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ATTACHMENTS FOR: AGENDA NO. 7/23
COUNCIL MEETING

Meeting Date: **Tuesday 25 July 2023**
Location: **Council Chambers, Level 1A, 1 Pope Street, Ryde and Online**
Time: **6.00pm**

ATTACHMENTS FOR COUNCIL MEETING

Item

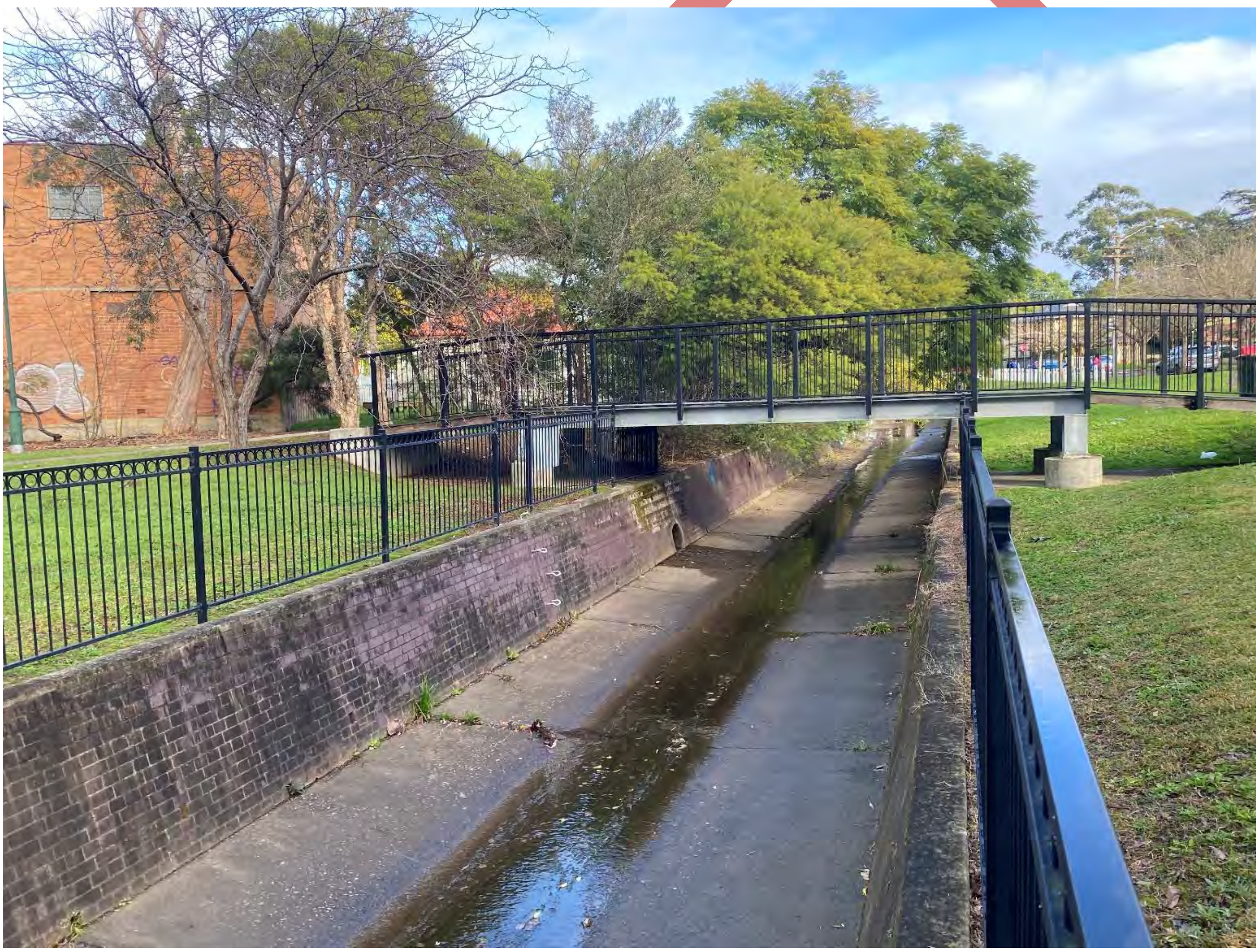
8 HARMONISING FLOOD STUDIES - PROJECT UPDATE

Attachment 1 Draft Flood Study - City of Ryde

CITY OF RYDE COUNCIL

FLOOD HARMONISATION STUDY – FLOOD STUDY UPDATE

DRAFT REPORT



JANUARY 2023

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FLOOD HARMONISATION STUDY – FLOOD STUDY UPDATE

DRAFT REPORT

JANUARY 2023

Project **FLOOD HARMONISATION STUDY – FLOOD STUDY UPDATE**	**Project Number** **120099**
Client City of Ryde Council	**Client's Representative** Manel Mariner
Authors Michael Ren Michael Reeves Rhys Hardwick Jones	**Prepared by** TO BE SIGNED FOR FINAL REPORT
Date 12 January 2023	**Verified by** TO BE SIGNED FOR FINAL REPORT

Revision	Description	Distribution	Date
1	Working Draft Report	City of Ryde Council	March 2022
2	Draft Report	City of Ryde Council	June 2022
3	Draft Report	City of Ryde Council	July 2022
4	Draft Report for Public Exhibition	City of Ryde Council	January 2023

EXECUTIVE SUMMARY

This flood study update provides information about the existing flood risk across the Ryde Local Government Area (LGA). The study includes a comprehensive update to the previous four Flood Studies (FS) and Floodplain Risk Management Study and Plans (FRMS&P) that cover the 14 catchments within the Ryde LGA.

The previous Flood Studies include:

- Eastwood and Terrys Creek
- Macquarie Park (includes Mars Creek, Shrimptons Creek, Industrial Creek, Porters Creek and Lane Cove River)
- Buffalo and Kittys Creek
- Parramatta River – Ryde Sub-Catchments (includes Archer Creek, Denistone, Charity Creek, Gladesville and Parramatta River)

Following a detailed review of the available data and information, the previous flood models that were developed for these studies were consolidated and updated. This included updating the DRAINS hydrologic models and TUFLOW hydraulic models. The flood models were calibrated to the available historic flood data, including the events of November 1984 and February 1990. The overall calibration outcome was considered to be reasonably good, with the models suitable for the estimation of design flood behaviour across the Ryde LGA.

Design flood behaviour was mapped across the LGA, including peak flood depths and levels, peak velocities, hydraulic hazard classification and flood function. Flood risk precincts and flood emergency response classifications were also defined for the LGA. A flood tagging process was also undertaken to identify lots subject to flood planning controls. This was undertaken for the 1% AEP event and the PMF event, in accordance with the relevant legislation.

Model parameter sensitivity assessments were also undertaken to understand how sensitive flood levels are to adopted model parameters. The impact of climate change (increase in rainfall intensity and sea level rise) was also considered as part of this assessment.

This flood harmonisation study provides the basis for progressing to a Floodplain Risk Management Study and Plan, which will use the models developed as part of this study to develop and assess potential flood risk mitigation measures.

FLOOD HARMONISATION STUDY – FLOOD STUDY UPDATE

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FOREWORD

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

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

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

1. Flood Study
- Determine the nature and extent of the flood problem.

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

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

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

ACKNOWLEDGEMENTS

This study was undertaken by WMAwater Pty Ltd, on behalf of the City of Ryde Council. Council has prepared this document through its Floodplain Management Program.

A number of organisations and individuals have contributed both time and valuable information to this study. The assistance of the following in providing data and/or guidance to the study is gratefully acknowledged:

- Floodplain Risk Management Committee
- Residents of the study area
- City of Ryde Council
- State Emergency Service
- Sydney Water Corporation
- NSW Department of Planning and Environment

1. INTRODUCTION

WMAwater was commissioned by the City of Ryde in May 2021 to undertake a flood harmonisation study across all 14 catchments within the Ryde Local Government Area (LGA). The study includes a comprehensive update to the four Flood Studies (FS) and Floodplain Risk Management Study and Plans (FRMS&P) for each catchment across the LGA. This report documents the updated Flood Study component of the study.

The City of Ryde LGA is located between the Parramatta and Lane Cove rivers and has 16 suburbs within its boundaries, which are Chatswood West (part), Denistone, Denistone East, Denistone West, East Ryde, Eastwood (part), Gladesville (part), Macquarie Park, Marsfield, Meadowbank, Melrose Park (part), North Ryde, Putney, Ryde, Tennyson Point and West Ryde.

Council has undertaken several studies on overland flow and localised flooding of the Eastwood and Terry's Creek Catchment, Macquarie Park Catchment, the Buffalo and Kittys Creek Catchments and the Parramatta River Ryde Subcatchments in the past. They were undertaken in different years and some of the studies are over 10 years old. Much of the City was developed by the 1980s, while over the last decade, rapid residential development along with a growing population has occurred especially in the vicinity of the Macquarie Shopping Centre, Macquarie Park, North Ryde and in Meadowbank towards Shepherds Bay. Several major drainage improvement projects have been completed to alleviate the historical flooding problems. The previous flood studies became inadequate to represent the current conditions. There is a need to review and update the existing studies.

In addition, the previous flood studies followed Australian Rainfall and Runoff 1987 guidelines (ARR87, Reference 1). These guidelines were updated in a major draft revision in 2016, which was finalised in 2019 (ARR19, Reference 2). With an additional 30 years of data and improvements in computing technology, ARR19 presents a significant update to design flood estimation methods. These updates include the following:

- Rainfall Intensity-Frequency-Duration (IFD) Data
- Rainfall Temporal Patterns
- Rainfall Losses
- Areal Reduction Factors

The update to the Flood Study includes the use of the revised ARR19 methods and data, more recent terrain and built form survey, and current best practice modelling techniques.

1.1. Scope and Objectives

This flood harmonisation study is aimed to provide an integrated flood study across the whole LGA. The updated Flood Study will provide a foundation for development of a robust floodplain risk management plan in the future stage of the program.

In conjunction with the NSW Floodplain Development Manual (Reference 3) and NSW Flood Prone Land Policy, the Flood Harmonisation study will enable Council to:

- Understand the current flood risk across the catchment
- Provide up to date flood data as current for all end users
- Enable future development planning
- Control cumulative impacts of future development
- Assess the effectiveness of potential flood mitigation measures

Design flood events modelled in this Flood Study update study include the 50%, 20%, 10%, 5%, 2%, 1% Annual Exceedance Probability (AEP) events and the Probable Maximum Flood (PMF) event across the study area. It involved the following broad tasks:

- Collection of data and information relevant to the study,
- Preparation of hydrologic and hydraulic models capable of defining the flood behaviour,
- Calibrate the hydrologic and hydraulic models using available data from historical flood events,
- Simulate design flood behaviour across the study area for a range of probabilities,
- Interpretation and presentation of design model results to define flood behaviour including peak flood levels, depths, velocities, peak flows, and flood extent,
- Determine hydraulic categories and provisional hazard categories,
- Provide information relating to the consequences of flooding, emergency response, and land use planning,
- Determine preliminary flood planning level and flood planning area, and
- Undertake sensitivity analyses under climate change scenarios and sea level rise conditions.

1.2. Report Outline

This report documents the data, methodology and outputs from the Flood Study update stage of the Flood Harmonisation Study. The structure of the report is as follows:

Section 1: introduces the study
Section 2: provides background information for the study
Section 3: outlines the available data used in the study
Section 4: details the development of the hydrologic model
Section 5: details the development of the hydraulic model
Section 6: describes the model calibration process and results
Section 7: outlines the design flood event modelling process
Section 8: documents the design flood event modelling results
Section 9: provides results of the model sensitivity assessment
Section 10: provides the references used in this study

There are also several appendices which support this report, and provide calibration results, design flood mapping and sensitivity assessment results.

2. BACKGROUND

2.1. Study Area

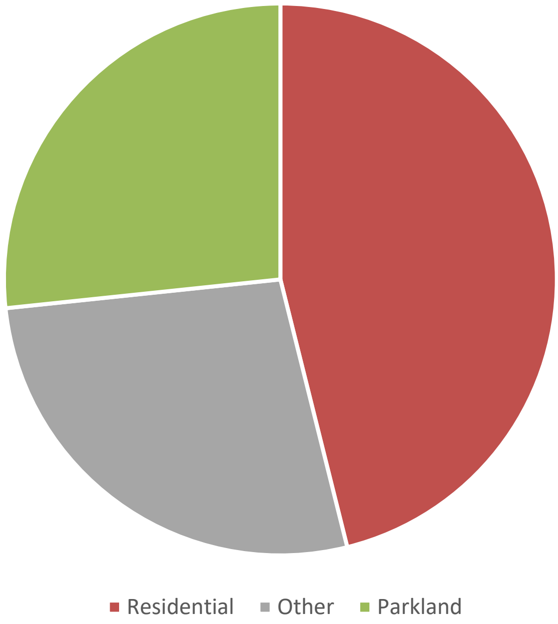
The City of Ryde LGA has 16 suburbs totalling 40.7 km^2 in area, with an estimated population of around 133,000 in 2019[1]. It is located in northern Sydney between 8 and 15 km north-west of the Sydney CBD. It is bounded by Lower Parramatta River to the south and by Lane Cove River and Terrys Creek to the north, neighbouring the Peninsula of Hunters Hill to the east and the City of Parramatta to the west. The city is serviced by the Northern railway line, along which Eastwood Station, Denistone Station, West Ryde Station and Meadowbank Station are within the study area, as well as a number of main roads including Victoria Road, Pittwater Road, Church Street, Devlin Street, Lane Cove Road, Blaxland Road, Epping Road, Marsden Road, and the M2 Motorway. The map for the study area with key features can be seen in Diagram 1, with further detail shown in Figure 1.

Diagram 1: Study Area

1 Estimate Resident Population Australian Bureau of Statistics (3218.0), 30th June 2020.

As shown in Diagram 2, the land area is largely occupied by residential dwellings with 47% of total land use. At the last census in 2016, detached dwellings make up less than half (46.2%) the dwelling types in Ryde. Medium to high density dwellings make up a significant proportion of the remainder with 24,419 (52.9%). Parklands and other lands including industrial, commercial, institutional areas and other special uses make up the remainder of the total area.

Diagram 2: City of Ryde Land Use (Reference 4)



The study area was divided into several subcatchments subject to individual flood study investigations in the past, as summarised in Table 1 and discussed below. These catchment areas are shown in Figure 2.

Table 1: Study Area Catchments

Catchment	Area (km²)	Flows to
Eastwood Catchment[1]	1.69	Lane Cove River
Terrys Creek[1]	3.26	
Mars Creek (including University Creek)	3.27	
Shrimptons Creek	5.55	
Industrial Creek	1.48	
Porters Creek	2.25	
Lane Cover River Catchment[2]	3.03	
Kittys Creek	1.93	
Buffalo Creek	5.5	
Archer Creek	2.86	Parramatta River
Denistone Catchment	2.15	
Charity Creek	2.47	
Parramatta River[3]	1.58	
Gladesville Catchment	3.66	
TOTAL	40.68	

1 The total Terrys Creek catchment (including Terrys Creek and Eastwood drainage areas) is approximately 10.12 km², however, the upstream portion is located within the City of Parramatta LGA (approximately 1.60 km²), and parts of the northern side of the catchment are located within the Hornsby Shire Council LGA (approximately 3.57 km²). and these areas outside the Ryde LGA have not been included in this table.

2 Area within the Ryde LGA that drain directly to the Lane Cove River

3 Area within the Ryde LGA that drain directly to the Parramatta River

2.1.1. Eastwood and Terrys Creek

A significant portion of the Terrys Creek catchment (with an area of 1.6 km²) is upstream of the Ryde LGA within the Parramatta City Council LGA, entering at Terry Road (Photo 1). Within Ryde, this creek runs through the Eastwood town centre via Glen Reserve (Photo 2 and Photo 3) and under the Northern Rail line (Photo 4).

Photo 1: Terrys Creek downstream of Terry Road

Photo 2: Terrys Creek junction upstream of Glen Reserve

Photo 3: Terrys Creek upstream of Eastwood Town Centre

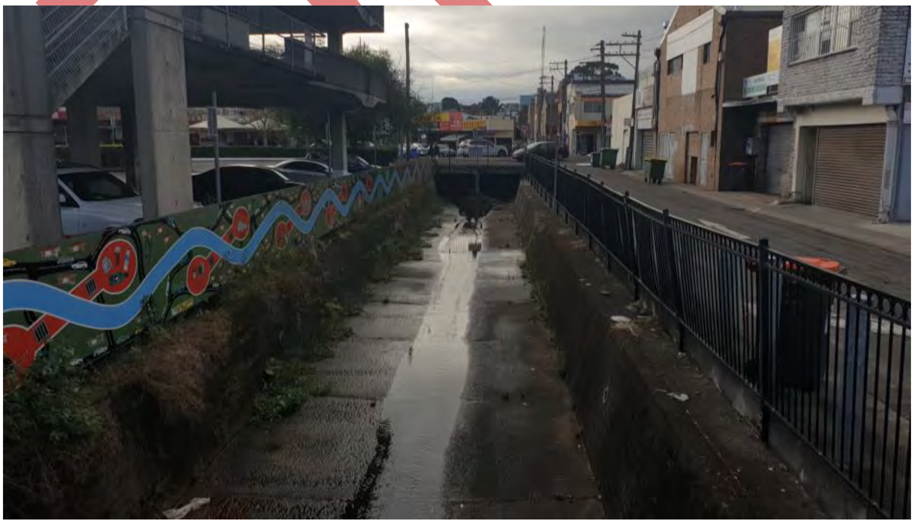


Photo 4: Terrys Creek crossing at railway line (upstream side)

There is a significant tributary catchment known as the Balaclava Road branch that comprises much of the area east of the railway line between Blaxland Road and Balaclava Road. The creek channel is concrete-lined between Terry Road and for a significant distance downstream of the railway line (Photo 5) until downstream of Blaxland Road near Somerville Park. Downstream of the railway line the creek generally forms the boundary between the Ryde and Hornsby Shire Council LGAs, and there is additional catchment area of 3.57 km^2 to the north-west within the Hornsby LGA.

Photo 5: Terrys Creek at 606-608 Blaxland Road

2.1.2. Mars Creek

The Mars Creek catchment lies entirely within the Ryde LGA, comprising three separate watercourses that flow north-east to the Lane Cove River. Two of the watercourses including Mars Creek are located primarily within Macquarie University, and the other flows through Marsfield Park and Waterloo Park.

2.1.3. Shrimptons Creek

Shrimptons Creek is one of the creeks in the Macquarie Park area that flows north-east to the Lane Cove River, with a much larger catchment area than the neighbouring creeks (such as Mars Creek, Porters Creek and Industrial Creek). The main creek line extends from Santa Rosa Park (Photo 6), passes through ELS Hall Park, then across Bridge Road (Photo 7), Kent Road, Epping Road and into the major underground conduits under Waterloo Road, with a significant overland flow path through the Macquarie Shopping Centre (Photo 8). The culverts discharge into an open channel downstream of Talavera Road, which crosses under the M2 Motorway and eventually joins the Lane Cover River. There are several overland flow tributary branches through residential areas which flow to Shrimptons Creek.

Photo 6: Shrimptons Creek at Santa Rosa Park near Flinders Road

Photo 7: Shrimptons Creek crossing at Bridge Road

Photo 8: Overland flow path through Macquarie Shopping Centre at Waterloo Road

2.1.4. Industrial Creek, Porters Creek and Lane Cove River Catchment

The Industrial and Porters Creek catchments comprise several overland flow paths through the industrial park/technology centre in Macquarie Park. The creeks have generally been built over and replaced by pipes, in some cases without formal overland flow paths for major flows that exceed the pipe capacity. The open channels only extend from downstream of the M2 Motorway before flowing into the Lane Cove River. In the Lane Cove River Catchment, Pages Creek, running from west to east, carries flows from the industrial area at the southern side of Delhi Road and a small portion of the residential area upstream of Pittwater Road (such as the catchment to Morshead Street, Photo 9).

Photo 9: Overland flow path from sag point in Morshead Street near Chisholm Street

2.1.5. Kittys Creek

Kittys Creek extends from Macquarie Hospital (North Ryde) to downstream of Pittwater Road into the Lane Cover River. The creek line upstream of Harford Street is generally developed with a combination of pipe drainage and overland flow paths through private property. Downstream of Harford Street the creek is mostly open channel (Photo 10).

Photo 10: Kittys Creek downsteam of Harford St

Photo 11: Martins Creek at Carramar Avenue

A tributary called Martins Creek carries flows from the northern part of the catchment, running from North Ryde Oval to the west of Pittwater Road near Carramar Avenue (Photo 11), and joining Kittys Creek before it crosses Pittwater Road. Kittys Creek is heavily vegetated and is generally in a relatively natural state compared to other creeks in the study area. Flood issues in this catchment are generally associated with overland flow paths in the upper tributary areas where the natural creek line has been developed and replaced with a major/minor stormwater network.

2.1.6. Buffalo Creek

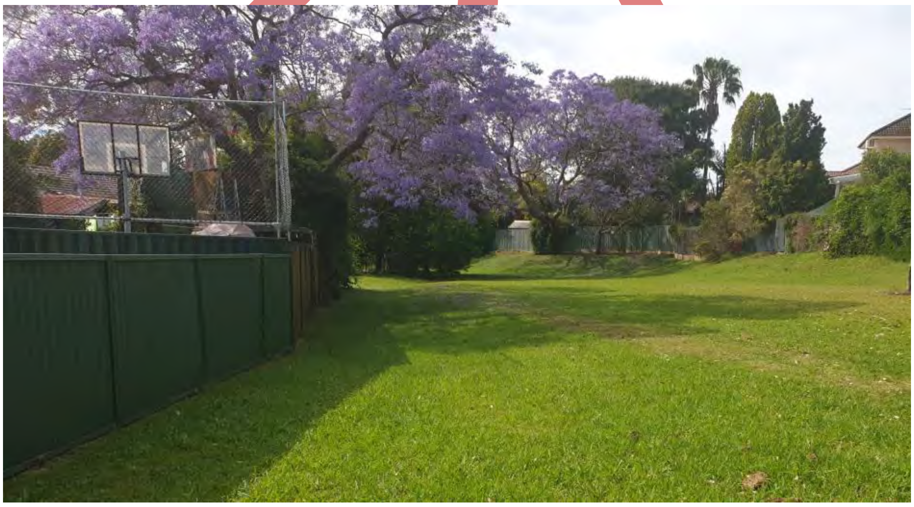
The Buffalo Creek catchment has a relatively large area covering large parts of the suburbs of Ryde, East/North Ryde and Gladesville. Similar to Kittys Creek, it is generally in a relatively natural state apart from the upper catchment areas where development has occurred across the creek line. These developed upper catchment areas are more extensive than in the Kittys Creek catchment. There are extensive sections of overland flow path and floodway through private property upstream of Princes Street on the main branch, and upstream of Gardiner Avenue and Minga Street on the tributary branches. Downstream of Princes Street there is a concrete lined portion of channel (Photo 12). Buffalo Creek generally flows in an easterly direction into the Lane Cove River through some heavily vegetated areas such as Burrows Park, Field of Mars Reserve and Buffalo Creek Reserve. Strangers Creek is a major tributary carrying a portion of flows from the northern part of the catchment, joining Buffalo Creek before it crosses Pittwater Road. Top Ryde Shopping Centre sits near the upstream border of the catchment. Flood issues in this catchment are generally associated with overland flow paths in the upper tributary areas where the natural creek line has been developed and replaced with a major/minor stormwater network.

Photo 12: Buffalo Creek downstream of Princes Street

2.1.7. Archer Creek

The Archer Creek catchment originates south of Eastwood, runs through Denistone West, West Ryde and Melrose Park. An additional contributing area of approximately 0.5 km^2 is located west of Wharf Road within the Parramatta City Council. This tributary has an overland flow path and trunk drainage line through Jennifer Park (Photo 13) from What Road to Cobham Avenue, which joins Archer Creek within the Ryde Parramatta Golf Club.

Photo 13: Jennifer Park overland flow path



The main drainage line passes through Brush Farm Park, Pt Lambert Park, Maze Park and Ryde Parramatta Golf Club and flows into a concrete open channel in Meadowbank Park, before discharging into the Parramatta River. Significant sections of the upper creek branches run through developed areas, with a combination of overland flow paths and piped drainage. In some locations these flow paths are within public reserves (such as West Denistone Park) before entering private property at the downstream end (Photo 14).

Photo 14: Overland flow path in West Denistone Park near Morvan Street

2.1.8. Denistone Catchment

The Denistone catchment comprises most of Denistone and the middle part of West Ryde. Runoff is directed into the concrete open channel that starts downstream of Constitution Road through Meadowbank Park, eventually flowing into the Parramatta River. There are two main watercourses downstream of the railway line, passing through Darvall Park (Photo 15) and Miriam Park upstream of the West Ryde town centre.

Photo 15: Darvall Park overland flow path at Denistone Sports Club

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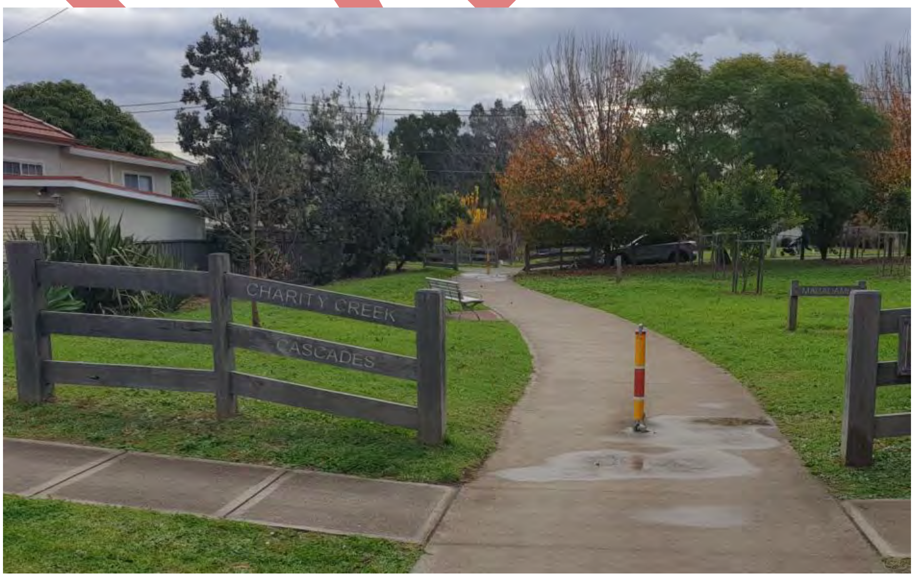
The West Ryde town centre area is subject to flooding due to the depressed topography relative to the hydraulic control of Victoria Road. To alleviate the downstream flooding problems identified in the 1980s, the West Ryde Tunnel was built in 1999 starting from Miriam Road. When runoff exceeds the pipe drainage system capacity, the only exit for overland flow from the sag point within Graf Avenue is via Station Street or the passageways between the shops at 997 Victoria Road (Photo 16).

