KU-RING-GAI COUNCIL

LANE COVE – NORTHERN CATCHMENTS FLOOD STUDY

DRAFT STAGE 4 REPORT – DRAFT FLOOD STUDY

APRIL 2024

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Project Lane Cove – Northern Catchments Flood Study	**Project Number** 121054
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Cover photo: channel upstream of Warrowa Avenue, West Pymble (Degotardi, Smith & Partners)

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LIST OF ACRONYMS

1D	One-dimensional
2D	Two-dimensional
AEP	Annual Exceedance Probability
AMS	Annual Maximum Series
AMC	Antecedent Moisture Condition
ARI	Average Recurrence Interval
ARR	Australian Rainfall and Runoff
BoM	Bureau of Meteorology
DCP	Development Control Plan
DPE	Department of Planning and Environment
ELVIS	Elevation Information System
ERP	Emergency Response Planning
EY	Exceedances per Year
FERC	Flood Emergency Response Classification
FIE	Flooded Isolated Elevated Areas
FIS	Flooded Isolated and Submerged Areas
FRMS&P	Floodplain Risk Management Study and Plan
GIS	Geographic Information System
GSDM	Generalised Short Duration Method
HEC-RAS	Hydrologic Engineering Centre's River Analysis System
IFD	Intensity, Frequency and Duration (Rainfall)
KRGC	Ku-ring-gai Council
LEP	Local Environmental Plan
LGA	Local Government Area
LPI	Land and Property Information
mAHD	meters above Australian Height Datum
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SCIMS	Survey Control Information Management System
SES	State Emergency Service
SIX	Spatial Information Exchange
SW	Sydney Water
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software (hydraulic model)

ADOPTED TERMINOLOGY

Australian Rainfall and Runoff (ARR, ed Ball et al, 2019) 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 2019 recommends the use of Annual Exceedance Probability (AEP). 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.

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 the term Exceedances per Year (EY) is recommended. Statistically a 0.5 EY event is not the same as a 50% AEP event, and likewise an event with a 20% AEP is not the same as a 0.2 EY event. For example an event of 0.5 EY is an event which would, on average, occur every two years. A 2 EY event is equivalent to a design event with a 6 month ARI where there is no seasonality, or an event that is likely to occur twice in one year.

The Probable Maximum Flood (PMF) 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.

This report has adopted the approach recommended by ARR and uses % AEP for all events rarer than the 50 % AEP and EY for all events more frequent than this as shown in the table below.

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	19.5
	0.02	2	50	49.5
	0.01	1	100	99.5
Very Rare	0.005	0.5	200	199.5
	0.002	0.2	500	499.5
	0.001	0.1	1000	999.5
	0.0005	0.05	2000	1999.5
Extreme	0.0002	0.02	5000	4999.5
			↓	
			PMP/ PMP Flood	

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***
    - 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 Ku-ring-gai Council. Ku-ring-gai Council has prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. The document does not necessarily represent the opinions of the NSW Government or the Department of Planning and Environment.

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:

- Ku-ring-gai Floodplain Risk Management Committee
- Residents of the study area
- Ku-ring-gai Council
- Department of Planning and Environment
- NSW State Emergency Service

EXECUTIVE SUMMARY

The following Stage 4 report comprises an overview of the work that has been undertaken by WMAwater on the Lane Cove - Northern Catchments Flood Study. Stage 4 comprises the Draft Flood Study.

Introduction

Ku-ring-gai Council (KRGC) engaged WMAwater to undertake the Lane Cove Northern Catchments Flood Study. The objective of this study is to improve understanding of flood behaviour and impacts, and better inform management of flood risk in the study area. The study area includes the suburbs of South Turramurra and parts of Pymble, West Pymble, Turramurra and Wahroonga. It covers an area of approximately 14.7 km^2 (1,470 ha) and consists of a number of headwater catchment areas flowing to the Lane Cove River, including Rudder Creek, Quarry Creek, Congham Creek, Avondale Creek, Fox Valley (Peppermint and Water Dragon Creeks) and Coups Creek. The upper parts of the catchments are largely residential development, while the lower parts are primarily steep forested valleys.

Available Data

As part of the data collection, WMAwater received the previous studies undertaken and models developed by KRGC. Additional data received included GIS datasets of stormwater assets, buildings, land use zoning, topography and aerial imagery. A detailed topographic survey was undertaken for select channels and culvert structures located within urbanised areas. Historic rainfall data was also obtained from the Bureau of Meteorology and Sydney Water.

Community Consultation

At the commencement of the project, the community were informed of the study and provided the opportunity to contribute their observations of flooding within the catchment. A total of 192 responses to a community questionnaire were obtained, with 82 of these indicating they had experienced flooding in the past.

Apart from residents with a creek at the rear of their property (or front, that is crossed by a driveway bridge/culvert), the majority of respondents described overland flow flood behaviour (i.e. the flood waters were not confined to a defined creek or channel). A large proportion of these also mention blockage of street drainage pits as contributing to flooding. The most common flood events mentioned were the recent events of 2020, 2021 and 2022.

