flood-related reports for the catchment.

The validated models were used to simulate the design 0.5EY, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP 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 and roadside swales are exceeded as well as inundation from elevated water levels in the Hawkesbury River. Flooding north of Studley Street is typically dominated by Hawkesbury River backwater levels while flooding south and west of Studley Street is typically dominated by runoff from the Rickabys and Torkington Creek catchments.
- 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 mainstream sections of the catchment, rainfall over a period of 6 - 24 hours will typically produce the worst-case flooding. Accordingly, flooding across the catchment may be produced by relatively short, high intensity thunderstorms through to longer rainfall events that may be generated by east coast lows.
- Inundation of over 1,200 properties is predicted at the peak of the 1% AEP flood (out of a total of 3,335 properties located within the study area). The most notable flooding "hot spots" include:
    - Carrington Road to Torkington Road, Londonderry
    - Torkington Road to Warrina Place, Londonderry
    - Nutt Road to Torkington Road, Londonderry
    - Hinxman Road near Isaac Smith Road, Castlereagh
    - Boscobel Road west of Londonderry Road, Londonderry
    - Mills and Studley St to Kenmare Rd, Londonderry

  - Hercules Close, Arcturus Close to Bellatrix St, Cranebrook

- 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 in the more urbanised portion of the study area, and by more than 10 hours in the more rural portion of the catchment. Roadways directly impacted by elevated water levels in the Hawkesbury River can be impassable for much longer.
- The regional (Hawkesbury-Nepean) emergency evacuation routes that pass through the study area all remain trafficable in events up to and including the 0.5% AEP apart from The Northern Road at Sutherland Road. However, in the PMF, The Northern Road and Londonderry Road are cut at multiple locations.
- A preliminary list of flood risk mitigation measures at flood 'hot spots' has been compiled as part of the study. It is recommended that these measures be considered and, where considered appropriate, assessed in detail as part of the floodplain risk management study.

10 REFERENCES

- Australian Building Codes Board (2022) *National Construction Code*
- Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors) (2019) *Australian Rainfall and Runoff: A Guide to Flood Estimation*, © Commonwealth of Australia (Geoscience Australia).
- BMT WBM (2023) *TUFLOW User Manual*. Version 2023-03-AA.
- Engineers Australia (1987). *Australian Rainfall and Runoff - A Guide to Flood Estimation*. Edited by D. Pilgrim.
- Cardno Lawson Treloar (2006). *Penrith Overland Flow Study, "Overview Study" – Flood Analysis for Central Urban (Zone 1), Northern Rural (Zone 2), Southern Rural (Zone 3)*. Prepared for Penrith City Council.
- Chow, V. T. (1959). *Open Channel Hydraulics*, McGraw-Hill Inc., New York, U.S.A.
- Engineers Australia (1987). *Australian Rainfall and Runoff - A Guide to Flood Estimation*. Edited by D. Pilgrim.
- Engineers Australia. (2012). *Australian Rainfall & Runoff – Project 15: Two Dimensional Modelling in Urban and Rural Floodplains*, Stage 1 & 2 Report, P15/S1/009. Editors: Mark Babister & Cathie Barton. ISBN 978-085825-9850
- Engineers Australia (2015), Blockage of Hydraulic Structures – Blockage Guidelines. Prepared by W. Weeks & E. Rigby.
- J. G. Nelson Pty Ltd (1985). *Londonderry Trunk Drainage*. Prepared for Land Commission of NSW.
- Molino Stewart (2006). *Torkington Creek, Londonderry - Flood Investigations*. Final Report. Prepared for Penrith City Council.
- NSW Public Works. (2005). *Hawkesbury Nepean Floodplain Management – Emergency Flood Evacuation Route Options for Richmond and Londonderry*. Prepared for the Roads and Traffic Authority
- NSW Government. (2005). *Floodplain Development Manual: the Management of Flood Liable Land*. ISBN: 0 7347 5476 0
- Rhelm/Catchment Simulation Solutions (2024) *Hawkesbury-Nepean River Flood Study.* Draft Final Report. Prepared for the NSW Reconstruction Authority
- Ryan, C (2013). *Using LiDAR Survey for Land Use Classification*. Paper presented at the 2013 Floodplain Management Authorities Conference, Tweed Heads.
- WMAwater (2019). *Hawkesbury-Nepean Valley Regional Flood Study*. Final Report. Prepared for Infrastructure NSW