Photo 16: Arcade passageway providing overland flow egress from Graf Avenue sag point

2.1.9. Charity Creek

The Charity Creek catchment includes parts of Denistone, Ryde, West Ryde and a portion of Meadowbank. There is a significant overland flow path from Shepherd Street to Victoria Road through the Charity Creek Cascades Playground and associated reserves (Photo 17). This flow path continues in a disjointed fashion through a densely developed industrial area downstream of Victoria Road between Falconer Street and Hermitage Road.

Photo 17: Charity Creek Cascades overland flow path



The railway line embankment forms a significant hydraulic control and obstructs overland flows. Downstream of the railway line, the creek is mostly contained within a large concrete open channel. This runs from Bank Street (Photo 18), passing under Constitution Road and through east side of Meadowbank Park to the Parramatta River. The TAFE NSW Ryde campus is located in the upper part of the catchment and the Meadowbank campus is in the mid-catchment, immediately upstream of the railway line.

Photo 18: Charity Creek downstream of Bank Street

2.1.10. Residual Parramatta River Catchment (Shepherds Bay)

There are several smaller unnamed urbanised tributaries that discharge directly into the Parramatta River. These areas include the western part of Putney, part of Meadowbank east of the railway, and parts of Ryde to the south-west of Victoria Road. Major re-development has occurred in this area in the last decade, and the development has included construction of significant drainage infrastructure, including a flow path from Ann Thorn Park and Constitution Service Road (Photo 19 and Photo 20) across Nancarrow Avenue (Photo 21), and into the Parramatta River between Hedgeland Close and Rothesay Avenue.

Photo 19: Upgraded cross drainage inlet at Constitution Road near Ann Thorn Park.

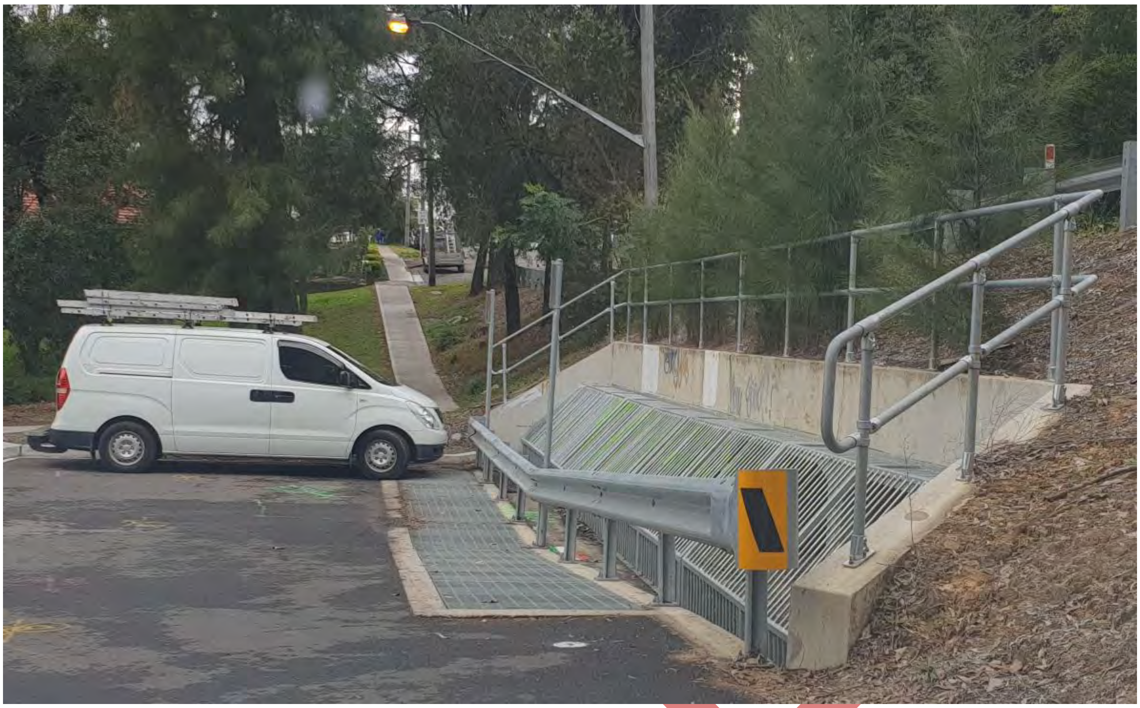


Photo 20: Overland flow easement downstream of Constitution Road

Photo 21: Floodway at Nancarrow Avenue

2.1.11. Gladesville Catchment

The Gladesville catchment includes parts of the suburbs of Ryde (south), Putney (east), Gladesville (south) and Tennyson Point. Major overland flow paths develop from downstream of the detention basin at Lardelli Park (Photo 22) and downstream of the Ryde Aquatic Leisure Centre. These drainage lines flow through Parry Park (Photo 23), Tyagarah Park, Bremner Park and adjoining residential areas before converging at Morrison Road and flowing via a concrete open channel through Morrison Bay Park. Another main flow path runs via Peel Park into the Glades Creek through Bill Mitchell Park. The area to the east between Victoria Road and Pittwater Road drains eastwards into the Hunters Hill Council area.

Photo 22: Outlet structure at Lardelli Park detention basin

Photo 23: Overland flow path within Parry Park (looking downstream)

2.2. Historical Flooding

The study area catchment has been subject to flooding in the past, with notable events occurring in November 1984, August 1986, December 1989, February 1990, March 1990, February 2010 and April 2012 according to the council's database and previous flood studies. More recent events including November 2018 and February 2020 have caused localised flood problems and damage. The historical event causing the most significant widespread damage was the November 1984 storm, which caused severe damages to a large number of houses, shops, vehicles, and infrastructure. A selection of photos from previous floods are shown below, with their location shown in Figure 3.

In Eastwood, there is a long history of flooding at the town centre. Flooding is known to have occurred with varying degrees of severity in 1967, 1971, 1979, 1984 to 1990, 2003, 2010, 2012 and most recently in 2018. In the 1984 event approximately 70 residential and commercial properties experienced above floor flooding. The embankment of the Northern Railway line is several metres higher than the ground levels upstream, and the limited capacity of the culverts under the railway line results in overbank flows being obstructed and trapped within the town centre area. There are additional areas in the catchment where there are known flood problems, such as Rowe Street where the overland flow path is blocked by a contiguous line of buildings, so runoff exceeding the stormwater system capacity is trapped in this area.

According to the Macquarie Park Flood Study (Bewsher 2010, Reference 6), around 56% of the responses from the community consultation in 2008 indicated some experience of flooding at their property, with 12 reporting above floor inundation in the worst flood (November 1984). Likewise in Parramatta River catchments, the 1984 event caused widespread flooding and considerable damages, especially around the West Ryde town centre where depths were reportedly up to 2 m in Graf Avenue, ruining goods in the shops (SKM, 2013).

There have been fewer reports of severe flooding in the past in the Buffalo and Kittys Creek Catchments. Based on the questionnaire in 2012 as part of flood study by GHD (Reference 7), about 8% of respondents reported being flood affected with notable events in February 1990 and May 1998.

Photo 24: Hillview Rd Eastwood (Nov 1984)

Photo 25: Railway Parade Eastwood (Nov 1984)

Photo 26: Shrimptons Creek at Waterloo Rd (Nov 1984)

Photo 27: Macquarie Shopping Centre Carpark (Nov 1984)

Photo 28: West Ryde Shopping Centre (Dec 1989)

Photo 29: Progress Ave Eastwood (April 2012)

3. AVAILABLE DATA

3.1. Previous Studies

Several Flood Studies (FS) and Floodplain Risk Management Studies and Plans (FRMS&P) have previously been completed within the City of Ryde catchments:

- Eastwood and Terrys Creek FS (Bewsher, 2008) and FRMS&P (Bewsher, 2009), Reference 5,
- Macquarie Park FS (Bewsher, 2010) and FRMS&P (Bewsher, 2011), Reference 6,
- Buffalo and Kittys Creek FS and FRMS&P (GHD, 2014), Reference 7,
- Parramatta River – Ryde Sub-Catchments FS (SKM, 2013) and FRMS&P (SKM, 2015), Reference 8.

The previous studies applied Australian Rainfall and Runoff 1987 (ARR87) methods for design flood modelling. Each study included the development of DRAINS hydrologic models and TUFLOW hydraulic models.

In addition to the catchment-wide studies, the Eastwood Town Centre Flood Study and Stormwater Upgrades Design investigation was completed in 2019 by Royal Haskoning DHV (Reference 9). The aim was to investigate mitigation measures in the Eastwood town centre, and the study included an update of the models using more detailed and recent information about the catchment. Australian Rainfall and Runoff 2016 (ARR16) methods were adopted for design modelling in the updated DRAINS and TUFLOW models.

Another local flood study at Macquarie Park was completed in January 2012 by Henry & Hymas (Reference 10), provided by Council. The study was to reduce the risk of flooding for some of the residential properties in Rogal Place, Fontenoy Road and Tuckwell Place, as well as to improve the overland flow paths. The study was based on the catchment-wide models, but included several refinements to the drainage network that were reviewed and incorporated in this study.

3.1.1. Eastwood and Terrys Creek FS and FRMS&P

The flood study assessed catchment flood behaviour for the historical event in 1984 November for model calibration and for a range of design flood events including 5-year, 10-year, 50-year, 100-year Average Recurrence Interval (ARI) and Probable Maximum Flood (PMF). Two DRAINS hydrologic models and a TUFLOW hydraulic model were developed for the study. In TUFLOW, a 3 m grid resolution was used, and Terrys Creek was modelled as 1D channel embedded in the 2D domain. The model was calibrated against historic information sourced from a Council database and previous Terrys Creek studies from 1991 and 2005. Critical storm durations of 120 minutes and 15 minutes were adopted for events up to 100-year ARI and PMF, respectively. The subsequent FRMS&P assessed and recommended floodplain management measures to reduce the flood risks. Options recommended included Mobbs Lane detention basin, several drainage augmentations, updated planning and development controls, improved emergency management operations and improved public awareness.

3.1.2. Macquarie Park FS and FRMS&P

The study covered the Mars Creek (including University Creek), Shrimptons Creek, Industrial Creek, Porters Creek and Lane Cove River Catchments. Three DRAINS models and three TUFLOW models were developed and run to simulate the November 1984 event and February 1990 event for calibration purposes. The TUFLOW model utilised a 3 m grid resolution with the main creeks and river being modelled using 1D elements. It was calibrated against flood marks from a 1990 study by Willing & Partners as well as flood records from a Council database and community questionnaire. The 20-year ARI, 100-year ARI and PMF design flood events were modelled to assess design flood behaviour across the study area. Critical durations for design events were typically two hours, with the nine hour also being relevant downstream of Fullers Bridge along Lane Cove River.

In the subsequent FRMS&P, a detailed evaluation of potential management measures was undertaken, and recommended measures were assessed, selected, and categorised into high, medium-high, medium, medium-low or low priorities.

3.1.3. Buffalo and Kittys Creek Catchments FS and FRMS&P

These Flood Studies were the first completed for the Buffalo and Kitts Creek catchments. A DRAINS model was developed and verified against peak flow calculations using the Rational Method for Urban Catchments. Two TUFLOW models were developed with a 2 m grid resolution, in which creeks were modelled in 2D domain. No historical events were simulated. The locations of flood observations in February 1990 from community consultation were reviewed in the model. A 1D HEC-RAS model was used to validate the TUFLOW model. The design events included the 20%, 5%, 2% and 1% AEP and PMF events. One to two hour design storm events were found to be critical.

A FRMS&P was then undertaken to identify and assess flood management options for Buffalo and Kittys Creek catchments.

3.1.4. Parramatta River FS and FRMS&P

The flood study defined the existing flood behaviour in the five catchments that drains to the Parramatta River, being Archer Creek, Denistone, Charity Creek, River and Gladesville catchments. Five separate DRAINS models were developed for the catchments. Design flow hydrographs were used as inflows into TUFLOW models, with a 3 m grid resolution except for the River Catchment (which was 2 m resolution). Concrete channels in the catchments were modelled as 1D elements. A joint-model verification was conducted by comparing the modelling results with the observed flood depths in November 1984 and the February 1990 events, which were sourced from the questionnaire responses and Council's database. Design flood behaviour was defined for the 20%, 10%, 5%, 2% and 1% AEP and PMF events. Generally, the 2-hour storm was the critical duration up to 1% AEP event.

As part of the FRMSP, options were recommended for future floodplain risk management with different priorities.

3.2. Light Detection And Ranging (LiDAR) Data

Aerial survey of the catchment was the primary source of topographic data used for this study. The aerial survey was acquired using a method called Light Detection and Ranging (LiDAR), a form of light-based radar. The most recent LiDAR data was obtained from NSW Spatial Services for the Ryde catchments by WMAwater. A combination of datasets acquired by the NSW Department of Land and Property Information in 2019 and 2020 covers the whole LGA. Figure 4 illustrates the extent of 2019 and 2020 LiDAR survey respectively.

The 1 m gridded Digital Elevation Models (DEMs) from the LiDAR dataset were used for modelling in this study. The stated accuracy of this information is 0.3 m (95% Confidence Interval, equivalent to a 0.15 m error for one standard deviation) vertical and 0.8m (95% Confidence Interval) horizontal. These specifications are equivalent to the LiDAR used previously in Council's studies, although based on previous experience it is likely that the DEMs are higher quality than earlier LiDAR datasets, due to advances in the filtering, tinning and gridding algorithms used to process the LiDAR.

WMAwater undertook a comparison of the 2019/2020 LiDAR data with:

- Survey Control Information Management System (SCIMS) survey benchmarks from NSW Spatial Services,
- point survey in Morrison Bay and Meadowbank parks (supplied by Council), and
- localised detail survey at Melba Drive and West Parade (supplied by Council).

71 of the SCIMS benchmarks are within the 2019 LiDAR extent and 282 are within the 2020 LiDAR extent. The histograms of the differences between the 2013 / 2019 / 2020 LiDAR and the SCIMS marks are plotted in Diagram 3 / Diagram 4 / Diagram 5 respectively.

Diagram 3: Histogram of Errors (2013 LiDAR vs SCIMS)

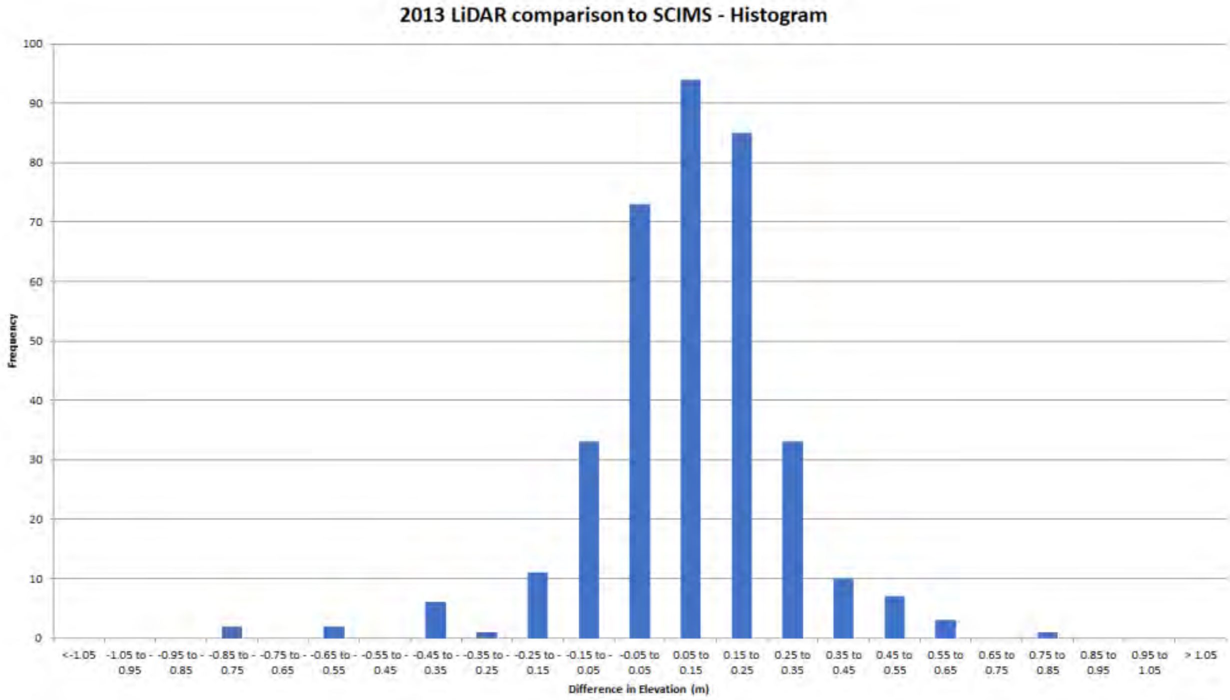


Diagram 4: Histogram of Errors (2019 LiDAR vs SCIMS)

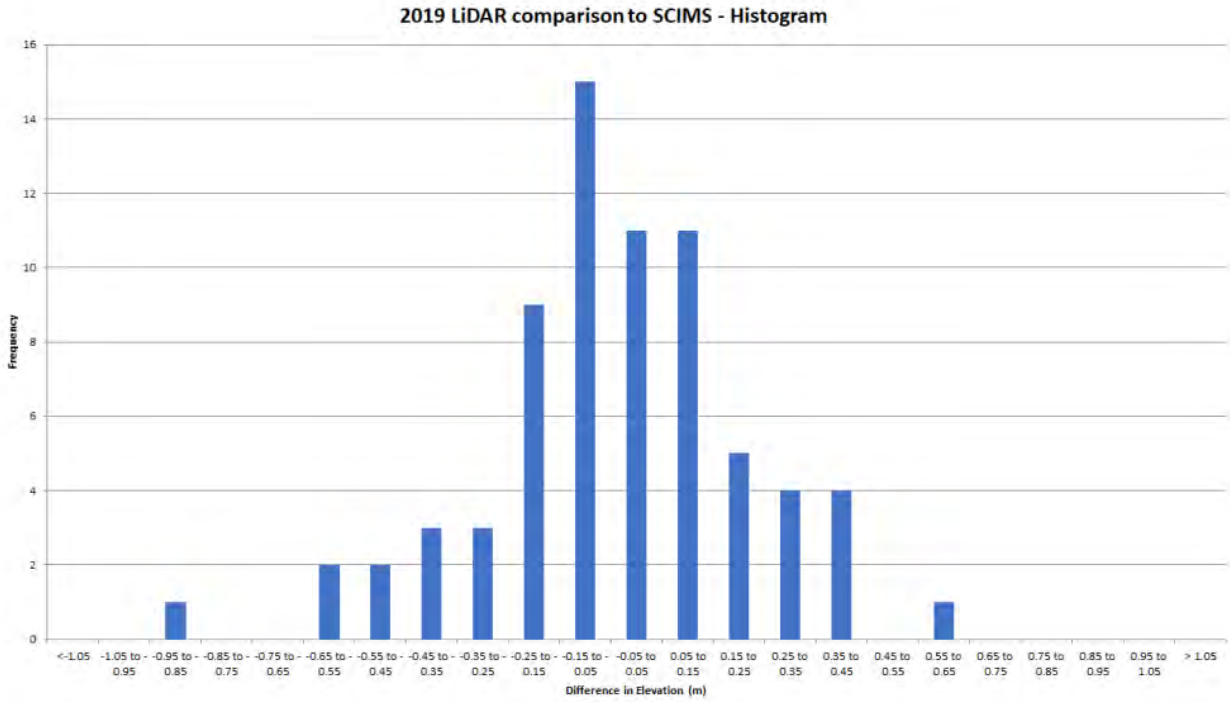
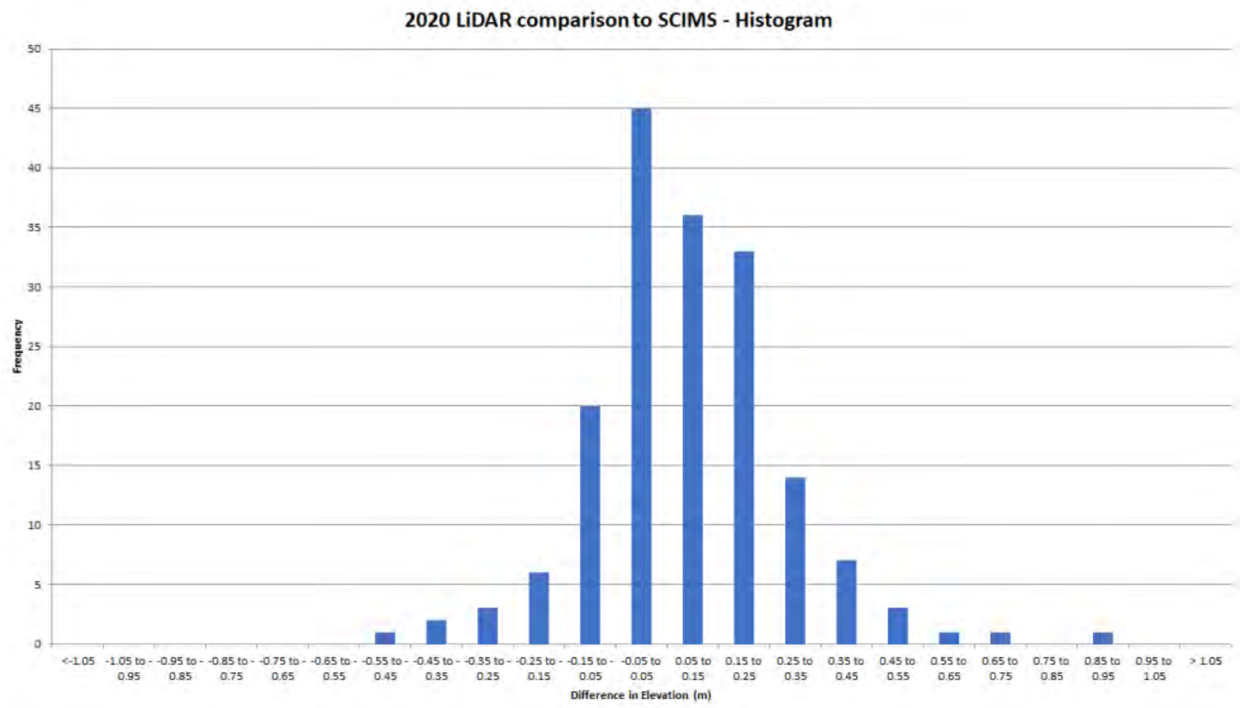


Diagram 5: Histogram of Errors (2020 LiDAR vs SCIMS)



A statistical summary of the differences is provided in Table 2. Each of the datasets shows a similar error profile, without significant bias. The standard deviation is slightly outside the stated accuracy of 0.15 m. Further investigation of the spatial variation of the errors indicates that the results were skewed by a significant error bias for benchmarks located within the railway corridor. This might be caused either by the quality of the LIDAR within the railway corridor, and the effect of the railway ballast on the results, or with the quality and placement of the survey benchmarks in the corridor. For example the benchmarks could be on top of posts sticking out of the ground to make them easier to find within the corridor, and this would not be captured in the LIDAR, resulting in “errors” of the magnitude found in the analysis. When these outliers were removed from the analysis, the standard deviation of the errors was within the stated accuracy of the datasets.

Table 2: Summary of LiDAR ground level errors (metres) versus SCIMS survey benchmarks (LiDAR minus SCIMS)

Error	2013 LiDAR Error	2019 LiDAR Error	2020 LiDAR Error
Average	0.10	-0.04	0.09
Median	0.10	-0.04	0.09
Standard Deviation	0.19	0.26	0.19

Further comparisons were made with detail survey available at West Parade for the 2019 LiDAR data and at Melba Drive for the 2020 LiDAR data. The statistics for the differences are provided in Table 3 for these comparisons, which indicate an excellent match.

Table 3: Comparison of LiDAR DEMs vs Local Road Detail Survey (LiDAR minus survey)

Error (m)	West Parade	Melba Drive
Average	-0.001	0.007
Standard Deviation	0.026	0.054

Further spot levels from detail survey within the parklands at Morrison Bay and Meadowbank were compared to the LiDAR from 2019 (for Meadowbank) and 2020 (for Morrison Bay). Diagram 6 and Diagram 7 show the scatter of these points around the zero-error line with ±0.3 m (95% confidence) bounds.