Model Development

The models developed to simulate flood behaviour in the study area consist of a two-stage process:

1. Hydrologic modelling using DRAINS to convert rainfall to runoff
2. Hydraulic modelling using TUFLOW to estimate overland flow distributions, flood depths, levels and velocities.

The DRAINS hydrologic model was developed from an existing model. A total of 2,394 sub-catchments were delineated and sub-catchment parameters were assigned based on topographic and land use data. Typical model parameters were adopted as there is not sufficient information to adjust these for local catchment conditions.

The TUFLOW hydraulic model covers the entire study area and a portion of Coups Creek outside the study area. The model consists of a 1 m by 1 m regular grid. The best available terrain and structure data was incorporated into the model, along with model adjustments to ensure that hydraulic features (including gutters and channels) were adequately represented. The simulated runoff hydrographs from the DRAINS model are applied to the TUFLOW model as inflows.

Model Calibration

There is only limited data for model calibration. Records of overland flow and flooding throughout the study area generally consist of qualitative descriptions, rather than recorded flood levels for specific events. As such, a full model calibration was not undertaken. Rather, a validation was undertaken.

The historic events of 1991, 2020, 2021 and 2022 were simulated and compared with the observed flood behaviour, primarily captured by the questionnaire provided to residents as part of this study. The results (Appendix E) indicate an overall good match to the observed flood behaviour with key flow paths being represented and modelled flood depths typically being within ±0.2 m of that observed.

Design Flood Modelling

Design flood modelling was undertaken in accordance with Australian Rainfall and Runoff (ARR) 2019 guidelines, including adoption of design rainfalls from the Bureau of Meteorology, consideration of areal reduction factors and blockage of hydraulic structures. ARR 2019 requires an ensemble of temporal patterns to be run for each duration and these were simulated in the hydrologic and hydraulic model. The critical storm duration (duration that produces the highest flood level) was determined based on the mean of the 10 temporal patterns for each duration and varied across the catchment from 10 minutes to 45 minutes. A 45 minute storm was found to adequately represent the typical behaviour across the study area. The design flood events simulated were the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% annual exceedance probability (AEP) events. The Probable Maximum Flood (PMF) was also simulated in accordance with the Generalised Short Duration Method. The critical duration for the PMF was 15 minutes and 30 minutes. Design flood depths, levels, velocities, hydraulic hazard and hydraulic categories were mapped and are provided in Appendix F. Flood results were also tabulated and plotted at key road crossings, with results presented in Appendix G.

Sensitivity Analysis

A sensitivity analysis (Appendix H) was undertaken for key modelling parameters by varying the adopted values and assessing the change in peak flood levels. Peak flood levels are relatively insensitive to changes throughout the urban areas, with increasing sensitivity in the downstream forested areas. Flood levels were most sensitive to the antecedent moisture conditions and rainfall losses.

1. INTRODUCTION

1.1. Study Objectives

Ku-ring-gai Council (KRGC) engaged WMAwater to undertake the Lane Cove – Northern Catchments Flood Study. This study is jointly funded by the NSW Department of Planning and Environment (DPE) and the KRGC. The flood study is the first step in the NSW flood program and will provide the basis for subsequent steps such as the Floodplain Risk Management Study and Plan (FRMS&P).

The Lane Cove – Northern Catchments Flood Study will define the existing flood behaviour within the study area using computer models. Those models will use the data available to create an accurate representation of the existing catchment flood behaviour. Once the models are established, calibrated and validated, they can be used to subsequently undertake a FRMS&P to identify existing flood risk and develop mitigation options to reduce this risk. The outputs of the study will also be used in planning for future development of the catchment and providing advice to the community and emergency response agencies.

The objective of this study is to improve understanding of flood behaviour and impacts, and better inform management of flood risk in the study area. It aims to provide an understanding of the full range of flood behaviour and consequences in the study area.

1.2. Study Area

The study area is located north-east of the Lane Cove River and covers the northern catchment area of local creeks within the Ku-ring-gai Local Government Area (LGA) that drain to the Lane Cove River. The study area is approximately bounded by Ryde Road to the east (catchment divide with Blackbutt Creek), the Pacific Highway and North Shore railway line to the north-east (Lane Cove River catchment boundary) and Coups Creek to the north (LGA boundary with Hornsby Shire Council). It includes the suburb of South Turramurra and parts of Pymble, West Pymble, Turramurra and Wahroonga. It covers an area of approximately 14.7 km^2 (1,470 ha) and consists of a number of headwater catchment areas flowing to the Lane Cove River. The area is drained by a series of creeks including Rudder Creek, Quarry Creek, Congham Creek, Avondale Creek, Fox Valley (Peppermint and Water Dragon Creeks) and Coups Creek. The creeks generally drain in a south-westerly direction to the Lane Cove River. A summary of the catchment areas for these creeks is provided in Table 1. The study area is shown in Figure 1.