11 GLOSSARY AND ABBREVIATIONS

Term	Shortened form	Definition	Context for use/additional information
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	AEP is generally the preferred terminology. ARI is the historical way of describing a flood event, for example, a 1% AEP flood has a 1% or 1 in 100 chance of being reached or exceeded in any given year
Australian height datum	AHD	A common national surface level datum often used as a referenced level for ground, flood and flood levels	0.0 m AHD corresponds approximately to mean sea level
Average recurrence interval	ARI	The long-term average number of years between the occurrence of a flood equal to or larger in size than the selected event	ARI is the historical way of describing a flood event. AEP is generally the preferred terminology, for example, a 100-year ARI flood that has 1 in 100 chance of being reached or exceeded in any given year. It is equivalent to a 1% AEP flood
Catchment		The area of land draining to a specific location	It includes the catchment of the primary waterway as well as any tributary streams and flowpaths
Catchment flooding		Flooding due to prolonged or intense rainfall (e.g. severe thunderstorms, monsoonal rains in the tropics, tropical cyclones)	Types of catchment flooding include riverine, local overland and groundwater flooding
Chance		The likelihood of something happening that will have adverse or beneficial consequences	In FRM this generally relates to the adverse consequences of floods with chance being related to AEP, for example, 1% chance or 1 in 100 chance per year is equivalent to 1% AEP
Coastal inundation		Inundation due to tidal or storm-driven coastal events, including storm surges in lower coastal waterways. This can be exacerbated by wind-wave generation from storm events	
Consent authority		The authority or agency with the legislative power to determine the outcome of development and building applications	This may be the relevant local council or Minister
Consequence		The outcomes of an event or situation affecting objectives, expressed qualitatively or quantitatively	Consequences can be adverse (e.g. death or injury to people, damage to property and disruption of the community) or beneficial
Continuing flood risk		Risk to existing and future development that may be reduced by EM measures	Flood risk to the existing development and future development may be reduced by EM measures depending on flood constraints, however, these

Term	Shortened form	Definition	Context for use/additional information
			measures cannot remove all risk and a residual risk will remain
Defined flood event	DFE	The flood event selected as a general standard for the management of flooding to development	Aims to reduce the frequency of flooding but does not remove all flood risk, for example, in selecting a 1% AEP flood as a DFE you are accepting that there is a 1 in 100 chance that a larger event will occur in any year. This risk is being built into the decision
Design flood		The flood selected as part of the FRM process that forms the basis for physical works to modify the impacts of flooding	The design flood may be considered the flood mitigation standard, for example, a levee may be designed to exclude a 2% AEP flood, which means that floods rarer than this may breech the structure and impact upon the protected area. In this case, the 2% AEP flood would not equate to the crest level of the levee, because this generally has a freeboard allowance, but it may be the level of the spillway to allow for controlled levee overtopping
Development		May be treated differently depending on the following categorisation: · **infill development**: the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under current land zoning · **new development**: development of a completely different nature to that associated with the former land-use (e.g. the urban subdivision of a previously rural area) · **redevelopment**: rebuilding in an area (e.g. as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale)	New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power Redevelopment generally does not require either rezoning or major extensions to urban services
Development control plan	DCP	See *Environmental Planning and Assessment Act 1979*	