Diagram 6: Comparison of 2020 LiDAR Data vs Survey Points (Morrison Bay)

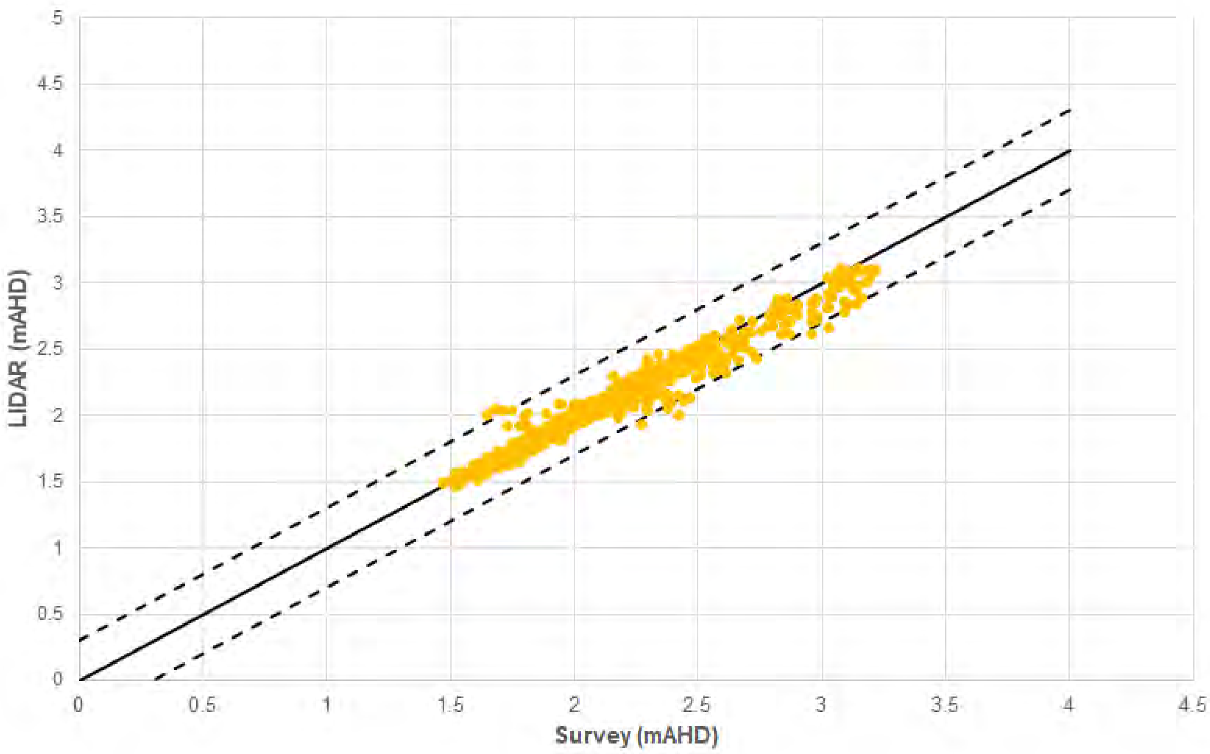
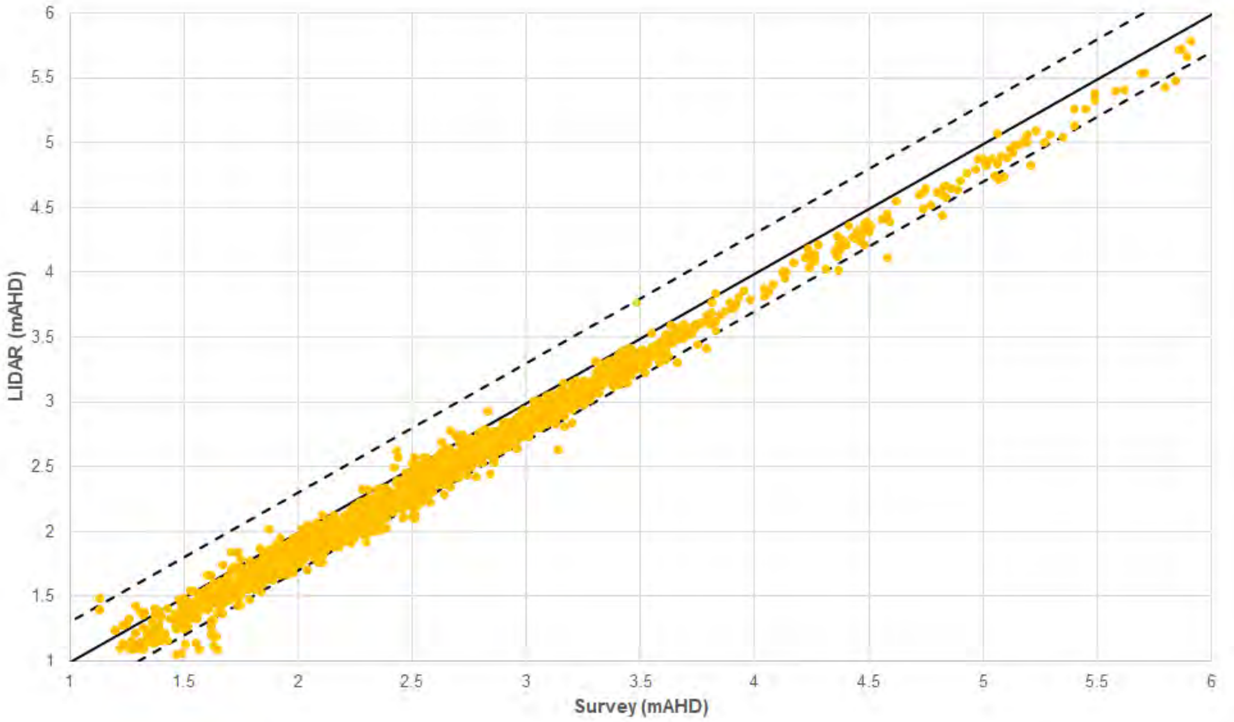


Diagram 7: Comparison of 2019 LiDAR Data vs Survey Points (Meadowbank)



These results indicate errors within expected limits for Morrison Bay, but a bias exists for the Meadowbank comparison of about 0.16 m (the LiDAR produces levels generally lower than the detail survey within this park). The match for Morrison Bay is very good both in terms of error magnitude and bias. For Meadowbank, the standard deviation of the errors is very good and well inside the expected limits, but there is a bias in the errors of about 0.16 m. This could be due to several factors, including the selection of the local benchmark for reducing the detail survey, as well as ground conditions within the park at the time of either of the survey. For example soil moisture and expansion of the ground could potentially account for this difference depending on whether the surveys were obtained during particularly wet or dry periods.

Based on the analysis above, WMAwater concluded that the 2019/2020 LiDAR DEMs were within the stated accuracy bounds and suitable for use as the base topography in the updated hydraulic models for this study.

3.3. Aerial Imagery

Electronic versions of recent and historical aerial imagery were provided by Council. The most recent image, dated on 6 December 2020 has 6.5 x 6.5 cm pixel resolution. The spatial location of the 2020 imagery appears to match reasonably well with 2019/2020 LiDAR data from spot checks. Historical images of 2016, 2014, 2010 with 10 cm pixel resolution and a 2001 image with 7.5 cm resolution were also provided. In addition, photos from 1981 and 1982 for most of the study area were provided in PDF format with a lower resolution than the other images.

These images are useful for understanding the state of the catchment at the time of this study update, as well as the changes over time (and particularly the changes that have occurred since the dates of the various historical flood events used for model calibration).

3.4. Major Development

A comparison of the LiDAR between the most recent merged grid and 2013 LiDAR grids was undertaken to identify areas of major developments in the recent years. The most recent aerial image supplied by Council was also used to identify the presence of the buildings that have been recently built to verify the change of topography. As shown in Figure 5, significant development has occurred in the vicinity of the Macquarie Shopping Centre, Macquarie Park near Wicks Road, in Meadowbank towards Shepherds Bay, as well as in Ryde to the southwest of Lardelli Park. Sites with major development were investigated during the site inspections, and data about the relevant development applications (DA's) was obtained from Council where possible.

3.5. Buildings

The hydraulic modelling method used for this study incorporates information about building footprints (see Section 5.3.1). Datasets released by Microsoft Bing Maps in GeoJSON format were obtained and used as the base information for the spatial extent of buildings within the catchment (Reference 11). These datasets provide country-wide building footprints in Australia, generated using Bing Maps algorithms based on satellite imagery mainly from 2018. The evaluation metrics suggested that its quality was similar to hand digitised buildings in OpenStreetMap for the vast majority. Potential errors might be produced for very small buildings and connected buildings in dense urban areas, as well as for roof areas with open areas underneath (such as carports). The base data was reviewed and updated where appropriate in the process of the TUFLOW model schematisation.

3.6. Stormwater Infrastructure

3.6.1. Pit and Pipe Network

Asset databases of stormwater infrastructure were provided by Council in GIS and spreadsheet format and provided to WMAwater for the current study. The data was up to date as of 13 August 2021. Each of the GIS and spreadsheet formats had two separate files containing point data (inlet pits, headwalls, GPTs, manholes, junctions etc.) and line data (pipes, channels and culverts) respectively. The spreadsheet and GIS information primarily contained the same features with different information which could be matched via unique identifiers for each feature. There were some discrepancies with less than 1% of the features in one of the datasets not containing equivalent features in the other dataset.

Table 4 Summary of Information Gaps in Spreadsheet Stormwater Database

Data in Spreadsheet	Number	Percentage of Total
Pits in total	12,069	
Pits with lintel sizes	6,876	57.0%
Pits with grate sizes	5,788	48.0%
Pits with inverts	8,781	72.8%
Conduits in total	11,340	100.0%
Circular conduits with known sizes	10,550	93.0%
Box conduits with known sizes	755	6.7%
Conduits with known sizes	11,305	99.7%
Conduits with known inverts on both ends	6,805	60.0%

Table 5: Summary of Information Gaps in GIS Stormwater Database

Data in GIS Database	Number	Percentage of Total
Pits in total	11,975	
Pits with lintel sizes	6,861	57.3%
Pits with grate sizes	5,764	48.1%
Pits with inverts	0	0.0%
Conduits in total	11,241	100.0%
Circular conduits with known sizes	10,466	93.1%
Box conduits with known sizes	740	6.6%
Conduits with known sizes	11,206	99.7%
Conduits with known inverts on both ends	0	0.0%

Table 4 and Table 5 summarise the information gaps for each of the datasets. The hydraulic modelling requires information about invert levels, pipe sizes, pit inlet sizes and other geometry for every feature, so it was necessary to develop a process for estimating these values where data was missing. These processes are described in Section 5.3.2.

3.6.2. Culverts and Bridges

The primary source of culvert information was Council's stormwater datasets as presented in the previous section. Other available sources of information for culvert and bridges were as follows:

- Council structure assets data in EXCEL, dated 12 April 2019, provided details of bridges and culverts except invert levels.
- Structure inspection reports in PDF and DOC completed by Pitt & Sherry in June 2019 provided details of bridges and culverts except invert levels. Inspection photos were also available.
- PDF design drawings of footbridges supplied by Council.
- The Terrys Creek Capacity Assessment report in 2002 (Reference 12) supplied by Sydney Water Corporation (SWC) provided details of culverts along the Terrys Creek channel.
- Data extracted from previous flood studies provided details of surveyed cross sections along creeks and channels.
- Work-As-Executed (WAE) survey and design drawings for the recent major

developments in Macquarie Park and Meadowbank provided some information of recently installed cross drainage structures.

The coverage and quality of data for the culverts and bridges within the study area was adequate for the required modelling purposes. The main hydraulic structures are shown in Figure 6.

3.7. Gauge Data

There are no streamflow gauges in the study area. The rainfall gauge locations are shown on Figure 7 and discussed in the following sections.

3.7.1. Daily Rainfall

Daily rainfall is recorded as 24-hour rainfall totals to 9:00 am. There are 23 daily rainfall gauges either currently or historically operational in the vicinity of the study area, for which data is available from the Bureau of Meteorology (BoM). These are shown in Figure 7. Diagram 8 plots the operational periods of these gauges. 9 gauges are currently operating.

For the events of interest for model calibration, spatial patterns were generated based on rainfall totals from Table 6, and using the Natural Neighbour interpolation method to develop a spatial grid. The interpolated depths are mapped on Figure 8, Figure 11, Figure 14 and Figure 17 for the 1984, 1990, 2010 and 2018 events, respectively.

Diagram 8: BoM Daily Rainfall Station Periods of Operation

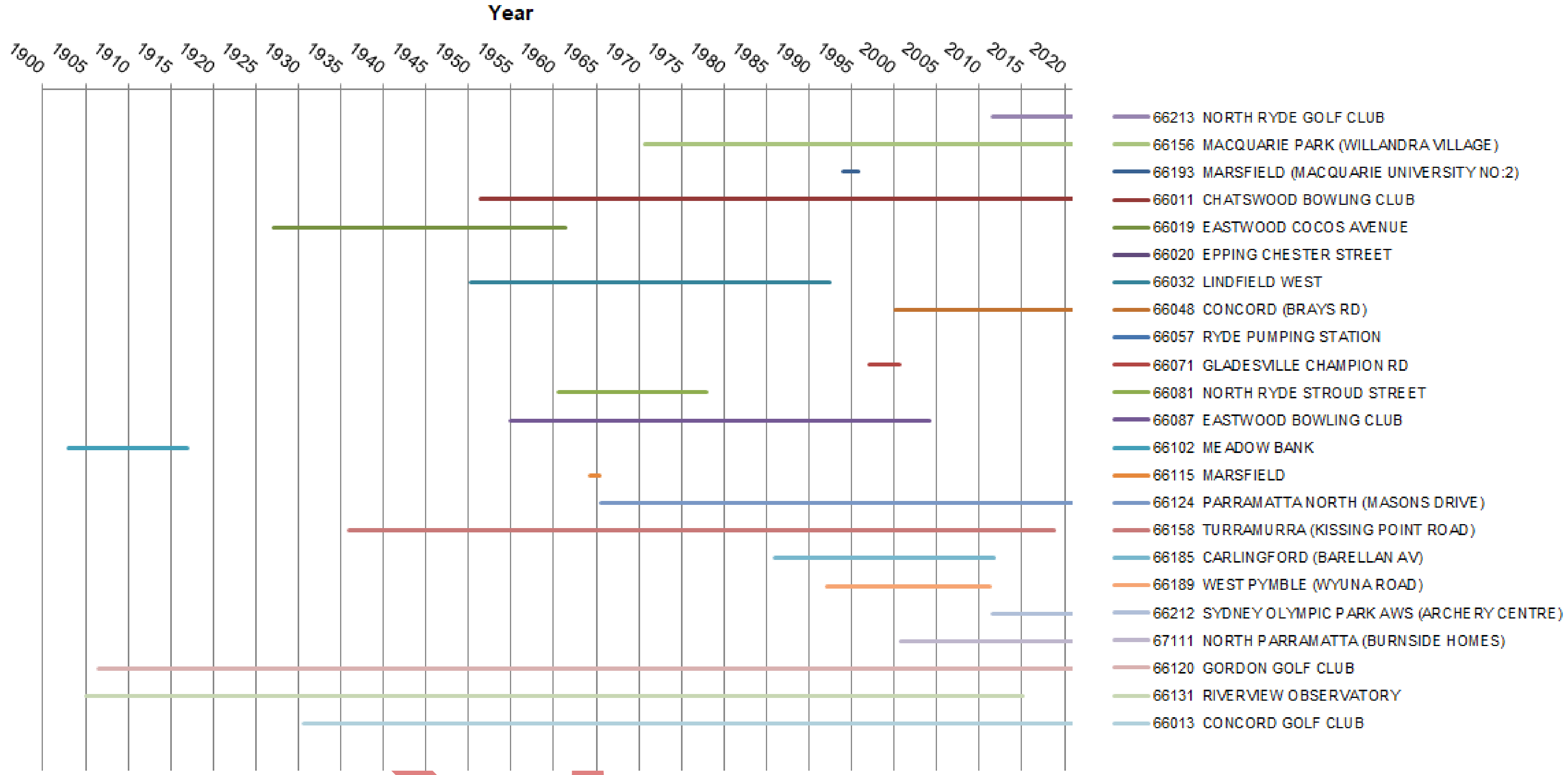


Table 6: Daily Rainfall Data for the 1984, 1990 and 2010 Events

ID	Gauge Name	Rainfall (mm)			
		1984 Nov 7th to 8th	1990 Feb 7th to 8th	2010 Feb 13th to 15th	2018 Nov 28th to 29th
66019	Eastwood Cocos Avenue	-	-	-	-
66020	Epping Chester Street	176	-	-	-
66032	Lindfield West	178.8	145.2	-	-
66057	Ryde Pumping Stn	-	-	-	-
66071	Gladesville Champion Rd	-	-	-	-
66081	North Ryde Stroud Street	-	-	-	-
66087	Eastwood Bowling Club	206.2	-	-	-
66102	Meadow Bank	-	-	-	-
66115	Marsfield	-	-	-	-
66120	Gordon Golf Club	205.8	94.8	74.5	129
66131	Riverview Observatory	-	-	91.2	-
66185	Carlingford (Barellan St)	-	148.2	134.4	-
66189	West Pymble (Wyuna Rd)	-	-	-	-
66013	Concord Golf Club	133	162	72	100.8
66213	North Ryde Golf Club	-	-	-	94
66212	Sydney Olympic Park AWS (Archery Centre)	-	-	-	95.8
66011	Chatswood Bowling Club	-	-	83	138
66124	Parramatta North (Masons Drive)	130	140.5	87.2	56
67111	North Parramatta (Burnside Homes)	-	-	82.5	58.5
66156	Marsfield (Willandra Village)	223.4	151.6	118.6	121.2
66193	Marsfield (Macquarie University No:2)	-	-	-	-
66048	Concord (Brays Rd)	-	-	85.8	98
66158	Turramurra (Kissing Point Road)	299.4	98.4	85.8	-

3.7.2. Sub-daily Rainfall

Pluviometers measure rainfall in small increments and provide a sub-daily breakdown of the timing of rainfall. Pluviometers are generally less numerous than daily rainfall gauges, and there is limited pluviometer rainfall data available in the study area, as shown in Figure 7. No BoM pluviograph gauges were found. Pluviograph gauges managed by Sydney Water in the study area and in its vicinity were obtained, 4 of which are located within the catchments. Two of them, West Epping Bowling Club 566040 and North Ryde Reservoir 566029, were terminated in July 2012 and January 2001 respectively. There are some additional gauges outside the study area within approximately 5 km. During the storm events in 1984, 1990 and 2010, the gauges with available data were summarised in Table 7. Data at these gauges were processed into 5-minute intervals to generate temporal patterns for calibration purposes. According to the Macquarie Park FS (Reference 6), it was noted that in the 'worst' 1984 event, rainfall patterns were recorded at a Marsfield Station by Macquarie University, which was used in the current study as another source for model calibration. Further details are presented in Section 6.1.2.

Table 7: Pluviograph Data Availability for the 1984, 1990 and 2010 Events

Flood Event	Stations with sub-daily rainfall data
8/11/1984	West Ryde Pumping Station (566037) West Epping Bowling Club (566040) Chatswood Bowling Club (566017)
7/02/1990	West Ryde Pumping Station (566037) West Epping Bowling Club (566040) Chatswood Bowling Club (566017) Pymble Bowling Club (566073)
12/02/2010	West Ryde Pumping Station (566037) West Epping Bowling Club (566040) Chatswood Bowling Club (566017) Pymble Bowling Club (566073) Gladesville Bowling Club (566087)
28/11/2018	Ryde Pumping Station 566037 Chatswood Bowling Club 566017 Pymble Bowling Club 566073 Gladesville Bowling Club 566087 North Ryde Golf Club 566008 North Epping Bowling Club (Composite) 566083

3.8. Flood Information

Historical flood information was extracted from previous flood studies and from Council's database supplied for this study. Details on data quality and data analysis of the flood information are presented in the section of model calibration (Section 6.1.1).

3.9. Site Visit

Site visits were conducted by WMAwater team members on 24 February 2021 and 22 June 2021. WMAwater personnel visited the majority of areas identified as flood problem areas based on the modelling from previous studies. A good understanding of ground conditions, hydraulic structures and floodplain characteristics was obtained. Photographs, observations and measurements taken during the site visit were used to inform model schematisation and adjustments during model setup and calibration where appropriate.

The site visit identified several important hydraulic structures and features that were not readily apparent from other available data, such as the subsurface floodway constructed beneath 30-32 Herbert Street, West Ryde (Photo 30).

Photo 30: Subsurface floodway at 30-32 Herbert Street, West Ryde

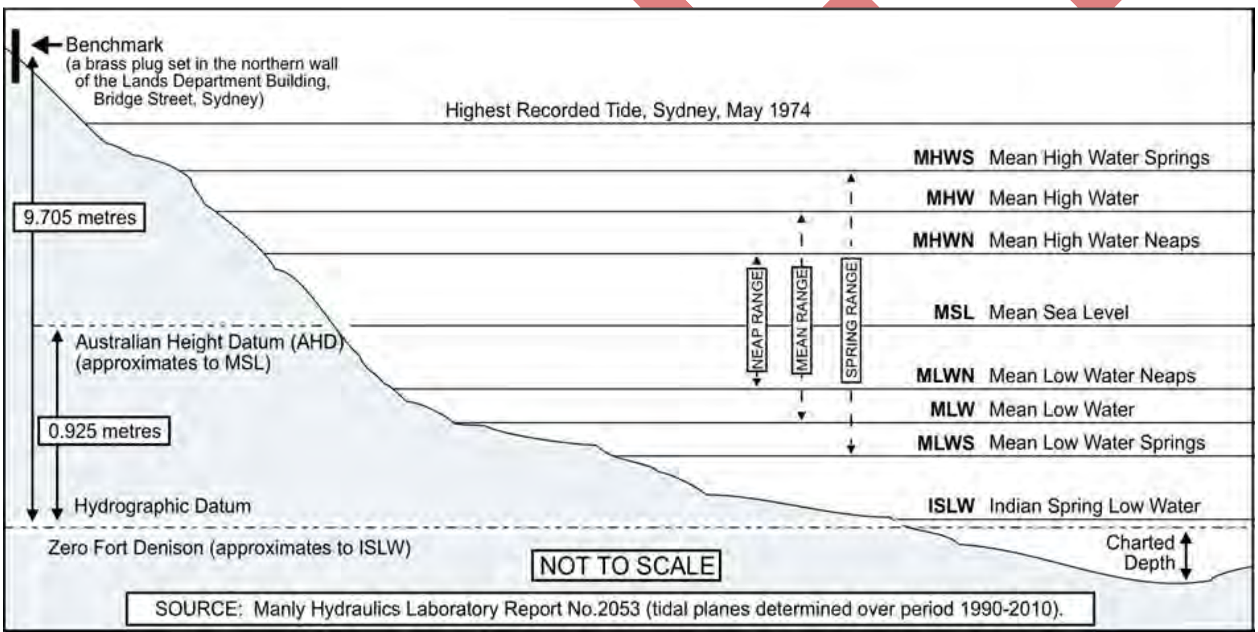
3.10. NSW Tidal Planes Analysis

Manly Hydraulics Laboratory prepared the *NSW Tidal Planes Analysis: 1990-2010 Harmonic Analysis* report on behalf of OEH (Reference 13). It was released in October 2012 and was based on data from 188 tidal monitoring stations from 1st July 1990 to the 30th June 2010. Data from the relevant stations are shown in Table 8 with a tidal plane diagram shown as Diagram 9.

Table 8: Tidal Planes Analysis Results (MHL, 2012)

Tidal Planes	**Annual Average Amplitude (mAHD)**		
	Ocean Tide Gauge Port Jackson (213470)	**Ocean Tide Gauge Port Hacking (213473)**	**Cooks River at Tempe Bridge (213415)**
High High Water Solstices Springs (HHWSS)	1.00	1.04	1.06
Mean High Water Springs (MHWS)	0.65	0.68	0.70
Mean High Water (MHW)	0.52	0.56	0.57
Mean High Water Neaps (MHWN)	0.40	0.44	0.45
Mean Sea Level (MSL)	0.02	0.07	0.06
Mean Low Water Neaps (MLWN)	-0.36	-0.31	-0.33
Mean Low Water (MLW)	-0.48	-0.43	-0.46
Mean Low Water Springs (MLWS)	-0.61	-0.55	-0.58
Indian Spring Low Water (ISLW)	-0.86	-0.81	-0.84

Diagram 9: Tidal Planes Diagram



The Parramatta River is tidal in the vicinity of the Ryde LGA, meaning that water levels in the river are controlled by the ocean levels in Sydney Harbour. The Lane Cove River is also tidal up to the Lane Cove Weir, near the Delhi Road crossing (Fullers Bridge). The tidal planes analysis can be used to determine tidal inundation extents for the Parramatta River and Lane Cove River.

4. HYDROLOGIC MODEL SCHEMATISATION

4.1. Introduction

A hydrologic model is a tool for estimating the amount of runoff that flows from a catchment for a given amount of rainfall, and the timing of this runoff flow. Stream gauges (which measure water level in a stream, and from which flow rates can be inferred) are a way of directly measuring this information, but they are expensive to setup and maintain. They also require a long record (several decades) to be of most use for flood estimation. Most of the smaller creeks and urbanised areas in NSW do not have streamflow gauges, and there are no stream gauges within the study area that can be used for Flood Frequency Analysis. In such cases, using a computer-based hydrologic model to convert rainfall to runoff is the best practice method for design flood estimation, since rainfall data is more widely available in these areas. This type of hydrologic model is referred to as a runoff-routing model.

A range of runoff-routing hydrologic models is available as described in ARR19 (Reference 2). These models allow the rainfall to vary in both space and time over the catchment and will calculate the runoff generated by each sub-catchment. The generated flow hydrographs then serve as inputs at the boundaries of the hydraulic model, which provides details about flood levels and velocities.

For the studied catchments, DRAINS software had been selected for hydrologic modelling in the previous flood studies. DRAINS was first released in 1998 by Watercom Pty Ltd, for design and analysis of urban stormwater drainage systems and catchments. DRAINS can carry out hydrological modelling using ARR19 procedures with unsteady hydraulic modelling of systems of pipes, open channels and surface overflow routes. The software has been widely used in Australia. It is also a convenient tool for Council to undertake local drainage assessment without the need to run other hydraulic models. Different versions of DRAINS were used at the time of previous studies. Version 2020.061 (64bit) was used in this study. The existing DRAINS models were reviewed, updated and run to provide subcatchment inflows into the TUFLOW models.

4.2. Model Extent and Subcatchment Delineation

A total of 11 separate DRAINS models have been developed in the previous flood studies in the study area, as summarised in Table 9. The Eastwood and Terrys Creek models incorporated the portion of the catchment within the City of Parramatta Council and the Hornsby Shire Council, which were represented as RAFTS subcatchments in DRAINS. The models for the Mars Creek and Porters/Industrial Creeks in Macquarie Park include the upstream catchment areas of the Lane Cove River and therefore the total area represented is significantly larger than just the catchment areas of the creeks within the study area. The subcatchments were delineated mainly based on the topographic information, aerial images and stormwater networks. Details of the processes can be found in the previous studies (Reference 5 to Reference 9). No revisions to the subcatchment delineation were made for this study.

Table 9: DRAINS Model Total Catchment Areas

ID	Catchment	Area (km^2)
1	Eastwood Catchment	3.98
2	Terrys Creek	6.44
3	Mars Creek / University Creek (includes areas of the Lane Cove River catchment upstream of the Ryde LGA)	37.52
4	Shrimptons Creek	6.12
5	Porters Creek, Industrial Creek, Lane Cover River Catchment (includes additional tributary catchment areas from the other side of the Lane Cove River)	22.82
6	Buffalo Creek and Kittys Creek	5.98
7	Archers Creek	3.25
8	Denistone Catchment	2.13
9	Charity Creek	2.42
10	Residual Parramatta River Subcatchments	1.30
11	Gladesville Catchment	3.33
TOTAL		**95.29**

4.3. Model Configuration

The hydrologic model methods and parameters were typically retained from the previous modelling, including:

- The Horton (ILSAX) method for rainfall losses.
- The existing stormwater pit and pipe data as well as overflow paths were retained for all models. 20% blockage for on grade pits and 50% for sag pits was applied.
- Depression losses were assumed to be 1 mm for paved areas and 5 mm for grassed areas.
- Catchment soil conditions were consistent across all existing models and retained for this study. The soil type 3 was adopted.
- Antecedent Moisture Condition (AMC) of 3, representing 'rather wet', was applied, except for extreme events, where a value of 4 for saturated condition was applied.
- Subcatchment characteristics such as area, slope and impervious fraction from the existing models were retained in the current study.
- For design modelling in accordance with ARR19, the existing models were updated to be able to run in the new DRAINS "Lite" software version for this study. The main updates and fixes included:
    - o Applying new procedures for overflow routes,
    - o Extending pipes and channels to the minimum lengths required in DRAINS,
    - o Assuming 375 mm diameter for dummy pipes with a previously assigned diameter of 10 mm,
    - o Updating pipe inverts that were higher than the pit surface levels.

5. HYDRAULIC MODEL SCHEMATISATION

5.1. Introduction

Hydraulic modelling is the simulation of how water moves across the terrain and interacts with built structures. A hydraulic model is frequently used to estimate the flood levels, depths, velocities, and extents across a floodplain. Dynamic hydraulic models like TUFLOW can provide information about how the flooding changes over time. The hydraulic model can simulate floodwater both within the creek banks, and when it breaks out and flows overland, including flows through structures (such as bridges and culverts), over roads and around buildings.

Two-dimensional (2D) hydraulic modelling is currently the best practice standard for urban flood modelling in Australia. It requires high resolution information about the topography, which is available for this study from the LiDAR aerial survey. Various 2D software packages are available. The TUFLOW package was adopted as it meets requirements for best practice and is currently the most widely used model of this type in Australia.

The TUFLOW modelling package includes a finite difference or finite volume numerical model for the solution of the depth averaged shallow water equations in two dimensions. The TUFLOW software has been widely used for a range of similar floodplain projects both internationally and within Australia and is capable of dynamically simulating complex overland flow regimes.

The TUFLOW model version used in this study was 2020-10-AA-w64 (using the finite volume HPC solver), and further details regarding TUFLOW software can be found in the User Manual (Reference 14).

In TUFLOW the ground topography is represented as a uniform grid with a ground elevation and Mannings ‘n’ roughness value assigned to each grid cell. The size of grid is determined as a balance between the model result definition required and the computer processing time needed to run the simulations. The greater the definition (i.e., the smaller the grid size) the greater the processing time need to run the simulation.

5.1. Model Extent and Resolution

The extent of the existing models was reviewed and updated for the new consolidated models.

The previous four flood studies involved the development of nine separate TUFLOW models covering the various City of Ryde catchments. These models were reviewed, refined, and consolidated into four models as summarised in Table 10. The size of the total study area is approximately 40 km^2, which is relatively large for urban flood modelling. It is therefore still appropriate to keep the models for different catchments separate to some degree, generally aiming for model areas below around 15 km^2. This means the models will have more efficient utility in the future as they will not take as long to run when investigating the impacts of potential development or infrastructure works in a specific location.

Table 10: Consolidation of TUFLOW Models

Existing models	Area (km^2)	Previous Model Resolution (m)	Consolidated Model ID
Eastwood and Terrys Creek	4.95	3	TC Model
Macquarie Park – Mars Creek	3.27	3	MQ Model
Macquarie Park – Shrimptons Creek	5.55	3	
Macquarie Park – Porters / Industrial / Lane Cove	6.76	3	
Kittys Creek	1.93	2	BK Model
Buffalo Creek	5.5	2	
Parramatta River – Archers / Denistone / Charity	7.48	3	PR Model
Parramatta River – Minor River Subcatchments	1.58	2	
Parramatta River – Gladesville	3.66	3	

Each of the consolidated models uses a 2 m grid resolution. The embedded one-dimensional 1D representation of the concrete-lined channels has been retained from the Eastwood and Terrys Creek model and the Parramatta River models (see details in 5.3.5). The boundaries of the consolidated models and extent of 1D channel elements is shown on Figure 20.

The approximate ratio of run time and model time on WMAwater modelling computers is given in Table 11.

Table 11: Computational Run Time vs Model Time

Consolidated Models	Ratio (Run Time / Model Time)
TC model	~ 0.25
MQ model	~ 0.85
BK model	~ 0.20
PR model	~ 0.25

5.2. Base Terrain

As described in Section 3.2, the merge of 2019 and 2020 LiDAR grids covering the whole LGA was the main source of topography data. The consolidated models use the LiDAR DEMs to form the underlying terrain. Modifications were made to ensure topographic features were represented appropriately. For example, poor triangulation of ground level strikes was identified at the Top Ryde Shopping Centre, and this was addressed by assuming a consistent ground level of 60 mAHD throughout the site. These adjustments are made using development data supplied by Council or observations from site inspections.

5.3. Hydraulic Structures

Modelling of structures is one of the most time-consuming aspects of the model schematisation process. The structures schematised in the model include buildings, the pit/pipe stormwater network, open channels, cross drainage structures such as culverts and bridges, kerb and gutter systems, railway station platforms, walls and other obstructions, and detention basins. The methods of schematisation for various structures are detailed in the following sections.

5.3.1. Buildings

Buildings are modelled as “blocked out” cells that are deactivated in the 2D domain, essentially assuming that all flow is excluded from buildings. While this is unfortunately not the case for all existing buildings, it is an objective of the floodplain management process and of Council’s development controls that new development will be at high enough levels to prevent flow within buildings. It is reasonable to assume that steps have and will be taken to protect existing buildings from above-floor inundation. This approach is also in line with guidance from Reference 15, which found that the flow paths through built up areas were more accurately resolved by using the “block out” method, than by alternative mechanisms where flow through the buildings is assumed.

The raw building footprint data was reviewed and updated based on the December 2020 aerial image and information from Development Applications (DA’s) supplied by Council. There has been significant redevelopment in the vicinity of the Macquarie Shopping Centre, Macquarie Park near Wicks Road, in Meadowbank towards Shepherds Bay, as well as in Ryde to the southwest of Lardelli Park. The schematisation of buildings in these areas was corrected using the more recent aerial imagery from December 2020. The building schematisation across all models is shown in Figure 22.

Overland flow paths through buildings were identified in the study area during site inspections including:

- Flows through the ground level carpark driveway at Macquarie Shopping Centre
- Flows through two arcades at West Ryde Town Centre
- Flows through a concrete channel under the building complex at 22-26 Herbert Street, West Ryde

To represent these flow paths, building footprints and terrain were modified, and where appropriate the flow paths are modelled as 1D elements rather than in the 2D domain.

5.3.2. Pit and Pipe Network

The pit/pipe stormwater network was completely re-schematised using a standard approach across all models, using Council’s database as primary source of information. WMAwater developed a comprehensive procedure to review, process, and test the information in an iterative manner, so that missing or erroneous data could be corrected and the information could be imported into TUFLOW. This work accounted for a large proportion of the effort spent during the model development phase (particularly the identification of data gaps and review to determine appropriate assumptions for these data gaps). The following steps were undertaken to check the data, including relevant assumptions applied when data gaps were identified:

1. Council GIS stormwater layers were transferred into TUFLOW format. The extracted information included:
    - For pipes:
        - Upstream and downstream inverts of pipes based on connected pit inverts from the Council spreadsheet database.
        - Pipe lengths

        - Number of pipes
        - Diameter of pipes and width and height of box conduits
        - A pipe identification string assigned used the upstream connected pit ID.
    - For pits:
        - Invert levels were extracted directly.
        - Identification of which pits were inlets and which were junctions/manholes. Inlet pits use the "R" type method in TUFLOW.
        - Average lintel length and grate dimensions were obtained based on the available pit data. Where information was unavailable a typical 1.8 m effective inlet width was assumed for the kerb inlets and a 0.9 m x 0.45 m size was assumed for the grates.
        - Pit ID was renamed with a Prefix "Pt" and limited to 10 characters as required by TUFLOW.
        - Pit invert levels

2. The extracted pits and pipes were clipped within each model extent. Irrelevant network elements draining into other catchments were removed.
3. Open concrete channels in the pipes and pits layer were removed as they were already established in the channel layers.
4. By comparison to the network in the existing TUFLOW models, missing pipes and pits in the network were identified and added in to ensure continuity. In some cases where the outlet was not defined, the likely location was determined based on the LiDAR data and the aerial image. For those pipes without dimensions, the upstream pipe size was assumed if the pipe was located upstream of a junction pit before discharging into a larger pipe, otherwise the downstream pipe size was assumed.
5. Pipe directions were checked and fixed so that they were digitised from upstream to downstream.
6. Locations of any upstream and downstream headwalls were identified and invert levels at these inlets and outlets were obtained from the LiDAR data where they were not provided in the raw data. Pipes that discharge into the 1D concrete-lined channel were "snapped" to coincide with the appropriate connection locations. The inverts were updated based on the channel inverts where they were not provided in the raw data.
7. Geographical information was obtained including X/Y coordinates and LiDAR values at the start and end nodes of the pipes, and GIS length of the pipes. These lengths were compared with the database length to identify possible errors or inconsistencies in the dataset. Lengths from dataset were compared with GIS lengths to identify any incorrect data. The existing data was retained if the absolute relative difference between two sources was within 20%, otherwise the GIS length was applied after checking that the connectivity and alignment of the GIS line appeared correct.
8. The inverts were reviewed to identify locations where upstream inverts were lower than the downstream, or the pipes were above ground except for those at inlets and outlets with headwalls. The value of the DEM ground level minus pipe diameter (height for box conduits) and minus a minimum cover of 150 mm was assumed for those incorrect inverts at junctions and at the ends of network. Remaining inverts with missing data were linearly interpolated between upstream and downstream inverts available from the database. The resulting grade of the pipes was checked to identify further potential errors.
9. Remaining pipes with adverse grade were identified for review. Corrections based on estimates from consideration of the total system were required to resolve the issue where the database appeared to be erroneous.

10. For all pipes, energy loss coefficients were adopted based on Table 12 below. These losses are subject to automatic adjustment within TUFLOW depending on entry and exit velocities.
11. All pipes were assigned a Manning's 'n' of 0.015.

Table 12: Energy Loss Coefficients for Stormwater Network Pipes

Loss Coefficient	Circular pipes	Box culverts
Inlet height contraction coefficient (Inlet controlled flow)	0	0.6
Inlet width contraction coefficient (Inlet controlled flow)	1	0.9
Entry loss coefficient (Pressurised flow)	0.5	0.5
The exit loss coefficient (Pressurised flow)	1	1

A total of 11,448 pits and 11,496 pipes are included in the TUFLOW models and the respective number of pipes and pits in each consolidated model is summarised in Table 13. Road cross-drainage culverts were included in the same layer in addition to the underground stormwater network. The network across all models is shown in Figure 23.

Table 13: Summary of Pipes and Pits for Each Model

Model	Number of Pits	Number of Pipes
Terrys Creek and Eastwood Model	1,462	1,478
Macquarie Park Model	3,534	3,541
Buffalo / Kittys Creeks Model	1,970	1,957
Parramatta River Model	4,482	4,520
Total	**11,448**	**11,496**

5.3.3. Culverts and Bridges

Culverts were modelled as 1D elements in TUFLOW. Details such as dimensions and inverts were primarily based on the stormwater asset dataset from Council. They were checked and updated against Structure Inspection Reports (Pitt & Sherry, 2019). Invert levels were also obtained from Development Application (DA) Work-As-Executed (WAE) drawings and from surveyed data in previous flood studies. LiDAR data was assumed for any other missing inverts. The same Manning's 'n' coefficient and loss coefficients were applied as for the stormwater pipes discussed above.

Road bridges and footbridges over the concrete-lined channels were modelled as 1D elements (combination bridge/weir structures to represent the flow below and above deck respectively). Bridges located in 2D portions of the model were represented by TUFLOW features known as "layered flow constrictions," as per best practice guidance. Bridge details were obtained from surveyed data in previous flood studies and were checked against the Structure Inspection Reports. Missing information for small footbridges such as deck thickness and railing height was estimated based on the site visit photos and Google StreetView images. The loss coefficients of bridge piers, deck and handrails were estimated according to Hydraulics of Bridge Waterways (Bradley, 1978 – Reference 16).

By inspection there were some high road bridges partially blocked in the LiDAR data, for example at Busaco Road, Khartoum Road and Wicks Road under M2 Motorway, as well as at

Christie Road, Delhi Road and Epping Road over M2 Motorway. Modifications to the DEM were made to ensure the appropriate representation of flow paths at these crossings.

5.3.4. Roads and Railway Corridors

To capture the overland flows in urban areas where kerbs and gutters were built in along roads. Gutter breaklines were digitised based on the aerial image and added into the models to lower the terrain by 0.1 m. This enables a better representation of the conveyance of gutter flows.

As LiDAR did not capture the elevation of the railway platforms in the study area (at Eastwood, Denistone, West Ryde and Meadowbank stations), terrain modification layers were digitised within TUFLOW to raise the platform zones by an assumed height of 1 m above ground.

5.3.5. Channels and Creeks

In the model of Terrys Creek, the concrete-lined channel starts from downstream of Terry Road in Braemar Park, approaches the Eastwood Town Centre, transitions into the covered channel at Progress Avenue, then turns into open channel next to the railway embankment, continues to the east of Blaxland Road and ends behind 4 Cassia Place. Cross sections of the channel were sourced from TUFLOW model layers from previous Flood Study (Reference 5), which were based on survey completed at the time. Channel associated layers were refined geographically. Details of the road culverts along the channel were checked and updated according to the Terrys Creek Capacity Assessment report (Reference 12). This channel is modelled in the 1D domain and is shown in Figure 23.

In the Parramatta River subcatchments (PR) model, there are a total of four 1D concrete-lined channels discharging into Parramatta River that were retained from the previous Flood Study (Reference 8). Three of these flow through Meadowbank Park and the other one flows through Morrison Bay Park. The channels and associated layers were refined geographically based on the LiDAR data. Channel sizes were reviewed and updated based on the Structure Inspection Reports (Pitt & Sherry, 2019). These channels are shown in Figure 23.

DEM modifications were included for the major creeks and for many of the flow paths in the 2D domain, to ensure that conveyance of water is adequate in the model. Creek inverts were sampled from the LiDAR data, using a search algorithm to find the minimum points, and levels were interpolated between the sampling points to provide a continuous representation of the channel flow capacity.

5.3.6. Basins and Ponds

Detention basins in open space that are normally dry were generally well represented in the LiDAR data, while bathymetry was not captured for basins/ponds that contain permanent standing water or basins with significant vegetation. In these locations, adjustments were made to bed levels by setting a constant value estimated based on the LiDAR data. For example, for the MQ Model, major in-creek detention basins along Mars Creek at the Link Road crossing, Waterloo Road crossing, and University Lake Yerbury were refined using estimates for basin inverts. Breaklines for the levee crest of the Dunbar Park Basin were adjusted geographically. In the PR Model, the Lardelli Lake detention basin near the Royal Rehabilitation Centre was refined. Several ponds within the Ryde-Parramatta Golf Course were digitised and assigned estimated bed levels.

5.4. Surface Roughness

The surface roughness in the previous Flood Studies was reviewed and updated based on the land use zoning data and by visual inspection of the aerial image (December 2020). Land use polygons were reviewed and refined. To be consistent across all consolidated models, each land use category was assigned a single Manning's 'n' coefficient, as summarised in Table 14 (after model calibration – see Section 6.1.3). The coefficients were checked against the recommended values in Table 10-1 and in Table 10-3 from ARR Project 15 Report (Reference 15) to ensure they were within reasonable ranges. Figure 24 illustrates the spatial distribution of the roughness values.

Table 14: Adopted Hydraulic Roughness as Manning's 'n' Coefficients

Land Use Type	Mannings 'n'				
	Harmonis-ation Study	**2008 Eastwood and Terrys Creek FS**	**2010 Macquarie Park FS**	**2014 Buffalo and Kittys Creek FS**	**2013 Parramatta River Ryde Sub-catchments FS**
Roads	0.02	0.02	0.02	0.02	0.02
Rail corridor	0.035	0.05	-	-	0.04
Residential houses (gardens, fences included)	0.1	0.1	0.1	0.05	0.1
Residential high density/Industrial/Commercial (carparks/paved area included)	0.025	0.025	0.025	0.02 / 0.06	0.025 / 0.04
Maintained grass / parks / ovals	0.03	0.03	0.03	0.03	0.03
Moderate vegetation	0.06	-	0.06	-	0.05
Thick vegetation	0.08	0.08	0.1	0.1	0.07
Water ponds	0.02	-	0.02	-	0.03
Concrete channel (1D)	0.015	0.02	0.02	-	0.025
Buildings	Blocked Out	20	20	-	Blocked Out

5.5. Boundary Conditions

5.5.1. Inflows

Inflows for each subcatchment into the TUFLOW model were runoff hydrographs produced by the DRAINS model. The type and geometry of inflow boundaries used from the previous studies were different. For consistency in this study the approach was standardised across all models. 1D inflows into the 1D channels from previous studies were retained. 2D inflows were applied onto pits where subcatchments drain to pits and were applied at the subcatchment outlet on the ground elsewhere. Inflows into 1D elements were applied directly to the structure and 2D inflows were applied onto the 2D surface. These flows are assigned either directly to the surface at pit inlets, or to the lowest/wet cells where no pits are present. The locations of pit inflows were adjusted where appropriate based on the location of the updated stormwater network. A summary of inflows in previous and current study is in Table 15. They are illustrated on Figure 25. For the MQ model, in addition to the inflows from DRAINS (for both local creek catchments and broader Lane Cove River catchments outside the Ryde LGA), the flow hydrographs of Terrys Creek produced at the downstream boundary of the TC model were imported as 2D inflows near the upstream boundary of the MQ model extent.

Table 15: Summary of Inflows Boundaries

<table>
<tr><th rowspan="2">Models</th><th colspan="3">Previous Flood Studies</th><th colspan="3">Harmonisation Study</th></tr>
<tr><th>TUFLOW Inflow Boundary</th><th>Number</th><th>Total</th><th>TUFLOW Inflow Boundary</th><th>Number</th><th>Total</th></tr>
<tr><td rowspan="3">TC Model</td><td>1d_bc (nodes)</td><td>241</td><td rowspan="3">945</td><td>1d_bc (nodes)</td><td>16</td><td rowspan="3">994</td></tr>
<tr><td rowspan="2">2d_sa (nodes)</td><td rowspan="2">704</td><td>2d_sa (polygons - pits)</td><td>886</td></tr>
<tr><td>2d_sa (polygons - non pits)</td><td>42</td></tr>
<tr><td rowspan="2">MQ Model</td><td>1d_bc (nodes)</td><td>734</td><td rowspan="2">2454</td><td>2d_sa (polygons - pits)</td><td>2309</td><td rowspan="2">2454</td></tr>
<tr><td>2d_sa (nodes)</td><td>1720</td><td>2d_sa (polygons - non pits)</td><td>145</td></tr>
<tr><td rowspan="2">BK Model</td><td>1d_bc (nodes)</td><td>1708</td><td rowspan="2">1708</td><td>2d_sa (polygons - pits)</td><td>1702</td><td rowspan="2">1708</td></tr>
<tr><td>2d_sa</td><td>0</td><td>2d_sa (polygons - non pits)</td><td>6</td></tr>
<tr><td rowspan="2">PR Model</td><td>1d_bc</td><td>0</td><td rowspan="2">1439</td><td>2d_sa (polygons - pits)</td><td>1035</td><td rowspan="2">1437</td></tr>
<tr><td>2d_sa (polygons)</td><td>1439</td><td>2d_sa (polygons - non pits)</td><td>402</td></tr>
</table>

5.5.2. Outflow Boundaries

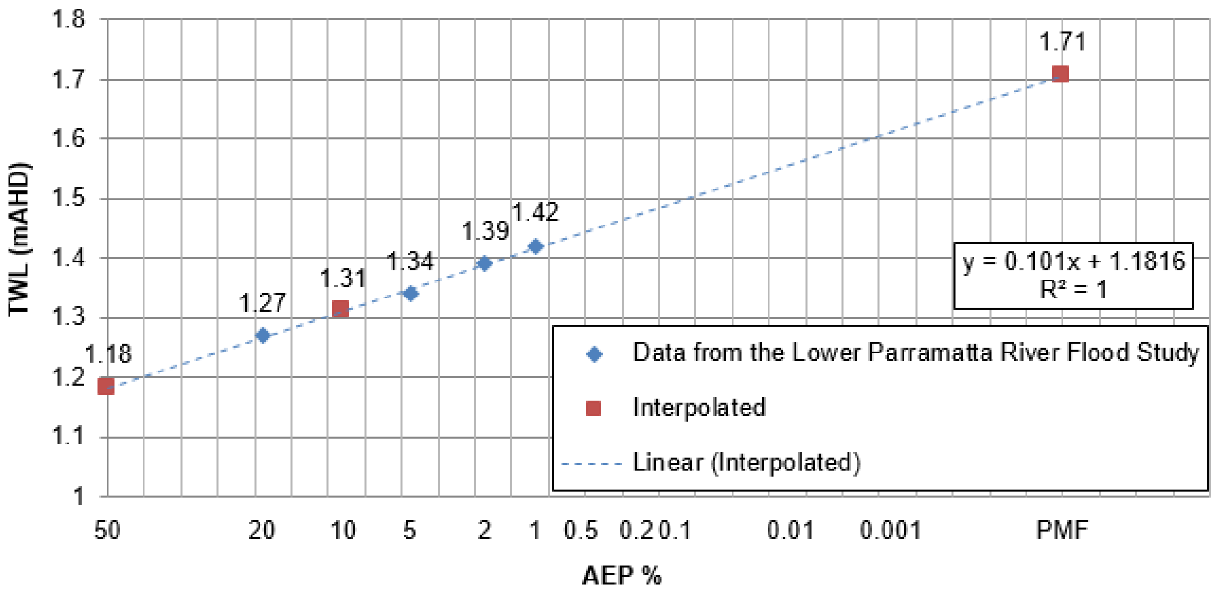
The downstream model boundaries were updated based on the refined model extents of the consolidated models. They were set up such that water levels within the study area should not be significantly influenced. The height-discharge rating curve adopted for the Eastwood and Terrys Creek FS (Reference 5) and Macquarie Park FS (Reference 6) were retained as the downstream boundary for the TC and MQ TUFLOW models, respectively. The TC model boundary is just downstream of the M2 Motorway crossing of Terrys Creek and the MQ boundary is just downstream of the Epping Road crossing of the Lane Cove River. In simulating the design flood events, it was found that the MQ boundary was sufficiently downstream such that tidal levels in the Parramatta River would dominate and the MQ downstream boundary was changed to be a tidal boundary for the design flood events.

The tidal boundaries were applied to the PR, BK and MQ models, based on water levels in the Parramatta River. Tide levels were analysed at the Fort Denison gauge for the Parramatta River as part of the Lower Parramatta River Flood Study (Reference 17), undertaken for City of Parramatta Council. This analysis was based on the frequency and magnitude of high tides at Fort Denison, as shown in Table 16 and Diagram 10. Estimates of the 50% Annual Exceedance Probability (AEP), 10% AEP and an Extreme water level (PMF) were estimated based on the water levels provided.

Table 16 Tailwater Levels at Fort Denison

AEP	Tailwater Level (mAHD)
50%	1.18
20%	1.27
10%	1.31
5%	1.34
2%	1.39
1%	1.42
PMF	1.71

Diagram 10 Interpolation of Tailwater Levels at Fort Denison



Additional stage-discharge boundaries were added in to allow water to exit the models at other locations where cross-catchment flows can potentially occur in extreme storm events. The rating curve at these outflow boundaries was generated automatically by TUFLOW based on estimated water surface slopes. The locations of these boundaries are shown in Figure 25.

5.5.3. 1D/2D Boundaries

Internal boundaries between 1D and 2D domain were reviewed and updated, including the interface between inlet pits and the ground surface, between culvert headwalls and the ground surface, between concrete channels and the adjacent overbank areas, as well as at the links of any other cross drainage features (i.e. tunnels and refined flow paths).

5.5.4. Initial Conditions

Initial water levels were set up to be consistent with the downstream boundary conditions. For example, where the Parramatta River tidal level was adopted for the downstream boundary conditions, the initial water level was also assumed to be at the same level. It was assumed there was no initial water in the free-draining detention basins, and initial water levels in permanent lakes/ponds were based on the level inferred from LiDAR information.

6. MODEL CALIBRATION

The aim of the calibration process is to ensure the modelling system can replicate historical flood behaviour. There are assumptions in the modelling inputs, such as the effect of vegetation on flow and the amount of infiltration into the soil, which can be adjusted to improve the match between observed and modelled flood levels. A good match to historical flood behaviour provides confidence that the modelling methodology and schematisation can accurately represent the important flood processes in the catchment.

Typically, in urban areas the records of flood levels are insufficient for a thorough calibration/validation process to be undertaken. Issues which require consideration when undertaking the calibration of urban hydrologic and hydraulic models are:

- Streamflow gauges are generally unavailable and there is only a limited amount of historical flood information available for the study area.
- Rainfall records and particularly pluviometer records for past floods within the catchment are limited. Rainfall gauges provide point measurements of rainfall which require spatial interpolation to generate an estimate of the total rainfall across the catchment. These estimates can have errors in the order of 50% depending on the density of gauges and the nature of the event. Flood-producing storms in the study area will tend to be convective storm events associated with broader scale weather patterns such as “East Coast Lows,” and these convective storm cells can produce highly localised intense rainfall bursts which are not well recorded by rainfall gauges. Radar measurements can provide a qualitative understanding of the movement and location of the storm bursts, but this information is not available for the historical events of interest.
- Changes to the catchment due to urban development may result in significant changes to topography, land uses and drainage structures.
- The duration of flooding in urban area is generally short. Accurate peak levels are hard to be recorded, especially when the storm occurs at night. Flood depths are shallow compared to riverine flooding, and flood level or depth observations can be highly influenced by localised features such as walls or vehicles. Observations of flood height are therefore generally less accurate than in broader riverine floodplains.

These limitations are typical of the majority of urban catchments. The calibration exercise undertaken here is in accordance with recommended practice as outlined in the Australian Rainfall and Runoff Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains (Reference 15).

6.1. Methodology

A joint calibration of hydrologic and hydraulic models was undertaken, whereby historical rainfall data (intensities and temporal patterns) were input to the hydrologic model and resulting peak flood levels were obtained from the hydraulic model. Parameters in both the hydrologic and hydraulic models were adjusted within acceptable bounds to ensure a match between the recorded and the model historical flood levels.

6.1.1. Selection of Historical Events

The selection of calibration/validation events depends on a combination of the severity of the flood event and the quality of the data available. Even if numerous observations of flooding are available from a particular storm event, the ability to utilise this information to calibrate the models can be limited by the quality of the rainfall data for the storm, since this is the most important input required to simulate the event. Generally speaking, the further in the past an event occurred, the less reliable the records of that flood will be, unless they were obtained and well documented immediately after the event.

The primary sources of historical flood information considered were the previous flood studies, Council's register of resident complaints and damages, flood marks, photos and other information from Council.

Notable flood events within the study area identified as part of the previous flood studies, include November 1984, August 1986, December 1989, February 1990, March 1990, February 2010 and April 2012. The historical events that were used for calibration/validation in the previous flood studies are summarised in Table 17. Based on the Buffalo and Kittys Creek FS report (Reference 7), the 1990 event was not modelled and the flood information was only used as a qualitative check of the model. This is a common approach for urban flood studies. Surveyed marks of flood levels are only available in the Eastwood and Terrys Creek catchment and the Macquarie Park catchments.

Table 17: Calibration Events in Existing Flood Studies

Flood Study	Event	Flood Data
Eastwood and Terrys Creek	Nov 1984	Council database: flood depths Surveyed flood levels from the 1991 Study and 2005 Study
Macquarie Park	Nov 1984	Community survey and Council database: flood depths and flood marks with surveyed levels
	Feb 1990	Community survey and Council database: flood depths
Buffalo and Kittys Creek	Feb 1990	Community survey: flood depths
Parramatta River	Nov 1984	Community survey and Council database: flood depths
	Feb 1990	

Table 18 and Table 19 summarise the resident complaints by month from Council's register (top 5 months with the most complaints are highlighted). It indicated that there were limited data entries since 2010. The three events with the most complaints were in February 1990, December 1989, and November 1984. The flood observations were extracted from the Council's database into GIS files for calibration. Where good descriptions of the location were not available it was necessary to infer the location using the provided information.