Term	Shortened form	Definition	Context for use/additional information
Emergency management	EM	A comprehensive approach to dealing with risks to the community arising from hazards. It is a systematic method for identifying, analysing, evaluating and managing these risks	May include measures to reduce flood frequency or consequences through prevention and mitigation measures, and preparation, as well as response and recovery should a flood occur (see PPRR)
Ecologically sustainable development	ESD	As outlined in the *Local Government Act 1993*	Principles of ESD are outlined in the *Local Government Act 1993*
Existing flood risk		The risk an existing community is exposed to as a result of its location on the floodplain	Existing flood risk may be reduced by existing or proposed FRM measures leaving a residual flood risk to the existing community. Residual flood risk may be further reduced by addressing continuing risk
Flood		A natural phenomenon that occurs when water covers land that is normally dry. It may result from coastal inundation (excluding tsunamis) or catchment flooding, or a combination of both	Flooding results from relatively high stream flow that overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flowpaths associated with major drainage, and/or oceanic inundation resulting from super-elevated ocean levels
Flood (hydrologic and hydraulic) modelling		Hydrologic and hydraulic computer models to simulate catchment processes of rainfall, run-off, stream flow and distribution of flows across the floodplain or similar	They typically involve consideration of the local flood history, available collected data, and the development of models that are calibrated and validated, where possible, against historic flood events and extended to determine the full range of flood behaviour
Flood affected land		Equivalent to flood prone land	See the definition of flood prone land
Flood awareness		An appreciation of the likely effects of flooding, and a knowledge of the relevant flood warning, response and evacuation procedures facilitating prompt and effective community response to a flood threat	In communities with a low degree of flood awareness, flood warnings may be ignored or misunderstood, and residents confused about what they should do, when to evacuate, what to take with them and where to go
Flood constraints		Key constraints that flooding place on land	These include flood function, flood hazard, flood range, and flood emergency response classification. These can be used to inform FRM including consideration of options such as mitigation works, EM and land-use planning
Flood damage		The tangible (direct and indirect) and intangible costs (financial,	Tangible costs are quantified in monetary terms (e.g. damage to goods)

Term	Shortened form	Definition	Context for use/additional information
		opportunity costs, clean-up) of flooding	Intangible damages are difficult to quantify in monetary terms and include the increased levels of physical, emotional and psychological health problems suffered by flood affected people that are attributed to a flood
Flood education		Seeks to provide information to raise community awareness of flooding so as to enable individuals to understand how to manage themselves and their property in response to flood warnings	
Flood evacuation		The movement of people from a place of danger to a place of relative safety, and their eventual return	People are usually evacuated to areas outside of flood prone land with access to adequate community support Livestock may be relocated to areas outside of the influence of flooding
Flood fringe areas		That part of the flood extents for the event remaining after the flood function areas of floodway and flood storage areas have been defined.	
Flood function		The flood related functions of floodways, flood storage and flood fringe within the floodplain	Flood function is equivalent to hydraulic categorisation
Flood hazard		A flood that has the potential to cause harm or conditions with the potential to result in loss of life, injury and economic loss	The degree of hazard varies with the severity of flooding and is affected by flood behaviour (extent, depth, velocity, isolation, etc.)
Flood impact and risk assessment	FIRA	A study to assess flood behaviour, constraints and risk, understand offsite flood impacts on property and the community resulting from the development, and flood risk to the development and its users	These studies are generally undertaken for development and are to be prepared by a suitably qualified engineer experienced in hydrological and hydraulic analysis for FRM
Flood liable land		Equivalent to flood prone land	See the definition of flood prone land
Flood plan (local or state)	Local (LFP)	A sub-plan of an EM plan that deals specifically with flooding; they can exist at state, zone and local levels	The NSW Government develops flood plans as a legislative responsibility to determine how best to respond to floods. These community-based plans describe the risk to the community, outline agency roles and responsibilities, the agreed community emergency response strategy and how floods will be managed
Flood planning area	FPA	The area of land below the FPL	The FPA is generally developed based on the FPL for typical residential development. Different types of development may have different FPLs applied within the FPA. In addition development