Table 18: Council Complaints Register with Approximate Depths of Flooding

Event	Entry	Event	Entry	Event	Entry	Event	Entry	Event	Entry
1984 Nov	134	1988 Dec	1	1989 Apr	1	1992 Dec	4	2003 May	1
1986 Nov	3	1988 Feb	1	1990 Oct	1	1992 Feb	4	2018 Nov	17
1986 Feb	2	1988 Apr	3	1990 Nov	1	1995 Sep	1		
1986 Aug	55	1988 May	14	1990 Dec	4	1996 Jan	1		
1986 Sep	1	1988 July	14	1990 Feb	328	1997 Oct	1		
1987 Oct	8	1988 Aug	1	1990 Mar	42	1998 May	2		
1987 Dec	1	1988 Sep	1	1990 Aug	2	2002 Nov	1		
1987 Aug	1	1989 Jan	2	1990 Sep	2	2002 Feb	3		
1988 Nov	1	1989 Dec	257	1991 June	1	2003 Oct	1		

Table 19: Council Complaints Register with Qualitative Information (Since 2010)

Event	Entry	Event	Entry	Event	Entry	Event	Entry
2010 Feb	1	2012 Feb	1	2015 Oct	1	2018 Mar	1
2010 Mar	3	2012 Mar	5	2015 Feb	1	2019 Feb	2
2010 June	1	2012 Apr	6	2015 Apr	1	2020 Feb	4
2011 Oct	1	2012 June	1	2015 June	1	2020 Apr	1
2011 Dec	1	2013 Mar	1	2015 July	1	2020 May	1
2011 Apr	1	2014 Jan	1	2016 Jan	1	2020 June	1
2011 July	3	2014 Aug	1	2017 Mar	1	2021 Jan	1
2012 Jan	1	2015 Jan	2	2018 Nov	2	2021 Mar	1

Table 20: Summary of Available Data Type from Council

Event	Available data type	Event	Available data type
8/11/1984	flood marks, observations, photos	20/04/2015	social media images
5/12/1989	flood marks, observations, photos	5/06/2016	photos, videos, social media
12/02/2007	observations	21/03/2017	videos
26/10/2007	observations, photos	28/11/2018	photos, social media images
12/02/2010	flood marks, observations, photos	20/12/2018	social media images
8/03/2012	observations, photos	27/11/2019	social media images
18/04/2012	flood marks, photos, social media	9/02/2020	social media images
18/08/2014	social media images	30/04/2020	social media images

Additional available flood information from council's database is shown in Table 20. It indicated that flood observations were most suitable for use in the calibration from the November 1984, December 1989, and February 2010 events. For the December 1989 event, flood marks including debris marks and water marks on buildings were at the West Ryde shopping centre only without accurate surveyed levels.

The relative scarcity of complaints from the last decade indicates there have not been additional major flood events during this time. At the inception of the study, Council indicated to WMAwater that there had not been additional major events in the last decade that has caused widespread flood damage or complaints comparable to the November 1984 or February 1990 storms, which were previously used for calibration and remain the most significant floods of record. There were also several complaints about flood damage from November 2018, primarily within the Eastwood and Terrys Creek catchment, and particularly around Rowe Street at the Eastwood Town Centre. Given the relative recency of this event it was also included as a validation event for the TC model.

Based on the analysis of the historical flood information above, along with the available pluviograph rainfall data in Table 7 (see in Section 3.7.2), the events selected for calibration for each of the consolidation model are listed in Table 21.

Table 21: Calibration Events

Model	Selected for Calibration
TC Model	8/11/1984
	12/02/2010
	28/11/2018
MQ Model	8/11/1984
	7/02/1990
BK Model	7/02/1990
PR Model	8/11/1984
	7/02/1990

6.1.2. Rainfall Patterns and Losses

For the calibration events, following figures were created to understand the historical rainfall data:

- Spatial patterns and rainfall isohyets are in Figure 8, Figure 11, Figure 14 and Figure 17.
- Temporal patterns and cumulative rainfall total are in Figure 9, Figure 12, Figure 15 and Figure 18.
- Burst intensities against the ARR16 IFD data are in Figure 10, Figure 13, Figure 16 and Figure 19.

The approximate equivalent flood magnitude in terms of design AEP according to the ARR16 IFD was estimated for the calibration events in Table 22.

Table 22: Estimated Storm Magnitude of the Calibration Events

Historical Event	Estimated AEP 1 to 2 hour duration	Estimated AEP 6 hour duration
8/11/1984	20% AEP (West Epping Bowling Club) to 1% AEP (Ryde Bowling Club)	20% AEP (West Epping Bowling Club) to 10% AEP (Ryde Bowling Club)
7/02/1990	20% AEP (West Epping) to 2% AEP (Pymble Bowling Club)	20% AEP (Pymble Bowling Club) to 5% AEP (Chatswood Bowling Club)
12/02/2010	20% AEP (Pymble Bowling Club) to 1% AEP (West Epping Bowling Club)	Less than 10% AEP
28/11/2018	20% AEP (Pymble Bowling Club) to 1% AEP (Chatswood Bowling Club)	Less than 20% AEP (Pymble Bowling Club) to 2% AEP (Chatswood Bowling Club)

Due to the spatial variability of rainfall within the study area, the rarity of these events varied considerably based on location. There is a consistent pattern to the historical events considered whereby the short burst intensities are considerably more intense at the West Epping Bowling Club to the north-west of the study area relative to the other pluviograph gauges further south-east within the study area. This is likely due to orographic effects, as the West Epping gauge is located at higher elevation, and the prevailing wind direction during East Coast Low storm events is likely to be from the east/south-east.

Since the only pluviograph gauges located within the study area are at West Ryde Pumping Station (within the PR model extent) and at Gladesville Bowling Club (next to the PR model and BK model boundary), rainfall depths and temporal patterns into the other models were applied from other available nearby gauges. Table 23 summarises the selected gauges for each model. It was noted from Figure 12 and Figure 13 that the recorded rainfall at the West Ryde Pumping Station and the West Epping Bowling Club were identical in the 1990 event, which is considered highly unlikely. It is possible one of the gauges was supplemented with data from the other. The quality code of data at the West Ryde Pumping Station was further checked through the BoM water data website[2]. It declares that the data was of Quality C, which was an estimate in February 1990. This finding increases the uncertainty when calibrating the models for the 1990 event. In the 1984 event for the MQ Model, based on the previous flood study, the Marsfield storm pattern as in Diagram 11 was adopted for the Mars Creek and Shrimptons Creek catchments and reduced by 20% for the neighbouring Ryde study area catchments to reflect reduced rainfall totals in those areas. The remaining Lane Cove River sub-catchments were modelled with 65% of the Marsfield pattern to reflect the lower rainfall event totals recorded at Chatswood.

2 http://www.bom.gov.au/waterdata/

Diagram 11: November 1984 Marsfield Rainfall Temporal Pattern (from 3 am) (Source: Reference 6)

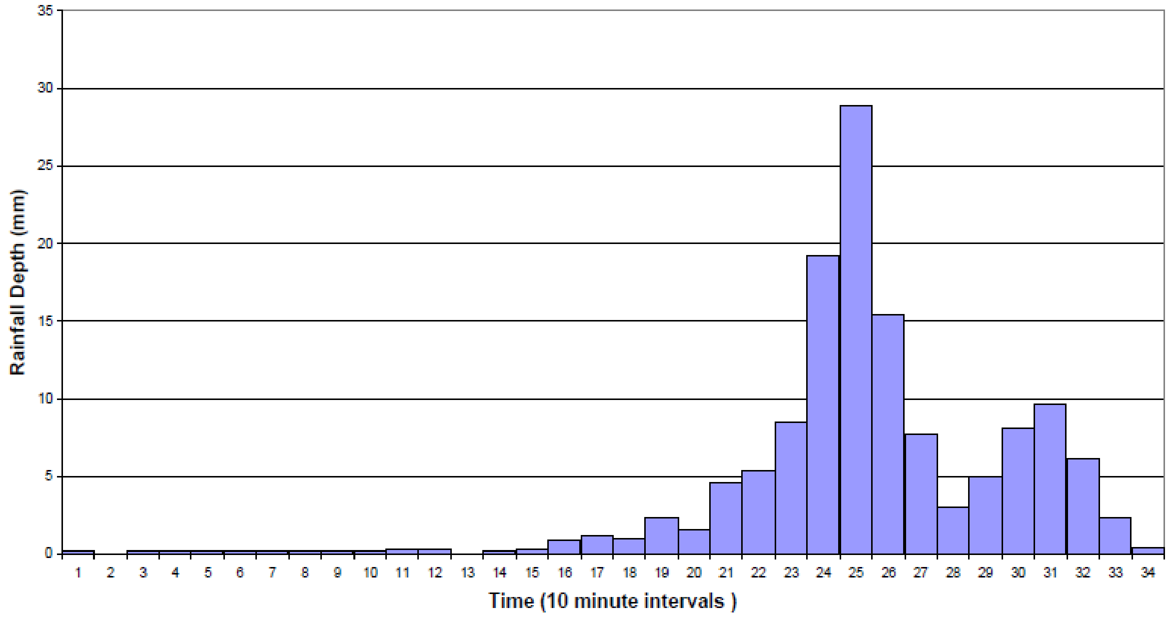


Table 23: Rainfall Gauge Data Selected for Calibration of Each Model

Model	Calibration Event	Gauge Selected for Calibration	Based on
TC Model	8/11/1984	West Ryde Pumping Station 566037	Previous flood studies
	12/02/2010	West Ryde Pumping Station 566037 West Epping Bowling Club 566040	Gauge location
	28/11/2018	West Ryde Pumping Station 566037	Gauge location
MQ Model	8/11/1984	Marsfield Station by Macquarie University	Previous flood studies
	7/02/1990	West Ryde Pumping Station 566037 Chatswood Bowling Club 566017	Previous flood studies and quality of rainfall data
BK Model	7/02/1990	West Ryde Pumping Station 566037 Chatswood Bowling Club 566017	Gauge location and quality of rainfall data
PR Model	8/11/1984	West Ryde Pumping Station 566037	Previous flood studies and gauge location
	7/02/1990	West Ryde Pumping Station 566037	Previous flood studies and gauge location

6.1.3. Calibration Process

The process of calibrating the models involved the adjustment of the model parameters where appropriate to more accurately reproduce observed behaviour. Parameters in the DRAINS models such as soil type, depression storage and AMC remained unchanged as specified in Section 4.3 since they well-represented the catchment conditions in a consistent manner and the calibration results did not warrant adjustment.

In several locations there have been changes to the topography or drainage network as a result of infrastructure works or development at locations of interest. Some of these works were in direct response to flood damages that occurred during the calibration events. It was therefore necessary to adjust the models to reflect the conditions prior to these modifications for the relevant events. The changes since 1984 compared to the current conditions including new embankments within parks, drainage network upgrades, new culverts, new development, changes to vegetation and creek rehabilitation works. The major changes that would affect the results at the location of flood marks were incorporated into the models to resemble the historical conditions, as summarised in Table 24.

Table 24: Model Changes for the Historic Conditions

Model	Event Date	Model Adjustment
TC Model	8/11/1984, 12/02/2010 and 28/11/2018	• Restore pit and pipe around Railway Pd, Fourth Ave, Vimiera Rd & Abuklea Rd to represent historical conditions prior to capital works upgrades completed by Council. • Remove roundabout at Hillview Rd and West Pd intersection to represent historical conditions. • In the 2018 event, add back yard fences and a 2 m gap through shops at 100-104 Rowe St to account for flow occurring through the doors and corridors of these buildings.
MQ Model	8/11/1984 and 7/02/1990	• Remove Dunbar Park detention basin embankment (as per Reference 6)
BK Model	7/02/1990	None
PR Model	8/11/1984 and 7/02/1990	• Remove West Ryde Tunnel by blocking inlets to represent conditions prior to the construction of this pipe. • Remove overland flow culverts across Constitution Rd near Ann Thorn Park • Restore pit and pipe conditions at Richard Johnson Cres as per Reference 8

Sensitivity tests suggested that the results were most sensitive to the assumed rainfall depths and temporal pattern than the other hydraulic parameters in the study area. The differences between modelled and observed results were broadly within the range of results bounded by the inherent uncertainties in these rainfall inputs. This is particularly noticeable in the results for the February 2010 event, which varied significantly depending on whether the rainfall data from the

West Ryde Pumping Station or West Epping Bowling Club was used. It was therefore not justifiable to modify the catchment-wide modelling parameters significantly from the initial values, which were selected based on recommended defaults or experience with similar catchments in the region.

Localised changes were made to the Mannings ‘n’ roughness parameter where appropriate based on the available information to improve the model calibration. The locations where the roughness was modified were as follows:

- Initial results indicated that the model produced excessive overland flow from Terrys Creek through the Eastwood town centre for the calibration events. This behaviour was found to be sensitive to the roughness and conveyance of the upstream open channel through Glen Street Reserve. The Mannings ‘n’ value was reduced to 0.015 from the initial value of 0.02, producing an improved match. This change was also applied to other concrete lined channels across the LGA.
- The Mannings ‘n’ value along the riparian corridor of Shrimptons Creek in Santa Rosa Park was increased from an initial value of 0.03 in the previous 2010 Macquarie Park Flood Study (Reference 6) model to 0.08. This value more accurately reflected the observed vegetation during the site visit, and improved the calibration results.

The Mannings ‘n’ values tabulated in Section 5.4 reflect the final values after the model calibration process.

6.2. Calibration Results

The calibrated model results were compared to the surveyed and flood levels and the approximate observed depths for each model in the selected flood events. The differences at these reported locations are mapped on Figure 26 to Figure 33. Full tables of comparisons with both recorded flood levels and depths, as well as the historical photos are included in Appendix B to Appendix E.

A summary of the calibration results is provided in Table 25. The following criteria were used to classify the result for each flood observation:

- “Excellent fit” refers to differences within ±0.1 m
- “Fair fit” if difference is within ±0.3 m

Generally speaking, the modelling results fit reasonably well to the recorded data historical flood data. The consolidated TUFLOW models are considered to be suitable for the estimation of design flood behaviour in the study area. Discussion of the results for each of the different model areas is provided below.

Table 25: Overview of Calibration Results

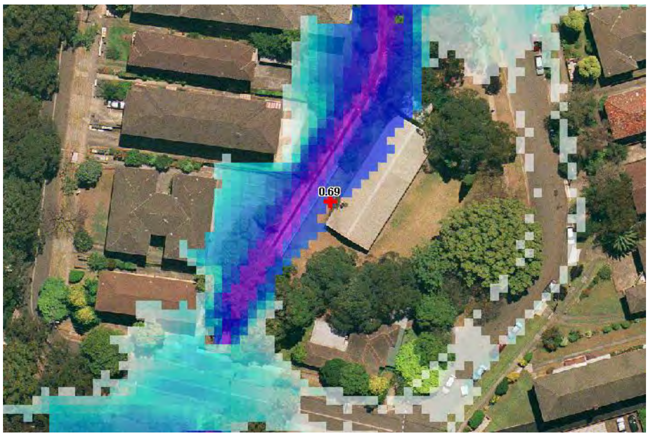
Model	November 1984	February 1990	February 2010	November 2018
TC Model	75% of surveyed levels and 66% of estimated depths within fair range	N.A.	58% of levels and 89% of depths within fair range (using West Ryde Pumping Station rainfall data) 26% of levels and 17% of depths within fair range (using West Epping Bowling Club rainfall data)	17 out of 18 depths within "fair" range
MQ Model	3 out of 6 surveyed levels "excellent" and all observed depths within "fair" range	74.5% of depths within "fair" range	N.A.	N.A.
BK Model	N.A.	60% of depths within "fair" range		
PR Model	all depths within fair range	87% depths within "fair" range		

6.2.1. Terrys Creek and Eastwood (TC) Model

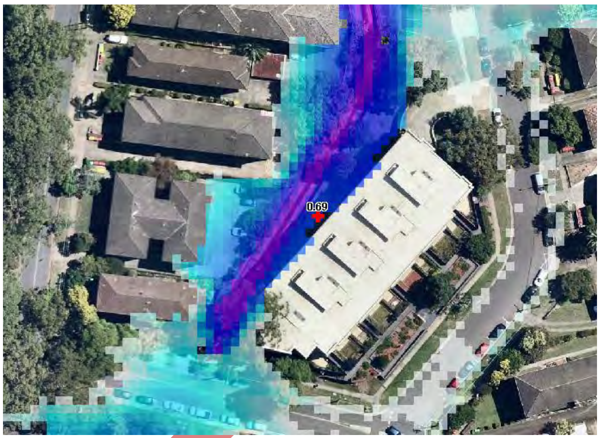
The model fits reasonably well in the Eastwood and Terrys Creek Catchments with few exceptions. For the 1984 event, a comparison with surveyed levels is provided in Table 26. At most locations, a fair fit could be achieved. Apart from the uncertainties of raw data in terms of the exact survey location and accuracy, the following considerations are relevant at the locations where the difference was not within the fair fit range:

- Debris marks are less reliable in conveyance areas where flood levels were significantly influenced by turbulence, especially where measured near a structure with flow restrictions. For example, the flood mark with ID "FS 6" was said to measure at channel wall and driveway. By inspection, the driveway was on a bridge over the channel which was overtopped with a shallow depth of ~0.2 m based on the modelled result. The flow behaviour at this location would be heavily influenced by the local effect of the bridge deck and highly sensitive to small fluctuations in the water level in the channel.
- Local topography changes around the surveyed location occurred since 1984 due to the development. For example, the modelled flood level at 20 Ball Avenue, Eastwood (FS 65) is higher than the surveyed level because overbank flows have been modified locally by the new apartment building and the associated raised ground levels as shown in Diagram 12 below. The house at 21 Denistone Road, Eastwood was a similar case subject to redevelopment and ground level changes. The construction of the Eastwood Library in 1991 also appears to have affected the flow behaviour in the vicinity, affecting the accuracy of the calibration match to flood observations immediately upstream.

Diagram 12: Calibration Results at Ball Avenue Eastwood, 2001 versus 2020 Aerial Image



20 Ball Ave Eastwood (2001 Aerial Image)



20 Ball Ave Eastwood (2020 Aerial Image)

- Levels and depths inside garages or buildings can be very different to the adjacent modelled flood results. For example, along Railway Parade, Eastwood (FS 41 and FS 45), the observed depths over floor were much lower than the modelled depths sampled from the road adjacent to the building. It is to be expected that water levels within the buildings will be lower due to the limited capacity for ingress from walls and closed doors preventing the full equalisation of water levels.
- Drainage modifications from upgrades, reconstruction and demolition in the past can lead to localised topography changes. An example is at 46 Fourth Avenue, Denistone (FS 61), where reconstruction drawings from Council identify modifications to the vehicle crossing, footpath and driveway into the property garage in 2014. The ground levels are slightly higher than in 1984, and the resultant flood level in the model was slightly higher than the surveyed level in garage

For the 2010 event, a comparison with surveyed levels is provided in Table 27. Since there was no pluviometer located within the catchment. Rainfall depths and temporal patterns from two nearby gauges, at West Epping Bowling Club (566040) and at West Ryde Pumping Station (566037), were applied as a sensitivity test. It indicated that results were generally higher than the recorded flood levels using the West Epping rainfall, while they were generally lower but closer to the recorded data using the West Ryde rainfall. This sensitivity indicates that the rainfall data is the critical contributing factor of the discrepancies and therefore fine tuning of model parameters to produce a closer match was not warranted, since there is insufficient rainfall data available to generate a more accurate understanding of the actual rainfall that occurred.

For the 2018 Event, a comparison with the observed depths is provided in Table 28. There is a good agreement between most flood observations and modelled flood behaviour. The modelled peak flood depths were slightly greater than observed in general. This could possibly be explained by the peak flood levels occurring early in the morning (between 6:30 am and 7:30 am) before the shopkeepers arrived. The depths provided are based on the debris marks when the flood subsided and could be underestimated to some extent. CCTV footage during the event at Hillview Lane next to the Terrys Creek was found in an online community forum[3]. It was

3 https://fb.watch/asGXDf2FWZ/, viewed January 2022

recorded at 6:37 am for about 30 seconds (assumed to be local daylight savings time). This is close to the modelled time of the peak (assuming that the rainfall data used to simulate the flood was not adjusted for daylight savings, and hence the modelled flood hydrograph was shifted forward in time by one hour). The peak flood depth near the entrance of the double box culverts was approximately 2.3 m to 2.4 m based on estimation of the position of the flood water surface in the video clip. The modelled peak flood depths upstream of the box culverts varied from 2.2 m to 2.5 m in the channel, which is considered a good match. A time series of the modelled flood depth upstream of the box culverts is shown in Diagram 13. The time of the video is also marked.

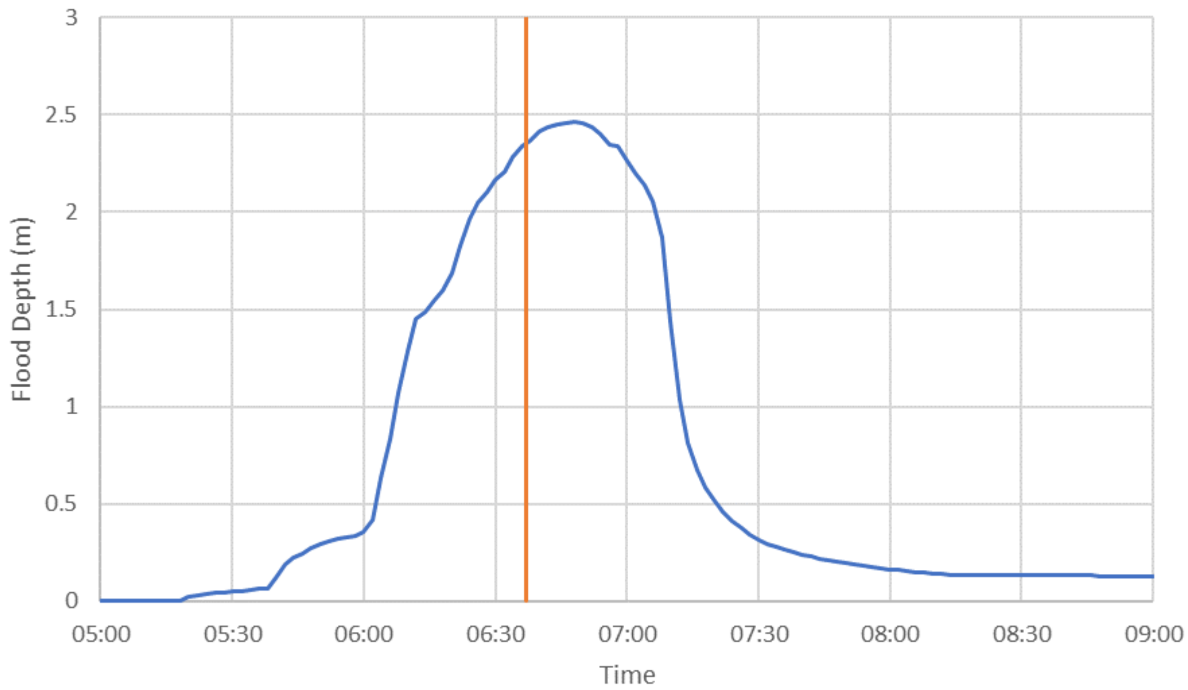


Diagram 13: Modelled peak flood depth in Terrys Creek adjacent to Hillview Lane for the November 2018 event. Time of CCTV footage marked by the orange line.

Table 26: Comparison of TC Model Results with Historical Levels for the 1984 Event

ID	Location	Observed Level (m)[3]	Observed Depth (m)	Modelled Level (m)	Modelled Depth (m)	Level Difference (m)[1]	Depth Difference (m)[1]	Quality[2]	Description (from Reference 5)
FS1	21 Terry Road, Eastwood	73		73.29	0.62	0.29		OK	BC-25, level in garage.
FS6	22 Auld Avenue, Eastwood	70.2		70.71	1.63	0.51		OK	BC-33, Level at channel wall and driveway.
FS13	190 Shaftsbury Road, Eastwood	69.52		69.61	0.65	0.09		OK	BC-34, Level in house.
FS14	1/3 Darvall Road, Eastwood	84.6		84.59	0.25	-0.01		OK	BC-46, Level at front garden - redevelopment took place since 1984
FS17	68 Rutledge Street, Eastwood	79.6		79.38	0.17	-0.22		OK	BC-44, Level at side gate.
FS18	66 Rutledge Street, Eastwood	79.3		79.16	0.25	-0.14		OK	BC-43, Level in house.
FS24	293 Rowe Street, Eastwood	71.87		71.86	0.25	-0.01		OK	BC-45, Level at front veranda.
FS25	4 Richards Avenue, Eastwood	70.3		70.02	0.64	-0.28		OK	BC-34, Level in house and depth over floor.
FS26	2 Richards Avenue, Eastwood	70.15		69.99	0.59	-0.16		OK	BC-35, Level in house and depth over floor.
FS35	10 Hillview Road, Eastwood	65.84		66.12	0.66	0.28		Low	BC-58, Just Rags
FS38	4 Hillview Road, Eastwood	65.58		65.73	0.48	0.15		OK	BC-54, Previously Beckers Hairdressers
FS40	6 Hillview Road, Eastwood	65.87		65.88	1.10	0.01		OK	BC-56, Previously Mr Craft
FS41	16 Railway Parade, Eastwood	68.37	0.15	68.33	0.66	-0.04	0.51	OK	BC-62, Previously Towntalk Butchery, Depth over floor.
FS45	13 Railway Parade, Eastwood	68.21	0.1	68.33	0.77	0.12	0.67	OK	BC-63, Previously Commonwealth Bank, Depth over floor.
FS55	263 Ryedale Road, Eastwood	79.28	0.25	79.29	0.41	0.01	0.16	OK	BC-64, Level and depth in garage.
FS59	21 Denistone Road, Eastwood	81.56	0.3	82.72	0.04	1.16	-0.26	OK	BC-65, Level and depth in garage.
FS61	46 Fourth Avenue, Eastwood	85.2	0.5	85.60	0.51	0.4	0.01	OK	BC-66, Level and Depth in garage.
FS65	20 Ball Avenue, Eastwood	61		61.69	1.53	0.69		OK	BC-70, Level at corner of hall.
FS66	2 Ball Avenue, Eastwood	59.9		61.08	1.29	1.18		Low	BC-71, Level at garage.
FS67	13 Bertram Street, Eastwood	58.51		59.05	1.16	0.54		OK	BC-72. Level under house.
FS70	5 Cassia Place, Eastwood	58.47	0.75	58.47	0.05	0.00	-0.7	OK	BC-74, Level and depth in caravan port.
FS72	26b Vimiera Road, Eastwood	57.84		57.75	0.34	-0.09		Low	Depth over floor. (+0.25m)
FS86	1 Eastwood Avenue, Eastwood	66.17		66.34	0.70	0.17		OK	BC-67, Level in garage.

[1] Difference was calculated as modelled result - recorded flood data

[2] General quality of flood data was estimated from High, OK, and Low based on the given location, measure of accuracy and source.

[3] Photos are not available at these locations

Table 27 Comparison of TC Model Results with Historical Levels for the 2010 Event

ID	Location	Observed Level (m)	Observed Depth (m)	West Epping Bowling Club 566040				West Ryde Pumping Station 566037				Quality	Description (from Council flood records)	Photo ID
				Modelled Level (m)	Modelled Depth (m)	Level Difference (m)	Depth Difference (m)	Modelled Level (m)	Modelled Depth (m)	Level Difference (m)	Depth Difference (m)			
1	2 Hillview Road, Eastwood Club. Corner of Coolgun Lane and Hillview Road.	65.27	0.27	65.70	0.86	0.43	0.59	64.91	0.08	-0.36	-0.19	High	Flood Mark on the external wall. Level taken on the corner of the external building wall.	1
2	8 Hillview Road, Eastwood. Corner of Coolgun Lane and Hillview Road.	65.33	0.42	65.88	1.08	0.55	0.66	64.89	0.08	-0.44	-0.34	High	Flood depth taken on the corner of the external wall approximately 420 mm above the footpath level. Approximately 230 mm above floor flooding within store. Substantial damage to contents.	2, 3, 4
3	Bing Lee, 2 Progress Avenue, Eastwood.	65.44	0.52	66.37	1.63	0.93	1.11	64.98	0.30	-0.46	-0.22	High	Flood level taken in the rear loading area at Coolgun Lane. 220 mm above floor flooding. Flood water entered from Progress Avenue. Excessive damage to contents.	5,6,7
4	Hill View Lane, Eastwood. Adjacent to Sydney Water open channel (yellow mark on the wall)	66.18	0.31	66.77	1.05	0.59	0.74	66.12	0.40	-0.06	0.09	High		8
5a	Glen Reserve/ Sydney Water open channel, Eastwood.	66.14	-	66.93	0.91	0.79	-	66.26	0.23	0.12	-	High	Reserve and Council's car park flooded. Refer photos for flood levels.	9,10, 11,12
6	188 Shaftsbury Road, Eastwood	69.11	1.1	69.42	1.71	0.31	0.61	68.83	1.12	-0.28	0.02	High	Storage area was flooded to a depth of 1.1 m. Fences, an old footbridge and plants were washed away.	13, 14
9	6-10 First Avenue, Eastwood	70.6	0.37	71.13	0.96	0.53	0.59	70.48	0.31	-0.12	-0.06	High	All garages were flooded. Not inspected internally.	19, 20
10	6-10 First Avenue, Eastwood (Garage 21)	70.76	0.44	71.13	0.94	0.37	0.5	70.54	0.35	-0.22	-0.09	High	Flood mark on garage no 21.	21
11	Foot bridge - Vimiera Road, Eastwood	56.63	0.6	56.69	4.11	0.06	3.51	56.27	3.69	-0.36	3.09	High	Footbridge obstructed the creek flow; The bridge frame collected and retained a large amount of debris.	22
12	5 Vanimo Place, Eastwood	57.72	0.87	58.14	1.39	0.42	0.52	57.57	0.82	-0.15	-0.05	High	Flood depth was taken in the front garage. Two cars were damaged. Flooding caused by the creek over bank flow. Only the garage flooding	23, 24
13	7 Vanimo Place, Eastwood	57.71	0.58	58.14	0.99	0.43	0.41	57.57	0.42	-0.14	-0.16	High	Flood depth was taken at the entrance of the house. 220 mm flooding inside the house.	25, 26
14	9 Vanimo Place, Eastwood	57.68	0.57	58.22	1.18	0.54	0.61	57.67	0.64	-0.01	0.07	High	A flood depth of 570 mm was taken in the garage. 440 mm flooding inside the living room. Caused by creek flooding. One car was damaged.	27, 28, 29
16	Vanimo Place cul-de-sac, Eastwood	57.8	1.7	58.14	2.00	0.34	0.3	57.57	1.44	-0.23	-0.26	High	Road flooding. Road is lower than the top of creek embankment. Backwater effect from the creek.	31
17a	1 Abuklea Road, Eastwood	62.97	-	62.97	0.71	0.00	-	62.47	0.21	-0.5	-	High	Debris on the rear fence and front gate brick wall. Flooding was caused by lack of overland flow route.	44, 45
18	1A Abuklea Road, Eastwood	63.39	0.08	63.49	0.34	0.1	0.02	Not Flooded	Not Flooded	-0.08	-0.08	High	Debris on the front door. Flooding was caused by lack of overland flow route.	46
19	3 Abuklea Road, Eastwood	64.24	0.14	64.33	0.28	0.09	-0.05	Not Flooded	Not Flooded	-0.14	-0.14	High	Debris on the front door	47
20	3A Abuklea Road, Eastwood	65.13	-	65.31	0.54	0.18	-	64.94	0.17	-0.19	-	High	Debris on the garden shed. Flooding was caused by lack of overland flow route	48
5b	Glen Reserve/ Sydney Water open channel, Eastwood.	66.65	-	67.09	0.34	0.44	-	66.44	0.07	-0.21	-	High	Reserve and Council's car park flooded. Refer photo for flood levels.	10
17b	1 Abuklea Road, Eastwood	62.99	0.35	62.96	0.19	-0.03	0.16	Not Flooded	Not Flooded	-0.35	-0.35	High	Debris on the rear fence and front gate brick wall. Flooding was caused by lack of overland flow route.	44, 45

Table 28 Comparison of TC Model Results with Observed Depths for the 2018 Event

ID	Location	Depth in Yard (mm)	Depth in Garden (mm)	Depth on Floor (mm)	Max Observed Depth (m)	Modelled Depth (m)	Depth Difference (m)	Quality	Description (provided by Council)
1	32 Lovell Road, Eastwood	300-400	90	0	0.4	0.32	-0.08	OK	The property is located in the medium risk zone of 1 in 100 yr overland flowpath. Surcharging and flooding occured during storm event last 28/11/18 around 6:30-7:00AM 2. Impermeable fencing across overland flow path contributed to flooding 3. Inlet pit (T1101960) grate not bolted/fitted and can be washed away. Refered to Pit Replacement Program
2	124 Rowe Street, Eastwood	0	0	50	0.05	0.05	0	OK	Flood level (~50mm) observed around 8:30-9:00am when the flood already subsided. Tenants noted that the flood First Avenue (Eastwood) is approximately 200-300mm high
3	122 Rowe Street, Eastwood	0	0	50	0.05	0.06	0.01	OK	Flood level (~50mm) observed around 8:30-9:00am when the flood already subsided.
4	120 Rowe Street, Eastwood	0	0	0	0	0.09	0.09	OK	Shop owner noted that they are not flooded. Same owner for 118 and 120 Rowe St
5	118 Rowe Street, Eastwood	0	0	0	0	0.03	0.03	OK	Shop owner noted that they are not flooded. Same owner for 118 and 120 Rowe St
6	114-116 Rowe Street, Eastwood	0	0	50	0.05	0.16	0.11	OK	Flood level (~50mm) observed around 8:30-9:00am when the flood already subsided.
7	112 Rowe Street, Eastwood	0	0	10-20	0.02	0.24	0.22	Low	Flood level (~10-20mm) observed around 12:00nn when the flood already subsided.
8	100-104 Rowe Street, Eastwood	500-600	500-600	250	0.6	0.61	0.01	Low	Council Property. Flood levels observed around the peak time. Drainage pit at the rear of the property was blocked with dirt/sediment. Watermarks show 500-600mm flood depth at the yard/garage, and ~250mm inside the store
9	102 Rowe Street, Eastwood	500-600	500-600	0	0.6	0.77	0.17	Low	Flood levels observed during the peak time (6:00am) at the year is around 500-600mm. Flood depth inside the store around 9:30am is approx 50mm
10	100 Rowe Street, Eastwood	600-800	600-800	0	0.8	0.63	-0.17	OK	Water marks at yard / rear toilet show approx 600-800mm flood depth. Noted damages to electronics and appliances (fridge). Shop owner noted the flood fully subsided around 9:00am
11	98 Rowe Street, Eastwood	200-300	200-300	50-100	0.3	0.61	0.31	OK	Stormwater pits and pipes located at the rear of the property are blocked by dirt. Water depth at yard around 200-300mm and 50-100mm inside the store. Noted that the another property's fence located at rear of the site was damaged and caused the dirt to be washed to the drainage pit/pipes. Damages to electronics, phone lines are noted in the investigation.
12	96 Rowe Street, Eastwood	500-600	500-600	180-200	0.6	0.56	-0.04	OK	Flood depth is approx above the car tire at the garage/yard. Some electrical damage noted in the investigation
13	94 Rowe Street, Eastwood	300	300	0	0.3	0.32	0.02	OK	Tenants arrived in the store around 10:00am and noted that the flood already subsided. Flood depth based on water marks at the yard is approx 300mm, while the store front is not affected. Damages to food ingredients noted by the owner
14	92 Rowe Street, Eastwood	200	200	0	0.2	0.13	-0.07	OK	Flood depth based on water marks at the yard is approx 200mm, while the store front is not affected. No electrical damage noted by the owner/tenant
15	86 Rowe Street, Eastwood	0	0	0	0	Not Flooded	0	OK	Not flood affected. Damage to medicine supply due to roof leak
16	82 Rowe Street, Eastwood	0	0	0	0	Not Flooded	0	OK	
17	9 Vanimo Place, Eastwood	0	0	0	0	1.19	N.A.	OK	Customer advised that again Terry's Creek overflowed on Wednesday and that of the three houses affected, his was the most impacted. Mr Snelling had the water through both the garage and house itself
18	First Avenue, Eastwood	0	0	200-300	0.3	0.26	-0.04	High	Tenants noted that the flood First Avenue (Eastwood) is approximately 200-300mm high

6.2.2. Macquarie Park (MQ) Model

For the 1984 event, a comparison with historical levels is shown in Table 29. The historical photos are provided in Appendix B. The flood marks and photos were obtained from the previous 2010 Macquarie Park Flood Study (Reference 6). The historical levels were estimated based on the hand drawn debris lines on the post-event photos, which were not surveyed. The flood marks were taken mostly upstream or downstream of the major roads along the Shrimptons Creek. Modelling matched three of these levels well near Epping Road and Kent Road, while modelled levels at upstream of Bridge Road and Lucinda Road were significantly higher than the recorded levels.

The modelling of these areas was reviewed and the sensitivity of various modelling configurations and parameters was tested to see if the model calibration could be improved. The tests included refining the creek topography using previous model cross-sections, blockage at cross-drainage structures, and a range of Mannings 'n' roughness values to reflect possible variation in the creek vegetation over time. The model results were found to be relatively insensitive to these changes. It is possible there have been other alterations to the creek or overbank topography and localised features that have altered the flood behaviour over the last thirty years. It is also possible that the data points are of low quality – the debris lines were noted as "indistinct."

For the 1990 event, only estimated depths were available. The comparison table of modelled flood depths with 55 estimated depths is included in Appendix C. Modelling results for approximately 75% of the marks were within the 'fair fit' criteria. The recorded rainfall is likely to the primary source of uncertainties as the nearest gauge was located approximately 5 km away from the Shrimptons Creek catchment, given the presumption that rainfall data at Ryde Pumping Station was copied from the West Epping Bowling Club Station.

6.2.3. Buffalo and Kittys Creeks (BK) Model

The calibration process for the BK Model was the most limited compared to the other models due to the lack of reliable data. For the selected 1990 event, a total of 41 flood observations with approximate flood depths were compared to the TUFLOW model results. Approximately 60% of them were within the fair fit range. The full table of comparisons is included in Appendix C. The primary source of uncertainty is the rainfall data, as mentioned in the same event for the MQ model calibration. Most of the data points are related to shallow overland flooding, which can also be unreliable since the behaviour can be significantly affected by localised small topographic features. 7 of the observed flood depths were less than 0.1 m.

Table 29: Comparison of MQ Model Results with Historical Levels for the 1984 Event

ID	Location	Recorded Level (m)	Observed Depth (m)	Modelled Level (m)	Modelled Depth (m)	Level Difference (m)	Depth Difference (m)	Quality	Description (from Reference 6)	Photo No	Comment
FS_1	Debris under Fullers Bridge deck	4.6		5.52	8.32	0.92		Low	Debris on top of bridge column, say 200-300mm deep. Top of bridge column, RL 4.3m AHD (from original bridge plan)	L1	Verification not possible since debris cannot lodge on bridge between RL 4.3 and 5.3m AHD.
FS_5	Debris line on creek bank just upstream of Epping Road	48.2		48.12	3.24	-0.08		Low	Culvert obvert (as surveyed) is RL 45.68-45.77m AHD. Debris line level appears to be similar to adjacent bridge culvert obvert. But debris also observed over the bridge guard rails (1m height estimated)	S16	Location estimated roughly
FS_6	Debris at Kent Road power pole	50.32	0.3	50.32	0.21	0.0	-0.09	Good	Ground level of 50.02m AHD; Debris is trapped against based of pole say 300mm deep	S23	Good fit
FS_7	Debris at Kent Road southern handrail	50.95	0.7	50.95	0.72	0.0	0.02	Good	Ground level of 50.25m AHD. Debris is caught against handrail post, top of debris say about 700mm above ground level	S24	Good fit
FS_8	(Indistinct) debris line at upstream face of Lucinda Road Footbridge	53.05		53.88	1.26	0.83		Low	As surveyed, underside of beam varies between RL 52.95 & 53.05m AHD. Hand drawn HWL line in photo appears to be similar to mid depth of footbridge structural beam	S26	Debris mark not clear
FS_11	(Indistinct) water line at upstream side of Bridge Road culvert	56.25		57.44	2.91	1.19		Low	As surveyed, culvert obvert is RL 56.55m AHD. Hand drawn HWL line in photo appears to be about 300mm under bridge obvert	S31	Debris mark not clear

6.2.4. Parramatta River Catchments (PR) Model

For the 1984 event, historical photos and the model comparison against 39 observed flood depths are included in Appendix B. The observed data was a combination of datasets from Council and the previous flood studies. The available data points were mostly collected within the Archers Creek Catchment, Denistone Catchment and Charity Creek Catchment. All depths were considered within the fair fit range. It was noted that reported depths at shops in relation to frontages to Graf Avenue in the West Ryde Shopping Centre were not consistent, ranging from 0.3 m to 2 m. The approximate locations of them were identified where good fits between the modelled depths and observed depths were achieved generally.

For the 1990 event, a comparison with a total of 38 observed flood depth is included in Appendix C. Model results at approximately 87% of the observations were within the fair fit range. Those outside the fair range were generally lower than the observed/estimated depths. As considered for the other models in the same event, the rainfall depths are likely to be the primary source of uncertainty and error. For a particular location at 3 Henry Street, Ryde, the modelled depth was likely lower due to the construction of detention basins at the Royal Rehabilitation Centre as well as the development of the Centre itself.

6.3. Summary of Calibration Results

The model fits reasonably well to the recorded data in the study area overall. The consolidated TUFLOW models are considered to be suitable for the estimation of design flood behaviour in the study area. The calibration exercise was subject to limitations which are typical for smaller urbanised catchments such as those in the study area:

- Sparsely positioned rainfall gauges which are often unable to adequately describe the spatial and temporal pattern of rainfall within the catchment, especially for localised convective storm bursts.
- Uncertainty regarding the original source and accuracy of much of the historical recorded flood level data.
- Uncertainty regarding the exact time of record and location of the historical flood marks and of the flood observations from previous questionnaires. For example, a post-event flood debris mark left during the flood recession might be at a lower level than the peak while it could be higher due to the turbulence or other dynamics. For an event that peaked during the night, for instance in the February 2010 calibration event, the accuracy of the flood marks can be compromised.
- A recent event in November 2018 was modelled for additional validation. For this event only observed depths were available, provided less reliable information than the surveyed flood marks from other events. The data was collected only around one flood problem area within the Eastwood and Terrys Creek Catchment. Validation using additional events for models in the other catchments is not feasible due to the lack of flood data in more recent events.

7. DESIGN EVENT MODELLING

7.1. Updates to Australian Rainfall and Runoff

Design flood modelling for this study was undertaken in accordance with the guidance for rainfall-runoff flood estimation techniques in the updated edition of Australian Rainfall and Runoff 2019 (ARR19, Reference 2). Since the last major edition of ARR was published in 1987 (ARR87), numerous technological developments and a larger set of recorded rainfall data has been available for updating the guidelines on design rainfall depths and temporal patterns. This set of data includes a larger number of rainfall gauges which continuously record rainfall (pluviometers) and a longer record of storms (inclusion of events from approximately 1985 to 2015). Prior to this, the sub-daily rainfall records in many locations apart from capital city centres covered only a ten to fifteen year period from the 1970s.

Compared to ARR87, there are three major updates to the rainfall-runoff design flood method as follows:

1. The Intensity-Frequency-Duration (IFD) design rainfall data and the initial and continuing loss values across Australia have been updated using the additional 30 years of data;
2. There is information about the amount of rainfall likely to occur before the main storm burst and how to incorporate this into model estimates;
3. The approach for assuming design temporal patterns and determining the critical duration has been significantly revised. ARR19 recommends that 10 temporal patterns should be analysed for each storm duration in order to determine the critical storm event. The critical storm event is not the event producing the maximum peak value for all the durations but the temporal pattern of the duration which produces the maximum average peak value from the 10 storms.

IFD rainfall data, initial loss and continual loss values were obtained from the ARR Datahub[4]. A copy of the Datahub data for each of the catchment areas (Eastwood & Terrys Creek, Macquarie Park, Buffalo & Kittys Creek and Parramatta River Ryde Subcatchments) is provided in Attachment A.

7.2. IFD Design Rainfall Data

Council's stormwater pit and pipe network systems and building floor levels are designed by engineers to meet a certain risk level. These risk levels are known as "design" standards and are quantified by the likelihood of occurrence in a given year (AEP, see Appendix A for discussion). Typical standards include the 20% AEP or 10% AEP design standard for pipe drainage and the 1% AEP standard for floor levels. Floor level requirements usually incorporating an additional freeboard or safety factor of an additional 0.5 m (see City of Ryde Development Control Plan 2014 for more details regarding freeboards – Reference 18).

Design flood levels corresponding to a given standard are often estimated by assuming that

4 http://data.arr-software.org/

rainfall of a given AEP will produce a flood of a similar AEP. This assumption was adopted for this study. It is an implicit assumption in the rainfall-runoff modelling technique that was used. The "design" standard rainfall data from the Bureau of Meteorology is used as an input to the hydrologic and hydraulic models to determine flood levels and extents.

A key consideration in this process is the intensity and duration of the design rainfall. On a larger river catchment like the Parramatta River or Lane Cove River, the storms that produce flooding will generally be of longer duration (several hours). For the smaller catchments which are the focus of this study, the durations that will cause the most intense local flooding are the short duration events of approximately 2 hours or less. This question of the critical storm duration is discussed further in Section 7.3.

Design rainfall data is expressed in terms of its Intensity, Frequency and Duration (IFD). The design IFD rainfall data was obtained from the BoM website[5] for each model area, using the centroid for each area as the value across that area. Data for the centroid of the Ryde LGA is provided in Table 30 as an indication of the adopted values. The design rainfall data varies for each model area, although is generally quite consistent across the Ryde LGA and does not vary significantly within each of the study area catchments.

Table 30: Design Intensity Frequency Duration Rainfall Data in mm (Ryde LGA centroid)

Duration (minutes)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
1	2.57	3.41	3.98	4.54	5.3	5.89
2	4.18	5.39	6.23	7.07	8.26	9.18
3	5.83	7.55	8.75	9.95	11.6	12.9
4	7.35	9.59	11.2	12.7	14.8	16.5
5	8.72	11.5	13.3	15.2	17.8	19.8
10	13.9	18.5	21.7	24.8	28.9	32.1
15	17.4	23.2	27.1	31	36.2	40.1
30	20	26.5	31.1	35.5	41.4	45.9
45	23.6	31.3	36.5	41.7	48.6	53.9
60	27.5	36.1	42	47.9	55.9	62.1
90	30.3	39.6	46.1	52.6	61.4	68.3
120	34.8	45.2	52.6	60	70.2	78.3
180	38.4	49.8	58	66.2	77.6	86.8
360	44.4	57.7	67.3	77.1	90.8	102

7.3. Critical Duration

The adoption of ARR19 has made a significant difference in critical duration analysis (the storm duration which produces the highest flood level at a given catchment location). Each AEP event may have a unique critical duration and critical storm on each catchment. The critical duration may vary throughout the catchment, with longer durations generally causing more severe flooding lower down in the catchment compared to the upper, as the total contributing catchment area size increases. The details of the critical analysis are provided below.

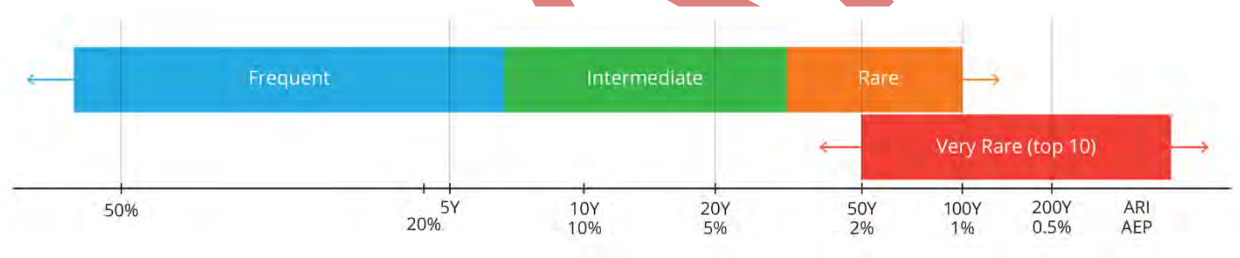
5 http://www.bom.gov.au/water/designRainfalls/revised-ifd/

7.3.1. Temporal Patterns

Temporal patterns are a hydrologic tool that describe how rain falls over time and are used in hydrograph estimation. There are significant updates in the application of temporal patterns for design events in ARR19. Previously, with ARR87 guidelines a single temporal pattern was adopted for each rainfall event duration. However, ARR19 discusses the potential inaccuracies with adopting a single temporal pattern and recommends an approach where an ensemble of different temporal patterns is investigated. It is widely accepted that there are a large variety of temporal patterns possible for rainfall events of similar magnitude. This variation in temporal pattern can result in significant effects on the estimated peak flow. As such, the revised temporal patterns have adopted an ensemble of ten different temporal patterns for a particular design rainfall event. The rainfall-runoff response can be quite catchment specific, and using an ensemble of temporal patterns attempts to produce the probability-neutral catchment response so that the AEP of the estimated peak flood levels is consistent with the AEP of the rainfall.

ARR19 provides 30 patterns for each duration and are sub-divided into three temporal pattern bins based on the frequency of the events. Diagram 14 shows the three categories of bins (frequent, intermediate and rare) and corresponding AEP groups. The very rare bin is currently experimental and not used in this flood study. There are ten temporal patterns for each AEP/duration in ARR19.

Diagram 14: Temporal Pattern Bins



The adopted temporal pattern out of the 10 is the pattern which produces the peak flow (or peak flood level) just greater than the average of the 10 peak flows (or levels). Thus, the temporal pattern adopted does not produce the largest peak flow (or level) for that storm duration. The critical storm duration is that which produces the maximum average peak flow (or level).

7.3.2. Representative Storm Burst Selection

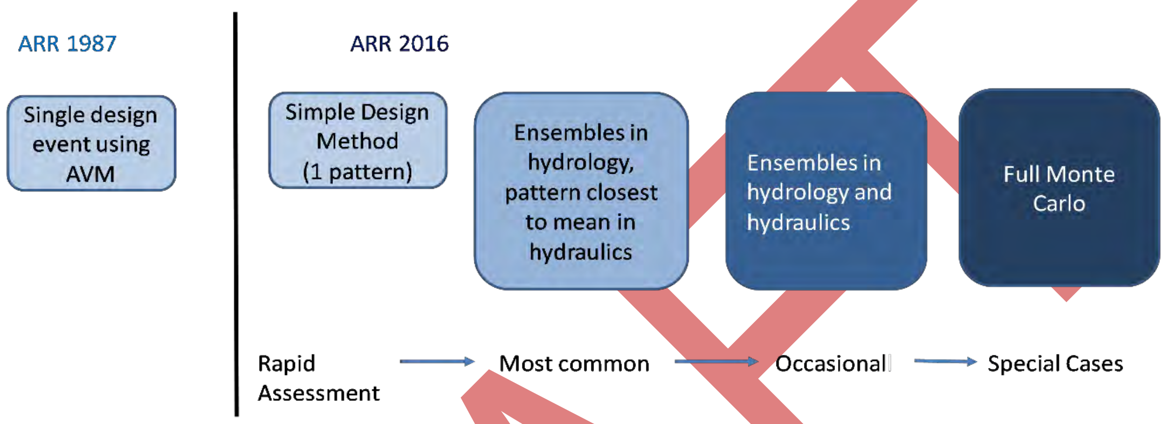
The critical duration is the temporal pattern and duration that best represents the flood behaviour (e.g. flow, level) for a specific design magnitude. It is generally related to the catchment size, as flow takes longer to concentrate at the outlet from a larger catchment, as well as other considerations like land use, shape, stream characteristics, etc.

With ARR2019 methodology, the critical duration is the storm duration that produces the highest mean flow or level at a point of interest (where the mean is calculated from the ensemble of ten temporal patterns for that duration). Where there are multiple locations of interest with different contributing catchment sizes, there can be multiple critical durations that need to be considered.

Once the critical duration is established, it is usually desirable to select a representative design storm temporal pattern that reproduces this behaviour for all points of interest. This representative storm can then be used for determining design flood behaviour and for future modelling to inform floodplain management decisions.

The potential methods for the ensemble modelling approach are outlined in Reference 19, reproduced in Diagram 15.

Diagram 15: Ensemble Hydrology Approaches in ARR2019



The "Most common" approach is to rely on a hydrologic model to determine the critical duration before proceeding with hydraulic modelling. For this study, due to the complex interactions between the hydrology and hydraulics, the relatively more complex "Occasional" approach was used where the full ensemble of temporal patterns were run in both the hydrologic and hydraulic models for a range of durations from 30 minutes to 720 minutes.

For each duration, a grid of the mean peak level at each grid cell was calculated, and then a maximum grid was calculated taking the highest peak mean level for each grid cell. The source of the peak mean level for each grid cell was mapped to show the variation in critical duration across the catchment (see Figure 34).

The process above indicated that the 30 minute to 45 minute durations are critical for the majority of the catchments in the study area, apart from some flood storage areas in lower catchment areas and open spaces such as parks, playing fields. Adopting a range of critical duration events across a catchment complicates future analysis and use of the modelling tools, as this may mean adopting different critical durations to represent the peak level, velocity or hazard. It also means that when undertaking sensitivity analysis or the modelling of options a multitude of durations must be run, increasing costs and time to use the modelling. Thus, it is preferable to adopt a single representative storm that is similar to the critical duration behaviour for each AEP where possible.

Generally, there is very little variation in mean peak flood level (i.e. within ±0.05 m) between these different durations, and a representative pattern of either 30 minutes or 45 minutes was typically suitable for each model area. Figure 35 shows the variation between the adopted representative pattern in each model area and the "true" critical duration peak mean result.

The representative storm patterns selected for the design event modelling are summarised in Table 31. For the PMF, an 'envelope' approach was adopted, taking the maximum of several durations across the study area.