Term	Shortened form	Definition	Context for use/additional information
			controls will vary across the FPA due to varying flood constraints
Flood planning level	FPL	The combination of the flood level from the DFE and freeboard selected for FRM purposes	Different FPLs may apply to different types of development Determining the FPL for typical residential development should generally start with a DFE of the 1% AEP flood plus an appropriate freeboard (typically 0.5 m). This assists in determining the FPA
Flood prone land		Land susceptible to flooding by the PMF event	Flood prone land is also known as the floodplain, flood liable land and flood affected land
Flood risk		Risk is based on the consideration of the consequences of the full range of flood behaviour on communities and their social settings, and the natural and built environment	See also risk. The degree of risk varies with circumstances across the full range of floods. It is affected by factors including flood behaviour and hazard, topography and EM difficulties
Flood risk management	FRM	The management of flood risk to communities	
Flood risk management manual: the policy and manual for the management of flood liable land	the manual	This manual	
Flood storage areas		Areas of the floodplain that are outside floodways which generally provide for temporary storage of floodwaters during the passage of a flood and where flood behaviour is sensitive to changes that impact on temporary storage of water during a flood	See also flood function, floodways and flood fringe areas
Flood study		A comprehensive technical investigation of flood behaviour undertaken in accordance with the principles in this manual and consistent with associated guidelines A flood study defines the nature of flood behaviour and hazard across the floodplain by providing information on the extent, level and velocity of floodwaters, and on the distribution of flood flows considering the full range of flood events up to and including extreme events, such as the PMF	A flood study is undertaken in accordance with the FRM process outlined in this manual to support the understanding and management of flood risk. It is different from a flood impact and risk assessment (FIRA)

Term	Shortened form	Definition	Context for use/additional information
Flood warnings		Warnings issued when there is more certainty that flooding is expected, are more targeted and are issued for specific catchments	Flood warnings include more specific predictions of the severity of expected flooding and may give quantitative figures such as expected river water heights at gauge stations
Floodplain		Equivalent to flood prone land	See the definition of flood prone land
Floodways		Areas of the floodplain which generally convey a significant discharge of water during floods and are sensitive to changes that impact flow conveyance. They often align with naturally defined channels or form elsewhere in the floodplain	See also flood function, floodways and flood fringe areas Floodways are sometimes known as flow conveyance areas
Flow		The rate of flow of water measured in volume per unit time, for example, cubic metres per second (m^3/s)	Flow is different from the speed or velocity of flow, which is a measure of how fast the water is moving
Freeboard		A factor of safety typically used in relation to the setting of minimum floor levels or levee crest levels	Freeboard aims to provide reasonable certainty that the risk exposure selected in deciding on a specific event for development controls or mitigation works is achieved. Freeboards for development controls and mitigation works will differ. In addition freeboards for development control may vary with the type of flooding and with the type of development
Frequency		The measure of likelihood expressed as the number of occurrences of a specified event in a given time	For example, the frequency of occurrence of a 20% AEP or 5-year ARI flood is once every 5 years on average
FRM measures		Measures that can reduce flood risk	FRM measures may include FRM, flood mitigation, EM and land-use planning measures
FRM options		The FRM measures that might be feasible for the management of a particular area of the floodplain	Preparation of an FRM plan requires a detailed evaluation of FRM options
FRM plan		A management plan developed in accordance with the principles in this manual and its supporting guidelines	Previously known as a floodplain risk management plan or floodplain management plan. It may describe how particular areas of flood prone land are to be used and managed to achieve defined objectives
FRM study		A management study developed in accordance with the principles in this manual and its supporting guidelines	Previously known as a floodplain risk management study or floodplain management study

Term	Shortened form	Definition	Context for use/additional information
Future flood risk		The risk future development and its users are exposed to as a result of its location on the floodplain	Future flood risk may be reduced by existing or proposed FRM measures and land-use planning controls that consider the flood constraints on the land. This leaves a residual flood risk to the new development and its users. This residual flood risk may be further reduced by addressing continuing flood risk
Gauge height		The height of a flood level at a particular water level gauge site related to a specified datum	The datum may or may not be the AHD
Hazard		A source of potential harm or conditions that may result in loss of life, injury and economic loss due to flooding	
Hydraulics		The study of water flow in waterways and flowpaths; in particular, the evaluation of flow parameters such as water level and velocity	
Hydrology		The study of the rainfall and run-off process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods	
Integrated planning and reporting framework	IP&R framework	The IP&R framework includes a suite of integrated plans that set out a vision and goals and strategic actions to achieve them. It involves a reporting structure to communicate progress to council and the community as well as a structured timeline for review to ensure the goals and actions are still relevant	Preparation of FRMS and plans and implementation and maintenance of works requires linkages to the IP&R framework
Likelihood		A qualitative description of probability and frequency	See also frequency and probability
Likelihood of occurrence		The likelihood that a specified event will occur	With respect to flooding, see also AEP and ARI
Local environmental plan	LEP	See *Environmental Planning and Assessment Act 1979*	
Local government area	LGA		The area serviced by the local government council
Local overland flooding	LOF	Inundation by local run-off on its way to a waterway, rather than overbank flow from a waterway	
Local strategic planning statement	LSPS		Local strategic planning statements assist councils to implement the priorities set out in their community strategic plan and actions in regional and district plans