Table 31: Selected Representative Design Storm Temporal Patterns

Model Area	Frequent 50% and 20%	Intermediate 10% and 5%	Rare 2% and 1%	PMF
TC Model	45 min TP4547	45 min TP4478	45 min TP4525	45min, 90 min GSDM TP
MQ Model	45 min TP4550; Lane Cove River: 720 min TP4810	45 min TP4478; Lane Cove River: 720 min TP4794	45 min TP4362; Lane Cove River: 720 min TP4785	30 min, 60 min, 120 min GSDM TP
BK Model	45 min TP4552	45 min TP4478	45 min TP4496	15 min, 30 min, 45 min GSDM TP
PR Model	45 min TP4547	30 min TP4511	30 min TP4498	15 min 45 min 120 min GSDM TP

7.4. Rainfall Losses

The term "rainfall loss" refers to rain that falls but does not end up flowing across the catchment, either in pipes or as overland flow. The primary mechanism by which rainfall is "lost" and does not runoff in urban catchments is through infiltration into the ground. A small amount of rainfall is remains clinging to trees, buildings and other catchment features and eventually evaporates rather than contributing to runoff volumes.

Methods for modelling the proportion of rainfall that is "lost" to infiltration are outlined in ARR19 (Reference 2). The methods are of varying degrees of complexity, with the more complex options only suitable if sufficient data are available. The method most typically used for design flood estimation is to apply an initial and continuing loss to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

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

The adopted loss parameters are summarised in Table 32. These are generally consistent with the parameters adopted flood studies in similar catchments within the Sydney metropolitan area.

Table 32: Adopted rainfall loss Horton/ILSAX parameters

RAINFALL LOSSES	
Paved Area Depression Storage (Initial Loss)	1.0 mm
Grassed Area Depression Storage (Initial Loss)	5.0 mm
SOIL TYPE	3
Slow infiltration rates (may have layers that impede downward movement of water). This parameter, in conjunction with the AMC, determines the continuing loss	
ANTECEDENT MOISTURE CONDITONS (AMC)	3 (4 for extreme events)
Description	Rather wet
Total Rainfall Preceding the Storm Burst	12.5 to 25 mm

For the DRAINS models in the Eastwood and Terrys Creek catchment, while subcatchments within the City of Ryde were represented in the Horton/ILSAX model, subcatchments within the City of Parramatta Council and the Hornsby Shire Council were represented as RAFTS nodes. The rainfall losses in the RAFTS model were updated based on the ARR Data Hub, as shown in Table 33.

Table 33 Adopted rainfall loss RAFTS parameters

Rainfall Losses	2008 Eastwood and Terrys Creek FS	Harmonisation Study
Impervious Area Initial Loss (mm)	10	1
Impervious Area Continuing Loss (mm/h)	2.5	0
Pervious Area Initial Loss (mm)	10	Probability Neutral Burst Initial Loss according to the ARR Datahub
Pervious Area Continuing Loss (mm/h)	2.5	0.72

The impervious proportion of each sub-catchment were retained from the existing DRAINS models for each study area.

7.5. Debris Blockage

Design blockage for hydraulic structures was adopted in accordance with ARR19 (Reference 2). The debris availability, debris mobility and debris transportability were deemed to be in the Low to Medium categories for each of the catchments. The overall debris potential was classified as Low. With this classification, an inlet headwall blockage of 50% was applied to culvert structures in the model with an opening size less than 1.2 m, and 20% blockage to culverts with a larger opening size. For bridges with relatively large spans across the waterway, 5% blockage was applied, or 0% blockage for clear-spanning structures with no piers.

8. DESIGN FLOOD MODELLING RESULTS

8.1. Mapping of Results

The 50%, 20%, 10%, 5%, 2%, 1%, and PMF design flood events were simulated. Maps of various model results are provided in Appendix F as follows:

- Peak depths and level contours on Figure F1 to Figure F7;
- Peak velocities on Figure F8 to Figure F14;
- Hydraulic hazard categories on Figure F15 to Figure F21;
- Hydraulic categorisation (Flood Function) for the 1% AEP event on Figure F22;
- Flood risk precincts on Figure F23;
- Flow paths on Figure F24;
- Flood emergency response classification on Figure F25; and
- Tidal inundation extents on Figure F26 and Figure F27.

The flood maps (Figure F1 to Figure F23) were filtered using the following criteria:

- Removal of areas where flood depths were less than 100 mm, except where the velocity-depth product was greater than 0.2. This removes areas affected by very shallow sheet flow not considered to be 'flooding' but rather 'local drainage' under the Floodplain Development Manual (Reference 3). Areas where the velocity-depth product is greater than 0.2 were retained, as this is fast moving water (velocity greater than 2 m/s) and likely to be a flow path.
- Removal of isolated areas of flooding where the area of inundation is less than 100 m^2. This removes disconnected patches of flooding that may remain after the depth filter is applied and any artefacts that have resulted from localised depressions.

Additional flood results are presented in the following tables in Appendix G:

- Peak flood levels at the selected locations for each model from Table G1 to Table G4.
- Peak flood depths at the selected locations for each model from Table G5 to Table G8.
- Peak flows at the selected locations for each model from Table G9 to Table G16. For each model, overland flows and conduit flows are tabulated separately.

Figure 36 shows the reporting locations for each model domain.

It is also noted that while flood behaviour for the Lane Cove River has been defined, the Lane Cove River is a reasonably large river system that has complex hydrology and hydraulics. This study is not intended to produce detailed information for the Lane Cove River, however, the approach undertaken for modelling the Lane Cove River the Macquarie Park Flood Study (Reference 6) was adopted for this study. There are several limitations to the modelling that was undertaken, including:

- Hydrology for large catchments outside of the Ryde LGA modelled in DRAINS using ILSAX with simplified parameters.
- The Macquarie Park Flood Study (Reference 6) adopted Terrys Creek flows into the Lane Cove River from the DRAINS model. This has been updated for this study to source the flows from the TC TUFLOW model, which is considered more reliable than

the DRAINS model.

- Bathymetry data is not available for the Lane Cove River, and hence ground levels below the water surface are unknown. It is also assumed that the LiDAR data is generally less accurate in this area due to the dense vegetation and incised nature of the valley. The terrain has been estimated based on the available LiDAR data and 1D cross sections used in the Macquarie Park Flood Study (Reference 6) model.

The modelling assumptions adopted are considered to be reasonable given the limited flood affectation of the Ryde LGA due to the Lane Cove River.

8.2. Description of Flood Behaviour

The design flood behaviour across the 14 catchments within the City of Ryde LGA can be seen in the peak flood depths and water level contour maps (Figure F1 to Figure F7), peak velocity maps (Figure F8 to Figure F14) and tabulated peak levels, depths and flows in the selected locations (Table G1 to Table G16). The results are presented for the range of design flood events modelled from the 50% AEP event to the PMF event.

In the 50% AEP event, shallow overland flow (depths < 0.1 m) is generally contained within the gutters and dedicated drainage reserves while along some overland flow paths through properties, minor backyard flooding with depths typically in the range of 0.1 m to 0.3 m. In the major creeks, flow is typically contained within the channel. However, floodwater breaks out of the channel into the low-lying surroundings at some locations, for example Terrys Creek approaching the intersection of Hillview Lane and Lakeside Road, and Terrys Creek at Vanimo Place.

In the 20% AEP event, overland flow paths with depths greater than 0.1 m of water become more evident. Water within major natural creeks begin to flow onto the floodplain. This is extended further in the 10% and 5% AEP events. In the 2% and 1% AEP events, overbank flow areas become more extensive and convey a greater proportion of flow. Flood storage areas are more extensive and become deeper. For example, the sports field in the Eastwood Park to the west of West Parade becomes a large flood storage area, with flood depths of 0.3 m to 1 m. In the PMF event, new flow paths are activated and existing flow paths become wider and deeper. Overbank areas adjacent to the major creeks are generally inundated to depths greater than 1 m. Flooding is much more hazardous in the problematic areas.

The concrete channels in the upstream part of the Eastwood and Terrys Creek catchment are estimated to have capacity up to approximately a 20% AEP in most sections. Those concrete channels in the downstream areas of the PR model discharging to the Parramatta River have capacities of approximately 10% AEP to 5% AEP.

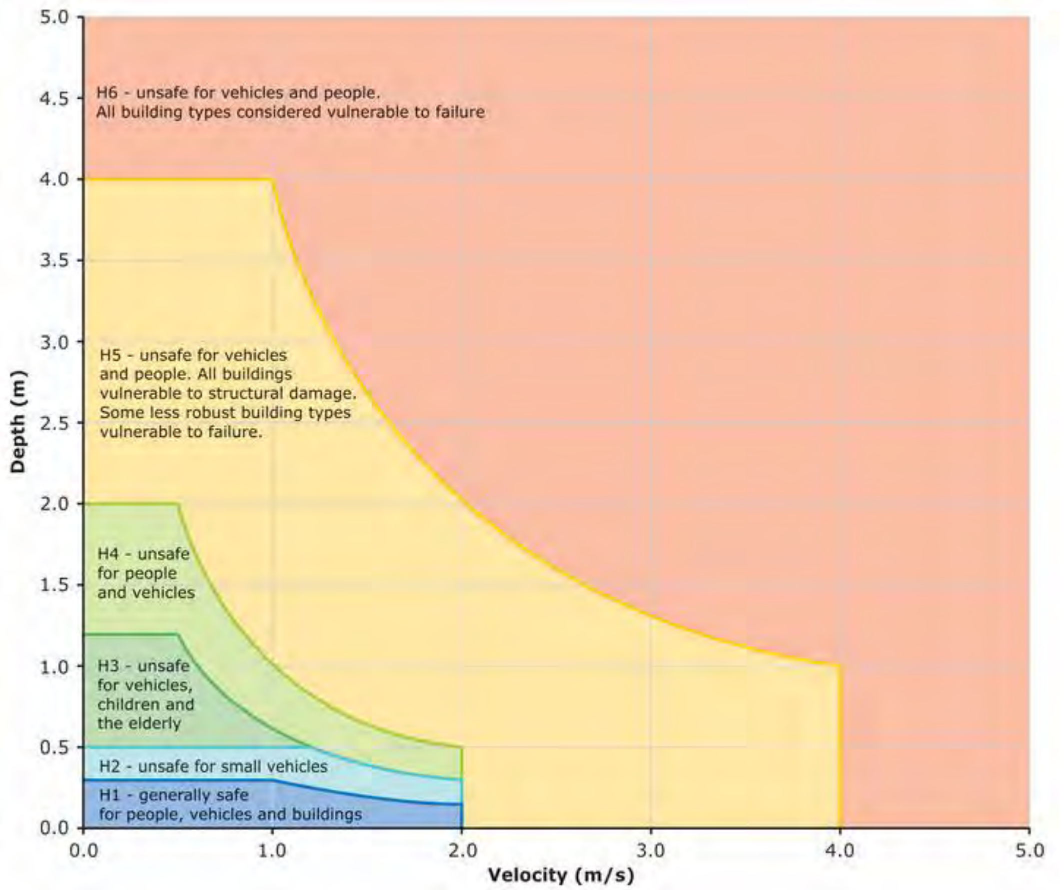
From the key road crossing locations where flood depths and levels have been tabulated (Appendix G), 14 road crossings would experience maximum depths of flooding that exceed 0.3 m in an event as frequent as 50% AEP, while approximately 50 crossings would be overtopped in the 1% AEP event. The depth of 0.3 m was selected as the threshold for defining the flood immunity of a road, as it is a typical depth threshold to indicate when a road is no longer trafficable for small vehicles, assuming velocity of flooding is not greater than 1 m/s. With higher velocities, this threshold reduces (see Section 8.3).

8.3. Flood Hazard

The risk to life and potential damages to buildings during floods varies both in time and place across the floodplain. In order to provide an understanding of the effects of a proposed development on flood behavior and the effects of flooding on development and people the floodplain can be sub-divided into hydraulic and hazard categories. This categorisation should not be used for the assessment of development proposals on an isolated basis, rather they should be used for assessing the suitability of future types of land use and development in the formulation of a floodplain risk management plan.

Hazard classification plays an important role in informing floodplain risk management in an area. Previously, hazard classifications were binary – either Low or High Hazard as described in the Floodplain Development Manual (FDM, Reference 3). However, in recent years there have been advances in the classification of hazard. *Managing the floodplain: a guide to best practice in flood risk management in Australia* (Reference 20), part of the Australian Disaster Resilience (ADR) Handbook Series, provides revised hazard classifications which add clarity to the hazard categories and what they mean in practice. The supporting guideline 7-3 (Reference 21) contains a more detailed distinction and practical application of hazard categories than the high/low classification method, identifying 6 classes of hazard. A summary of this categorisation is provided in Diagram 16.

Diagram 16: General flood hazard vulnerability curves (ADR)



- H1 – No constraints, generally safe for vehicles, people and buildings;
- H2 – Unsafe for small vehicles;
- H3 – Unsafe for all vehicles, children and the elderly;
- H4 – Unsafe for all people and all vehicles;
- H5 – Unsafe for all people and all vehicles. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure. Buildings require special engineering design and construction; and
- H6 – Unsafe for all people and all vehicles. All building types considered vulnerable to failure.

The velocity-depth criteria for the upper limit of the H3 category is roughly equivalent to the delineation between "High" and "Low" hazard using the classification from Figure L2 in the FDM (Reference 3). For the purposes of planning documents that refer to High/Low hazard, the classifications H3 and lower can be considered "Low", and the classifications H4 and above can be considered "High." The flood hazard for each of the modelled design flood events can be seen in Figure F15 to Figure F21.

8.4. Hydraulic Categorisation (Flood Function)

The 2005 NSW Government's Floodplain Development Manual (FDM, Reference 3) defines three hydraulic categories which can be applied to different areas of the floodplain depending on the flood function:

- Floodways;
- Flood Storage; and
- Flood Fringe.

Floodways are areas of the floodplain where a significant discharge of water occurs during flood events and by definition, if blocked would have a significant effect on flood levels and/or distribution of flood flow. Flood storages are important areas for the temporary storage of floodwaters and if filled would result in an increase in nearby flood levels and the peak discharge downstream may increase due to the loss of flood attenuation. The remainder of the floodplain is defined as flood fringe.

There is no quantitative definition of these three categories or accepted approach to differentiate between the various classifications. The delineation of these areas is somewhat subjective based on knowledge of an area and flood behaviour, hydraulic modelling and previous experience in categorising flood function. A number of approaches are available, such as the method defined by Howells *et al* (Reference 22) that has been adapted for use in numerous studies across the Sydney metropolitan area.

For this study, the Howells *et al* (Reference 22) method was applied. Since every floodplain is different and exhibits different flood behaviour, threshold values cannot be translated from one location to another. The threshold values adopted for the Ryde LGA were derived from inspection of flood grids (depth, velocity and velocity-depth product) to understand flood function across the catchments and testing of various combinations of values, with the aim of obtaining a flood function map that:

- Identifies key flow paths and the in-bank section of creeks as floodways, and
- Identifies floodways as relatively continuous areas of significant conveyance, and
- Identifies flood storage areas as relatively slow-moving water where a significant volume of water is stored, and
- Identifies shallow overland flows and peripheral areas of flooding as flood fringe.

The hydraulic categories were defined by the following criteria, which is considered to be a reasonable representation of the flood function for the Ryde LGA:

- Floodway is defined as areas where:
    - the peak value of velocity multiplied by depth (V x D) > 0.25 m^2/s, **AND** peak velocity > 0.25 m/s, **OR**
    - peak velocity > 1.0 m/s **AND** peak depth > 0.1 m.

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

- Flood Storage comprises areas outside the Floodway where peak depth > 0.2 m, and
- Flood Fringe comprises areas outside the Floodway where peak depth ≤ 0.2 m.

The hydraulic categorisation was undertaken for the 1% AEP event is shown in Figure F22.

8.5. Flood Risk Precincts

Flood risk precincts consider the probabilities and consequences of flooding over the full spectrum of flood frequencies that might occur at a site. Each flood risk precinct (low, medium and high) has different flood planning controls that apply (for example, minimum floor levels, suitable building components, flood affectation, emergency management, etc.) depending on the type of development. They were defined and adopted in three of the previous City of Ryde flood studies (Reference 5, 6, and 8). The City of Ryde Development Control Plan (DCP, Reference 23) defines these precincts broadly, without any prescriptive criteria. In this study, the flood risk precincts are defined as follows:

- High Flood Risk Precinct: includes 1% AEP high hazard area, defined as H4 and above (hazards that are unsafe for all people and vehicles). These are areas where high flood damages, potential risk to life, or evacuation difficulties are anticipated, or development would significantly and adversely affect flood behaviour. Most development is restricted within these areas.
- Medium Flood Risk Precinct: includes the remaining area within the 1% AEP flood extent that is not defined as the High Flood Risk Precinct, with filtering applied (see Section 8.1). These are areas where there is still a significant risk of flood damage or safety concerns, but where this risk can be minimised by the application of appropriate development controls.
- Low Flood Risk Precinct: includes the remaining area within the PMF extent that is not identified as either High Flood Risk or Medium Flood Risk Precinct, with filtering applied (see Section 8.1). The risk of flood damage in these areas is low and most land uses would be permitted.

The flood risk precinct map applying the above definitions is shown in Figure F23.

8.6. Overland Flow

Overland flow precincts were defined and adopted in two of the previous City of Ryde flood studies (Reference 5 and 6). This comprised areas of shallow inundation distant from major watercourses. Areas classified as overland flow precinct typically have 1% AEP flood depths greater than 0.1 m to 0.2 m, but less than 0.3 m to 0.5 m. The overland flow precinct essentially aims to delineate the medium flood risk precinct (the 1% AEP low risk areas) into shallow upstream overland flows (overland flow precinct) and deeper downstream flows where a defined flow path has formed (medium flood risk precinct). It is often difficult to provide a location where a flow path transitions from being 'overland flow' to 'mainstream flow'. This delineation was undertaken in a subjective manner in the previous studies (Reference 5 and 6), using flood depths as a guide.

It is noted that there would also be areas beyond the overland flow precinct, where inundation from shallow depths (typically less than 0.1 m) would occur and whilst they do not present a great risk to development, they must meet the minimum development control requirements to ensure there is adequate protection from any stormwater inundation.

The City of Ryde DCP (Reference 23), whilst defining the Overland Flow Precinct, does not provide many specific controls or concessions related to this precinct. The DCP, instead, heavily refers to 'overland flows' in general. Since much of the urban areas of the Ryde LGA are subject to overland flows, this study retains the medium flood risk precinct as the 1% AEP low hazard filtered extent (as defined in Section 8.5), without providing a separate category for the overland flow precinct. The flood tagging process (Section 8.9) should identify those properties impacted by the 1% AEP event (rather than affected by shallow overland flows).

Instead of the overland flow precinct, a map has been produced defining all flow paths across the LGA (both mainstream and overland). This was produced for the 1% AEP event and is shown in Figure F24. This is the modelled 1% AEP extent without any filtering applied and the flow paths mapped are expected to form such as event. While flow paths outside a high or medium flood risk precinct would not present a significant flood risk, they should be considered for any development that encroaches on them.

8.7. Flood Emergency Response Planning

8.7.1. Road Inundation

The overtopping depths for key access roads in the study area are summarised in Table G5 to Table G8. The locations are indicated in Figure 36. The AEP when the overtopping depth reaches hazardous levels of 0.3 m or greater (i.e., H2 hazard or greater) is when the road can be considered completely cut to vehicles. Road flood immunity is determined based on those tables and shown on the Flood Emergency Response Classification map in Figure F25. It is also noted that while flood depths less than 0.3 m are considered trafficable for this study, it is advised to never drive through floodwater of any depth.

8.7.2. Classification of Communities

To assist in the planning and implementation of response strategies, the NSW State Emergency Service (SES) in conjunction with the NSW Office of Environment and Heritage (OEH, now Department of Planning and Environment) has developed guidelines to classify communities according to the impact that flooding has upon them. These Emergency Response Planning (ERP) classifications (based on guidance in Reference 28 and Reference 29) consider flood affected communities as those in which the normal functioning of services is altered, either directly or indirectly, because a flood results in the need for external assistance. This impact relates directly to the operational issues of evacuation, resupply and rescue, which is coordinated by the SES. Reference 29 recommends classification according to the criteria in Table 34.

Table 34: Emergency Response Planning Classification of Communities

Primary classification	Description	Secondary classification	Description	Tertiary classification	Description	Example figures
Flooded (F)	The area is flooded in the PMF	Isolated (I)	Areas that are isolated from community evacuation facilities (located on flood-free land) by floodwater and/or impassable terrain as waters rise during a flood event up to and including the PMF. These areas are likely to lose electricity, gas, water, sewerage and telecommunications during a flood.	Submerged (FIS)	Where all the land in the isolated area will be fully submerged in a PMF after becoming isolated.	Figure 1 Figure 7 with ring levee Figure 8 with impassable terrain
				Elevated (FIE)	Where there is a substantial amount of land in isolated areas elevated above the PMF.	Figure 2
		Exit Route (E)	Areas that are not isolated in the PMF and have an exit route to community evacuation facilities (located on flood-free land).	Overland Escape (FEO)	Evacuation from the area relies upon overland escape routes that rise out of the floodplain.	Figure 3
				Rising Road (FER)	Evacuation routes from the area follow roads that rise out of the floodplain.	Figure 4, Figure 6 with levee
Not Flooded (N)	The area is not flooded in the PMF			Indirect Consequence (NIC)	Areas that are not flooded but may lose electricity, gas, water, sewerage, telecommunications and transport links due to flooding.	Figure 5
				Flood Free	Areas that are not flood affected and are not affected by indirect consequences of flooding.	

Notes:

1. Classifications are based upon the probable maximum flood (PMF) or a similar extreme flood, if the PMF is not available. Where classifications are being retrofitted to areas covered by existing studies and the PMF or a similar extreme flood is not available, and a decision is made to not estimate or approximate an extreme event, classifications should be clearly indicated as ‘Preliminary based upon the largest flood available’.
2. Isolated areas may also be known as:
   - flood islands, where areas are isolated solely by flood waters. Where flood islands are completely submerged in the PMF, these may be called low-flood islands. Where flood islands have elevated areas above the PMF, they may be called high-flood islands.
   - trapped perimeter areas, where areas are isolated by a combination of floodwaters and impassable terrain. Where trapped perimeter areas are completely submerged in the PMF, these may be called low-trapped perimeter areas. Where trapped perimeter areas have elevated areas above the PMF, they may be called high-trapped perimeter areas.

Key considerations for flood emergency response planning include:

- Cutting of external access isolating an area;
- Key internal roads being cut;
- Transport infrastructure being shut down or unable to operate at maximum efficiency;
- Flooding of any key response infrastructure such as hospitals, evacuation centres, emergency service sites;
- Risk of flooding to key public utilities such as gas, electricity and sewerage; and

- The extent of the area flooded and the duration of inundation.

Flood liable land within the study area where there are habitable areas (identified as buildings on the aerial imagery) have been classified according to the ERP classification above. When classifying communities, consideration was given to flood depths for the purpose of being able to move through floodwaters on foot or in a vehicle, drawing on hazards presented in the Australian Disaster Resilience Handbook Collection (Reference 21). It is noted that the guidelines are generally more applicable to riverine flooding where significant flood warning time is available and emergency response action can be taken prior to the flood, or where long-term isolation may occur requiring possible resupply or medical evacuation. The critical storm duration is 45 minutes for the catchment in the 1% AEP event, and flash flooding from local catchment and overland flow will generally occur as a direct response to intense rainfall without significant warning. For most flood affected properties in the catchment, remaining inside the home or building is likely to present less risk to life than attempting to drive or wade through floodwaters, as flow velocities and depths are likely to be greater in the roadway. This issue of flood isolation is unlikely to be significant as access would be cut for a very short period of time.

The ERP classification of communities for the study area are shown in Figure F25 for the 1% AEP event. These figures also show major access roads to each of the relevant areas, as well as the AEP when the road is cut by more than 0.3 m of flooding over the road. The following classifications are identified and shown on the map:

- FIE zones: Flooded-Isolated-Elevated
  Where a property is unaffected by above floor flooding, or an area contains enough land elevated above the flood level to contain residents of that area, but nearby streets and access routes are flooded and cut off. Vehicular access from the area may be blocked, causing inconvenience or potentially life-threatening circumstances if emergency medical care is required during a flood.
- FIS zones: Flooded-Isolated-Submerged
  Where properties are inundated and potential evacuation routes are unavailable at the peak of the flood. There are several areas under this classification. Key areas include Eastwood town centre shops at Progress Avenue upstream of Hillview Road, Morshead Street properties upstream of Epping Road, and the area around Speedway West Ryde upstream of Victoria Road.
- FER/FEO zones: Flooded-Escape Routes-Overland/Rising Road
  The remaining area of the LGA that experiences flooding is classified FER or FEO zones, which are not highlighted on the map for readability. These areas, if evacuation or emergency access is required, will still have all-weather uninterrupted rising road access (up to H1 hazard) or overland escape routes (up to H2 hazard) that can be accessed on foot.
- NIC: Not Flooded-Indirect Consequences
  The remaining area of the LGA, while not directly affected by flooding, is assumed to have indirect consequences, which are not highlighted on the map for readability. These areas may be indirectly affected, such as interruption to utility supply (water, sewerage, gas, electricity or telecommunications) and cut transport links.

8.8. Tidal Inundation Analysis

Parts of the Ryde LGA along the Parramatta River and lower Lane Cove River are subject to tidal water level variation. The extents of tidal inundation are mapped as follows, using the tidal levels for Port Jackson from Section 3.10:

- Mean High Water Springs (MHWS, tidal level 0.65 mAHD) on Figure F26, and
- High High Water Solstice Springs (HHWSS, tidal level 1.0 mAHD) on Figure F27.

The maps also indicate the extents for these tidal planes under scenarios with 0.4 m and 0.9 m sea level rise. The Lane Cove River is currently tidal up to the Lane Cove Weir, near the Delhi Road crossing (Fullers Bridge). It is assumed that the tidal limit would not extend beyond this for tide levels below 1.5 mAHD (the estimated crest level of the weir). For tidal levels above this (i.e. for the 0.9 m sea level rise scenarios), it is expected that the weir would be drowned out and that the tidal limit would extend upstream. Since bathymetric data is not available, the extent of tidal inundation upstream of the weir within the Lane Cove River channel is unknown, however, has been estimated to be up to the Lane Cove Road crossing (De Burghs Bridge) for the purposes of this assessment.

8.9. Flood Planning Area

WMAwater undertook a Flood Planning Area (FPA) assessment of the Ryde LGA using the previous Flood Study results as a supplementary part of this study. The methodology and outcomes of that assessment were documented in a separate report (Reference 24). This FPA assessment was then updated using the updated ARR19 model results from this study, using the same methodology. Key components of the background information and methodology for the FPA assessment from Reference 24 are reproduced below.