Term	Shortened form	Definition	Context for use/additional information
Loss		Any negative consequence or adverse effect, financial or otherwise	
Merit-based approach		Weighs social, economic, ecological and cultural impacts of land-use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and wellbeing of the state's rivers and floodplains	The merit approach operates at 2 levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk, which are formulated into council plans, policy and environmental planning instruments At a site-specific level, it involves consideration of the merits of a development consistent with council LEPs, DCPs and local FRM policies, and consistent with FRM plans
NSW Floodplain Management Program	The program	The NSW Government's program of technical support and financial assistance to local councils to enable them to understand and manage their flood risk	The program, manual and FRM guides support the delivery of the policy through a partnership across governments
NSW Flood prone land policy	the policy	The *NSW Flood prone land policy* included in this document	
Prevention, preparedness, response and recovery	PPRR	Involves: · **prevention:** to eliminate or reduce the level of the risk or severity of emergencies · **preparedness:** enhances the capacity of agencies and communities to cope with the consequences of emergencies · **response:** to ensure the immediate consequences of emergencies to communities are minimised · **recovery:** measures that support individuals and communities affected by emergencies in the reconstruction of physical infrastructure and restoration of physical, emotional, environmental and economic wellbeing	In the flood context prevention involves FRM (including flood mitigation), EM and land-use planning measures
Probability		A statistical measure of the expected chance of a flood	For example, AEP
Probable maximum flood	PMF	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation (PMP), and where applicable, snow melt, coupled with the worst flood-producing catchment conditions	This is equivalent to the probable maximum precipitation flood in Australian Rainfall and Runoff (ARR) The PMF in ARR is used for estimating dam design floods

Term	Shortened form	Definition	Context for use/additional information
Probable maximum precipitation	PMP	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 Organization 1986)	PMP is the primary input to PMF estimation
Rainfall intensity		The rate at which rain falls, typically measured in millimetres per hour (mm/h)	Rainfall intensity varies throughout a storm in accordance with the temporal pattern of the storm
Residual flood risk		The risk to the existing and future community that remains with FRM, EM and land-use planning measures in place to address flood risk	FRM measures cannot remove all flood risk, but rather they reduce residual flood risk
Risk		'The effect of uncertainty on objectives' (ISO 2018)	See also flood risk. Note 4 of the definition in ISO31000:2018 also states that 'risk is usually expressed in terms of risk sources, potential events, their consequences and their likelihood'
Risk analysis		The systematic use of available information to determine how often specified (flood) events occur and the magnitude of their likely consequences	
Run-off		The amount of rainfall that ends up as streamflow, also known as rainfall excess	
State environmental planning policy	SEPP	See *Environmental Planning and Assessment Act 1979*	
Scenario		A scenario may relate to current, historical or assumed future floodplain, catchment and climate conditions	Flood behaviour varies over time with changes in key catchment and floodplain (such as the scale of development) and climatic conditions (including climate change), and due to the implementation of FRM measures. A range of scenarios are generally needed to understand and assess flood behaviour
Stage		Equivalent to water level; measured with reference to a specified datum	Measurement may relate to AHD, a local datum or a local water level gauge
Storm surge		The increases in coastal water levels above predicted astronomical tide level (i.e. tidal anomaly) resulting from a range of location-dependent factors	These factors may include the inverted barometer effect, wind and wave setup and astronomical tidal waves, together with any other factors that increase tidal water level

Term	Shortened form	Definition	Context for use/additional information
Technical working group	TWG		
Velocity		The speed of floodwaters, measured in metres per second (m/s)	
Vulnerability		The degree of susceptibility and resilience of a community, its social setting, and the built environment to flooding	Vulnerability is assessed in terms of ability of the community and environment to anticipate, cope and recover from flood events