8.9.1. Background

Local Government has various floodplain management responsibilities under the NSW Flood Prone Land Policy. One of these responsibilities is to ensure that development is commensurate with flood risk. This is generally managed by the application of development controls to flood prone land through a Local Environment Plan (LEP) and Development Control Plan (DCP). Enforcement of these controls requires that Council understands the nature of flood risk within the Local Government Area (LGA), and identifies the land where such development controls are applicable, which is referred to as the Flood Planning Area (FPA). This land is generally subject to notification through Section 10.7 planning certificates under the NSW Environmental Planning and Assessment Act. This notification is referred to as "flood tagging" in this report.

Since the completion of the previous catchment studies in the Ryde LGA, there have been updates to legislation and planning guidelines outlined in Planning Circular PS 21-006 issued by the NSW Department of Planning, Industry and Environment on 14 July 2021 (DPIE, now Department of Planning and Environment, Reference 25). The circular provides information about changes to Clause 7A of Schedule 4 of *Environmental Planning and Assessment Regulation 2000* (the Regulation), contained in the *Environmental Planning and Assessment Amendment (Flood Planning) Regulation 2021* (the Amendment).

The updated legislation does not change the primary mechanism by which flooding is considered as part of land-use planning in NSW. The previous legislation also required identification of lots on planning certificates (known as Section 149 certificates before being changed to Section 10.7 certificates), but was more rigid in the description of the Flood Planning Area (FPA). This rigidity of the FPA definition in previous versions of the standard LEP instrument led to inconsistency with several elements of the Floodplain Development Manual (for example the application of varying freeboard rather than a single 0.5 m freeboard for all land use controls). The previous legislation also did not allow for the application of development controls for flood prone land beyond the 1% AEP extent (including freeboard), except via special planning provisions requiring submission to the Department of Planning and Environment (DPE). This also was inconsistent with the principles of the Floodplain Development Manual, which requires Council to consider and manage for the full range of flood risk, including extreme events with a probability less than (rarer than) 1% AEP. The primary changes resulting from the new legislation are:

- An altered definition of the Flood Planning Area, to be consistent with that in the Floodplain Development Manual. Properties subject to flood-related development controls within the FPA require notification on Section 10.7 certificates under Clause 7A(1) of the Regulation, and
- An additional clause allowing the application of flood-related development controls to land between the FPA and PMF extents, for hazardous or sensitive uses, or situations where there is a particular risk to life or flood-related evacuation consideration. Properties subject to these controls require notification on Section 10.7 certificates under Clause 7A(2) of the Regulation.

8.9.2. Technical Limitations for Defining a Flood Planning Area

The definition of the FPA usually includes consideration of relevant freeboard. For example, for residential land, the land up to 0.5 m above the edge of the 1% AEP flood extent would be included within the FPA. This 0.5 m freeboard can vary for different land-uses or development types. The methodology for determining the Flood Planning Area must therefore take freeboard into consideration.

Automated procedures exist for adding freeboard to modelled design flood levels, then “stretching” this surface to identify additional land that is above the estimated flood level, but below the level of freeboard. However there are several circumstances where these approaches do not work, particularly steeper areas of overland flow which are common to urbanised environments where the land adjacent to the flow path frequently does not rise significantly above the flow surface. Adding freeboard and stretching leads to erroneous identification of areas that are not flood prone, even in the PMF. The technical reasons for this are explained in Reference 26.

Furthermore, the DCP specifies different freeboard allowances for different development types and flow regimes. This means that for a given property, the definition of the Flood Planning Area can vary depending on the type of development proposed for the site.

As a result of the above, it is not feasible to define a single line that reliably delineates the boundary of the FPA. However it is feasible to review the flood modelling information and determine a “yes” or “no” answer to whether the flood risk within or adjacent to a given lot is sufficient to justify the application of flood-related development controls to development on that lot, with the nature of those controls then being determined at the stage that the development approval is assessed. Therefore, in urbanised areas, a more effective approach is generally to use a “lot-based” assessment technique to identify FPA extents. This allows a holistic assessment of each cadastral lot, including consideration of the flood mechanisms, freeboard requirements, and the severity of flood affectation for each lot, to make an assessment about whether it is appropriate to apply development controls to that parcel of land.

Lot-based approaches have been implemented by many Councils in the Sydney Metropolitan Area. The Department of Planning and Environment (DPE) via the technical oversight has approved this technique for use in several studies completed as part of the NSW Flood Program, via the technical oversight role the department performs for these studies.

8.9.3. Principles for Identifying Flood Prone Land

Based on the above considerations, the definition of the FPA for this study area is best completed by applying a lot-based flood tagging methodology, answering the following questions for each lot:

1. Is the 1% AEP flood inundation of the lot (including freeboard consideration) sufficient to warrant flood-related development controls in line with the provisions of the DCP; and
2. Is the PMF flood inundation of the lot sufficient to warrant flood-related development controls for vulnerable land uses.

The goal of this tagging process is to identify flood prone land should be to determine which lots are at risk of mainstream flooding, or significant overland flow from upstream areas, so that those flows can be managed and development can mitigate against damage to property and risk to life. The process does not seek to manage intra-lot drainage and minor overland flows from neighbouring properties that can be managed relatively easily without Council intervention, or via broad stormwater and construction guidelines that apply to all development.

The key to determining the answers to the above is to distinguish for each lot the degree of flood risk which is significant enough to warrant flood-related development controls. The weather events that produce flooding in the study area are relatively short duration storm events producing high intensity rainfall (large rainfall depths in a short period of time). During these events, the intense rainfall occurs across the entire LGA, and the 1% AEP rainfall intensity is sufficient to produce nuisance inundation and potentially damage for every property within the LGA. However for the majority of properties, risks from the direct impacts of the rainfall and localised runoff can be appropriately managed through the drainage provisions of stormwater design guidelines, building codes, and other controls that are broadly applicable to all development. These widespread drainage controls are termed “local drainage” or “intra-lot” drainage. It is not appropriate or necessary to apply more specific flood-related development controls to every lot in the LGA – those controls apply for areas of significant flood risk.

For the purposes of this assessment, the principles adopted by WMAwater to differentiate "mainstream or overland flooding" from "local drainage" are consistent with the definitions in the NSW Floodplain Development Manual (Reference 3) as follows:

- "Mainstream flooding" means inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary lake or dam.
- "Overland flow" is a flow path that is above ground, where runoff is concentrated as it flows downhill. The question of whether "overland flow" should be classified as flooding is complicated. It depends on the magnitude of flow and the consequent hazard and difficulty it presents for managing flood risk. Generally overland flow can be categorised as either:
    - "Major drainage" involves:
        - the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or
        - water depths generally in excess of 0.3 m. These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or
        - major overland flow paths through developed areas outside of defined drainage reserves; and/or
        - the potential to affect a number of buildings along the major flow path.
    - "Local drainage" – smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.

This assessment seeks to identify lots affected either by mainstream flooding or major drainage overland flow, and assumes that local drainage is managed by adhering to the drainage principles laid out in Council's Private Stormwater Code (City of Ryde DCP Part 8.2, Reference 23) and the National Construction Code (Reference 27).

8.9.4. Flood Tagging Methodology

WMAwater designed the methodology to identify flood prone lots (affected by either mainstream or major overland flow) as per the principles outlined above. Flood tagging was undertaken using the two-step process displayed in Diagram 17.

Diagram 17: Two Step Flood Tagging Process

Step 1.
GIS Algorithm

Automated spatial analysis identifying the properties subject to flooding from the modelling results of the flood study

- 1% AEP flood extent identified using hydraulic modelling.
- Filtering applied to results to remove areas of local drainage, and identify mainstream and significant overland flow areas.
- 0.5m freeboard added for mainstream areas and stretched across nearby terrain.
- Affected lots identified.

Step 2.
Desktop Assessment -
Flood Behaviour Review

Reviewing flow paths and flood behaviour to confirm the magnitude and distinguish between overland flow / local drainage

- Review of flood behaviour undertaken to broadly classify the flow path
- Modification of tagging status to ensure consistent outcomes along flow paths and among neighbouring properties, based on the flow path review.

The updated design flood modelling results were used to identify flood affected lots for the 1% AEP and PMF design events. Initially, an automated GIS algorithm was implemented to identify the relevant lots. Further review of each flow path was then undertaken to ensure consistent outcomes along flow paths and among neighbouring properties. This review was based on consideration of local topography, hydraulic behaviour, localised structures such as buildings and walls, proximity to drainage infrastructure, and freeboard considerations.

Step One – GIS Analysis

GIS software was used to identify the cadastral lots which intersected with the modelled flood extents. This step required filtering of the flood extents to remove areas of trivial affectation or local drainage. This filtering process is necessary because inflows to the flood model are typically applied at all stormwater pits across the catchment. This approach is necessary to accurately understand the aggregation of flow in the lower catchment (where flooding is more severe), since this behaviour depends on how flow arrives from upstream, including consideration of the pipes, road gutters, and other possible routes for the water. This means that the models include relatively trivial upstream flow paths in road reserves, or shallow sheet flow across steep terrain, which does not necessarily produce a flood risk severe enough to require development controls or Section 10.7 flood tagging.

The filtering process that was applied to identify areas of "**Mainstream**" flood affectation was as follows:

- Filter the results to remove areas with depth less than 250 mm.
- Review and ignore remaining areas of isolated flooding that do not meet "Mainstream" criteria.

- Add 0.5 m freeboard to the 1% AEP flood surface.
- Stretch this raised surface across the adjacent terrain to include areas of land that are above the 1% AEP level, but below the “Mainstream” Flood Planning Level.

The filtering process that was applied to identify “**Overland Flow**” was as follows:

- Filter the results to remove areas less than 0.15 m depth,
- Review and ignore remaining “patches,” areas of isolated flooding that are not associated with a continuous overland flow path.

This filtering process was undertaken using a search algorithm that link together discontinuous patches of flooding that are close to each other and are likely to form a continuous flow path, while removing the remaining isolated areas.

Properties intersecting either of the extents produced from the above filtering process were identified with a provisional “Yes” status for Clause 7A(1).

<u>Step Two – Flow Path Review</u>

The aim of the desktop analysis was to review each flow path and identify at a high level (i.e. for groups of properties around a given flow path) where the results from the automated GIS tagging process may not be correct, for a range of reasons. This step involves a holistic review of flow paths to appreciate the local conditions and understand the degree of real flood risk.

For each flow path in each catchment, WMAwater reviewed the flood behaviour in a systematic fashion for each area with consideration of total flow rates, pipe network capacity and alignment, topography, presence of obstructions, etc.

The available catchment-wide flood modelling does not always reflect flow paths with small scale drainage features particularly well. The modelling uses a grid cell resolution of 1 m, which is suitable for resolving major overland flow paths, but cannot resolve the conveyance of small features, such as when flow is fully contained within a kerb/gutter on one side of the road. These gutter features can be hydraulically quite efficient, with high capacity, especially when the slope is steep (in the order of 1% or greater). The modelling will sometimes indicate that some flow spills out of the gutter when in reality it is likely to be contained within the road. It is therefore necessary to apply some judgement to areas of shallow overland flow in the raw modelling results to determine the potential for the overland flow path to occur in reality.

This step included review of all properties tagged during the GIS process, as well as surrounding lots or lots in the vicinity of flow paths that were not automatically identified, with consideration of the following factors:

- The nature of the flood behaviour at the lot, and whether it is a conveyance (flowing) or storage (ponding) area;
- Topographic information and potential anomalies in the aerial survey data;
- Stormwater networks and other drainage data in the vicinity of the lot;
- Freeboard considerations for mainstream flood or ponded areas.

Flow paths were considered block by block, or around the point of influence of a particular hydraulic control (like a low-lying area in the terrain). This step aimed to confirm at a precinct scale whether or not the flood risk is sufficient to warrant the application of development controls (i.e. the flow path constitutes mainstream or major overland flow), or whether it can likely be managed

Clause 7A(2) Flood Tagging Process

Clause 7A(1), which relates to flood risk up to the 1% AEP probability, is the primary consideration for development control for most land-use types. Clause 7A(2) relates to the residual flood risk between the 1% AEP and PMF and is only relevant to sensitive or hazardous development, or land where the City of Ryde considers that there is a particular risk to life or evacuation concern requiring application of development controls.

As this clause is only likely to apply in certain circumstances, and a much larger proportion of the LGA is subject to some level of flood risk from a PMF magnitude event than a 1% AEP event, it is not appropriate to apply the same level of detailed tagging review as was applied for Clause 7A(1) tagging. It is sufficient to rely on the outcomes from the GIS analysis for the Clause 7A(2) tagging.

For Clause 7A(2) tagging, the filtering process that was applied to the PMF results was as follows:

- Filter the results to remove areas with hazard less than 150 mm depth
- Review and ignore remaining "patches," areas of isolated flooding that are not associated with a continuous overland flow path.

Properties intersecting the filtered extents produced from the above process were identified with a "Yes" status for Clause 7A(2). Properties identified with a "Yes" status from the Clause 7A(1) tagging process were also tagged "Yes" for Clause 7A(2).

8.9.5. Outcomes of Flood Planning Area Assessment

The adopted approach for defining the Flood Planning Area (FPA) is to identify individual lots subject to Clause 7A(1) and Clause 7A(2) of the Regulation. Clause 7A(1) relates to flood risk up to the 1% AEP, considered to be the FPA, while Clause 7A(2) relates to flood risk up to the PMF, considered to be all flood prone land. These lots, tagged under either clause, will be subject to Section 10.7 flood notification. The lots identified for flood tagging are shown on Figure F28.

Table 35 provides a summary of the total number of lots tagged as being within the FPA and subject to Section 10.7 flood notification under Clause 7A(1) of the Regulation.

Table 35: Lots subject to Clause 7A(1) flood notification

Cadastral Lots within Study Area	Lots Tagged Under Clause 7A(1)	Percentage of Lots Tagged Under Clause 7A(1)
28085	4452	15.9%

Table 36 provides a summary of the total number of lots tagged for Section 10.7 flood notification under Clause 7A(2) of the Regulation.

Table 36: Total number of properties subject to Clause 7A(2) flood notification

Cadastral Lots within Study Area	Lots Tagged Under Clause 7A(2)	Percentage of Lots Tagged Under Clause 7A(2)
28085	8092	28.8%

9. MODEL SENSITIVITY ASSESSMENT

Model sensitivity testing was undertaken to determine the change to 1% AEP peak flood level resulting from the following variations in model assumptions or input data:

- Reduced rainfall losses (Figure H1),
- Mannings roughness reduced by 20% (Figure H2),
- Mannings roughness increased by 20% (Figure H3),
- Pit blockage increased by 25% (Figure H4),
- No structure debris blockage (Figure H5),
- Increased structure debris blockage (Figure H6),
- Reduced hydraulic energy losses at bridges (Figure H7),
- Increased rainfall intensity using RCP 4.5 scenario in 2090 (Figure H8),
- Increased rainfall intensity using RCP 8.5 scenario in 2090 (Figure H9),
- Sea Level Rise of 0.4 m (Figure H10), and
- Sea Level Rise of 0.9 m (Figure H11).

The scenarios and details are summarised in Table 37. Due to the similar urban features across models in the study area, it is adequate to undertake sensitivity tests in one of the models – TC Model in order to understand the impact of the model parameters.

Table 37: Overview of Sensitivity Analysis

Scenarios/ Parameters	Model Applied	Description
Rainfall losses	TC Model only	Antecedent Moisture Condition in DRAINS was changed to Type 4 soils with high runoff potential, very slow infiltration rates; depression storage for pervious and impervious areas were minimised.
Manning's "n"		The hydraulic roughness values were increased and decreased by 20%
Pit Inlet Blockage		Sensitivity was assessed by increasing 25% blockage for all inlet pits.
Culvert/ Bridge/ Pit Inlet Blockage		All blockage is removed to 0.
Culvert and Bridge Blockage		Sensitivity to blockage of culverts and bridges was assessed for: • Increasing 10% blockage for bridges • Increasing 25% blockage for culverts
Energy Losses		Energy losses for bridges was doubled.
Climate Change	All models	Sensitivity to rainfall and runoff estimates were assessed by increasing the rainfall intensities by 10% (approximate for 2090 RCP4.5) and 20% (approximate for 2090 RCP8.5).
Sea Level Rise	MQ, BK and PR Models	Sea level rise scenarios of increasing tailwater levels by 0.4 m (year 2050) and 0.9 m (year 2100) were assessed.

Key findings from the sensitivity results are summarised as follows:

- Rainfall losses: reducing rainfall losses to the minimum in terms of infiltration losses and depression storage in the DRAINS model will increase the peak flood levels across the catchments in different degrees. Along the overland flow paths towards Terrys Creek, peak flood levels were increased slightly by 0.01 m to 0.05 m. Peak levels within the mainstream channels are generally 0.1 m to 0.3 m higher. There are a few locations where the increases are larger, such as upstream of Eastwood railway station, where flood levels were more than 0.3 m higher, indicating this area is more sensitive to the change in rainfall losses.
- Manning's Coefficient: peak flood levels were generally insensitive to the change of Manning's n coefficients by ±20%. The impacts were approximately between -0.1 m and +0.1 m in the upper catchment, while the downstream reaches of Terrys Creek were within ±0.2 m. Similar to the impacts of rainfall losses, it is noted that the area upstream of the Eastwood station is more sensitive to the change.
- Blockages:
  - A 25% higher pit inlet blockage resulted in only a minor change to peak 1% AEP flood levels, typically being within ± 0.1 m across the Terrys Creek catchment.
  - No culvert blockages and no pit blockage would lower the peak levels by up to approximately 0.1 m in the upstream portions of the catchment, with the exception of the area upstream of Eastwood station, which is more sensitive to the change (approximately 0.2 m). Peak flood levels in the natural sections of Terrys Creek would be slightly higher due to the greater efficiency in upstream conveyance.
  - On the contrary, high structure blockage increased the peak flood levels by up to 0.3 m in the upstream portion of the channel while decreased those along the natural portion of the creek by approximately 0.1 m to 0.3 m. It is noted again that the area upstream of the Eastwood railway station had more than 0.5 m higher flood levels due to the high blockage of the key culvert under the rail line.
- Energy losses: The change in peak 1% AEP flood levels due to the doubled bridge energy losses were negligible.
- Climate change – rainfall intensity: With 10% and 20% increase in rainfall intensity, peak flood levels in most of the overland flooding areas were up to 0.1 m higher. Key areas that are more sensitive include the area upstream of the Eastwood railway station, natural downstream sections of Terrys Creek, the main watercourse of Shrimptons Creek (especially between Kent Road and Epping Road), Wicks Road underpass of M2 Motorway, flow paths through Morshead Street towards Epping Road, Lane Cover River, the main watercourse of Buffalo Creek, the downstream reach of Kittys Creek, the downstream area of the Denistone Catchment, the channel at the downstream end of Charity Creek and to the east of Meadowbank railway station around the TAFE NSW Meadowbank campus. At these locations, flood levels are generally 0.1 m to 0.2 m higher and up to 0.5 m higher at a few specific locations in the 2090 RCP 4.5 Scenario (10% increase in rainfall intensity). In the 2090 RCP 8.5 Scenario, the impacts in these areas are typically within 0.1 m to 0.3 m, with some of these locations having impacts

greater than 0.5 m.

- Climate change – sea level rise: Sea level rise scenarios were assessed for the Lane Cove River (including Buffalo and Kittys Creeks in the BK model and tributaries within the MQ model) and Parramatta River (including all catchments draining to the Parramatta River in the PR model). The areas most susceptible to rising sea levels along the Parramatta River include the channels at the downstream end of Archers Creek, Denistone Catchment and Charity Creek. The majority of the residential properties would not be directly affected due to the sea level rises. However, the houses in Lancaster Avenue in the downstream reach of the Archers Creek catchment may be affected. For the Buffalo and Kittys Catchment, the sea level scenarios would increase peak flood levels within the downstream reaches of the creeks by up to 0.3 m by 2050 and by up to 0.5 m by 2090 (using the adopted sea level rise projections). However, the impacts would not directly affect residential properties in the catchments. Further upstream on the Lane Cove River, sea level rise impacts extend to approximately Lane Cove Weir, near the Delhi Road crossing (Fullers Bridge). The only area of development to be impacted is River Avenue, Chatswood West. In the 2050 sea level rise scenario, the impact in this area is less than 0.05 m, and in the 2100 sea level rise scenario, the impact is approximately 0.1 m. This is likely to not have a significant impact on most properties.

9.1. Comparison with Previous Flood Studies

A comparison of the 1% AEP peak flood levels between the current Flood Harmonisation Study and the previous Flood Studies (Reference 5, 6, 7 and 8) was undertaken. The results are presented in Figure H12. Due to the changes in the underlying DEM in the TUFLOW models, a comparison of peak flood depths was also undertaken, with results shown in Figure H13. It should be noted that for the purpose of these comparison maps, no filtering was undertaken for the previous or current Flood Study results. This was because different filtering approaches were undertaken in the previous studies, and hence a consistent approach was adopted for all results to enable a direct comparison.

The results indicate that there is generally a reduction of flood extent and peak flood levels across the catchments compared with the pervious results. In particular, overland flooding extent is reduced significantly within the upstream portion of the catchments where flood levels are generally lower by up to 0.3 m. Within the mainstream creeks, the reduction is more notable, with reductions of more than 0.5 m, such as Terrys Creek through Eastwood CBD, in the downstream reaches of Terrys Creek, along the main flow path towards Porters Creek, in the downstream reaches of Lane Cover River, in the natural channels of Buffalo and Kittys Creek, along the flow path through Denistone Sports Club, and in Charity Creek upstream of Meadowbank train station. There are few areas subject to a relatively high increase in peak flood level (> 0.5 m) such as in Shrimptons Creek between ELS Hall Park and Epping Road, and the Lane Cove River reaches above its confluence with Industrial Creek. The comparison between peak flood depths indicates a similar trend.

These differences in the modelled peak flood level can be attributed to the following factors:

- Update from ARR87 to ARR19 guidelines, including:
  - Changes to the design IFD data
  - Changes to the design rainfall temporal patterns
  - Changes to design rainfall losses (for RAFTS nodes within the Eastwood and Terrys Creek catchment DRAINS models)
  - Changes in the critical duration due to the changes in hydrology outlined above and a new selection process based on ensemble results
- Updates to the TUFLOW hydraulic model schematisation, including:
  - Reducing grid cell size (for some models) to 2 m
  - Updating of the approach to modelling buildings (for some models)
  - Updating the version of TUFLOW used, including changing to the HPC engine rather than the Classic engine
  - Updating of material roughness (for some models) to ensure consistency across the study area, and adjusted during the calibration process
  - Updating of hydraulic structures including the stormwater pit and pipe network, and updates to culverts and bridges where required
  - Updating of blockage of hydraulic structures to align with ARR19 procedures
  - Updating of some 1D channels to be represented in the 2D domain
  - Updating the tailwater boundary conditions for the Parramatta River and Lane Cove Rivers
  - Change in source of Terrys Creek inflows into the Lane Cove River. The DRAINS model was the previous source and this has been updated to the more accurate TUFLOW model hydrographs.
- Updates to the TUFLOW hydraulic model based on catchment changes, including:
  - Updating of the underlying DEM to the latest available LiDAR data
  - Updating of building footprints and material roughness for recent developments
  - Inclusion of new or upgraded hydraulic structures

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Attachment A: ARR Datahub Data

Attachment A

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APPENDIX A TERMINOLOGY AND GLOSSARY

Appendix A

LIST OF ACRONYMS

1D	one-dimensional
2D	two-dimensional
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ARR	Australian Rainfall and Runoff
AWS	Automatic Weather Station
BoM	Bureau of Meteorology
CPU	Central Processing Unit
DA	Development Application
DCP	Development Control Plan
DEM	Digital Elevation Model
DPE	Department of Planning and Environment
DPIE	Department of Planning, Industry and Environment (now DPE)
FFA	Flood Frequency Analysis
FPA	Flood Planning Area
FRMSP	Floodplain Risk Management Study and Plan
GIS	Geographic Information System
GPT	Gross Pollutant Trap
GPU	Graphics Processing Unit
HPC	Heavily Parallelised Compute
IFD	Intensity, Frequency and Duration (Rainfall)
LiDAR	Light Detection and Ranging – an airborne laser survey technique
LGA	Local Government Area
mAHD	meters above Australian Height Datum
OEH	Office of Environment and Heritage (now DPE)
PMF	Probable Maximum Flood
PMP	Probably Maximum Precipitation
RADAR	Radio Detection and Ranging – typically used to record spatial variability of rainfall
TIN	Triangular Irregular Network
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software (hydraulic model)
WAE	Works-As-Executed

Terminology used in Report

Australian Rainfall and Runoff (ARR, Reference 2) recommends terminology that is not misleading to the public and stakeholders. Therefore the use of terms such as “recurrence interval” and “return period” are no longer recommended as they imply that a given event magnitude is only exceeded at regular intervals such as every 100 years. However, rare events may occur in clusters. For example there are several instances of an event with a 1% chance of occurring within a short period, for example the 1949 and 1950 events at Kempsey. Historically the term Average Recurrence Interval (ARI) has been used.

ARR 2016 recommends the use of Annual Exceedance Probability (AEP). Annual Exceedance Probability (AEP) is the probability of an event being equalled or exceeded within a year. AEP may be expressed as either a percentage (%) or 1 in X. Floodplain management typically uses the percentage form of terminology. Therefore a 1% AEP event or 1 in 100 AEP has a 1% chance of being equalled or exceeded in any year.

ARI and AEP are often mistaken as being interchangeable for events equal to or more frequent than 10% AEP. The table below describes how they are subtly different.

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very Frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	20
	0.02	2	50	50
	0.01	1	100	100
Very Rare	0.005	0.5	200	200
	0.002	0.2	500	500
	0.001	0.1	1000	1000
	0.0005	0.05	2000	2000
Extreme	0.0002	0.02	5000	5000
			↓	
			PMP/ PMPDF	

For events more frequent than 50% AEP, expressing frequency in terms of Annual Exceedance Probability is not meaningful and misleading particularly in areas with strong seasonality. Therefore events more frequent than 50% AEP should be expressed as X Exceedances per Year (EY). For example, 2 EY is equivalent to a design event with a 6 month recurrence interval where there is no seasonality, or an event that is likely to occur twice in one year.

The Probable Maximum Flood is the largest flood that could possibly occur on a catchment. It is related to the Probable Maximum Precipitation (PMP). The PMP has an approximate probability. Due to the conservativeness applied to other factors influencing flooding a PMP does not translate to a PMF of the same AEP. Therefore an AEP is not assigned to the PMF.

Glossary – from the NSW Floodplain Development Manual (April 2005 edition)

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

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

	of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or

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

APPENDIX B NOVEMBER 1984 FLOOD CALIBRATION RESULTS AND PHOTOS

Appendix B

APPENDIX C FEBRUARY 1990 FLOOD CALIBRATION RESULTS AND PHOTOS

Appendix C

APPENDIX D FEBRUARY 2010 FLOOD CALIBRATION RESULTS AND PHOTOS

Appendix D

APPENDIX E NOVEMBER 2018 FLOOD CALIBRATION PHOTOS

Appendix E

APPENDIX F DESIGN FLOOD EVENT MAPPING

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Appendix F

APPENDIX G DESIGN FLOOD EVENT REPORTING TABLES

Appendix G