Table 1: Summary of Catchment Areas

Catchment	Area (km^2)
Rudder Creek	0.26
Quarry Creek	1.22
Congham Creek	0.39
Avondale Creek	5.17
Fox Valley (Peppermint Creek and Water Dragon Creek)	4.06

Catchment	Area (km^2)
Coups Creek	4.71 (2.44 within the study area)
Other Lane Cove River Tributaries	1.10
Total[1]	**14.64**

[1] Total catchment area within the study area. There are some minor additional areas within the study area that drain directly to the Lane Cove River that have not been included in this study.

The upper parts of the catchments are largely residential development, with stormwater pits and pipes collecting rainfall runoff. The land use in the upper part of the study area catchment is mostly low-density residential properties, with small areas of commercial development (primarily around the Pacific Highway), educational campuses, and open space (such as parks, golf courses and sporting fields). Further downstream there are overland flow paths and channels that form through these residential areas. When channels form, these are usually within vegetated corridors, such as that shown in Photo 1. There are some flow paths through private property along stormwater easements and open sections of creek channel. In the lower catchment areas, there are steep forested valleys primarily within bushland reserves and the Lane Cove National Park.

Photo 1: Example of a vegetated creek in the study area

The highest elevation is approximately 210 mAHD on the northern edge of the study area in Wahroonga and the lowest elevation of the urbanised areas is typically around 70 mAHD, although Gloucester Avenue in West Pymble descends to approximately 20 mAHD. The elevation of the study area is shown in Figure 2.

The upstream urbanised parts of the catchment drain into well-defined channels or informal overland flow paths that discharges into the forested areas downstream. In large rainfall events where the capacity of the pit and pipe system is exceeded, these overland flow paths are activated and can cause inundation and damage to property. Some roads are also prone to flooding and present a risk to motorists. There have been numerous reports of flooding within the catchment, the most recent being March 2022.

2. AVAILABLE DATA

2.1. Previous Studies

2.1.1. Lane Cove Northern Catchments Stormwater Planning Study, Cardno Willing, 2005

Cardno Willing was engaged by KRGC to establish a stormwater planning study for several sub-catchments within the Lane Cove Northern Catchments study area (Reference 1). These sub-catchments cover the Lane Cove 1, Lane Cove 2, Lane Cove 3 (Congham Creek), Avondale Creek, Fox Valley and Coups Creek catchments. The study prepared hydrological models (DRAINS) and water quality models (MUSIC), analysed the performance of the existing drainage system and identified priority capital works for improving stormwater system capacity and stormwater quality within the framework set up by the Lane Cove River Stormwater Management Plan (SMP). Stormwater capacity upgrades and water quality devices were recommended at various locations, with three sites being identified for potential integrated water quantity and quality management measures including West Pymble Primary School, The Glade Oval and Auluba Reserve. The adopted DRAINS models were provided by KRGC for use in this study.

2.1.2. Local Catchment Plan - Lane Cove River Southern Region Catchments, URS, 2006

URS was engaged by KRGC to develop a Local Catchment Plan for several catchments draining to Lane Cove River (Reference 2). Some of these are within the current study area, including Lane Cove 4 (Rudder Creek) and Lofberg Quarry Creek (Quarry Creek). This report established the status of the catchments, their drainage network, their water quality status and provided a prioritised set of options for improvement, concept designs and integrated outcomes to guide future capital works programs by KRGC. The study developed hydrologic models (DRAINS) and water quality models (MUSIC) and recommended a range of management measures based on results from these models. The adopted DRAINS models were provided by KRGC for use in this study.

2.1.3. Preliminary Flood Mapping Report, Mott MacDonald Hughes Trueman, 2011

KRGC commissioned Mott MacDonald Hughes Trueman to carry out a floodplain mapping exercise (Reference 3). The study relied on existing hydrologic models (DRAINS) and developed one-dimensional (1D) hydraulic models (HEC-RAS) across the KRGC LGA to derive flood extents. The models were not calibrated, however, hydrologic estimates were verified using the Rational Method. The 1 in 20 year and 1 in 100 year Average Recurrence Interval (ARI) flood extents were established for major drainage overland flow paths. A series of flood maps and associated GIS layers were developed for KRGC for the purpose of flood planning and was not considered to be a rigorous 'flood study'. The GIS layers were provided by KRGC.

2.1.4. Norman Griffiths Oval Flood Assessment Final Compendium Report, Jacobs, 2018

KRGC engaged Jacobs to undertake a flooding assessment for Noman Griffiths Oval in West Pymble (Reference 4). The report is a summary of all the investigations undertaken for the oval upgrade, which serves as a detention basin on Quarry Creek. Drainage works were required to achieve 2% AEP flood immunity of a new synthetic field surface. The objectives of these studies were to:

- Review the data and DRAINS model available for the Lofberg Quarry catchment.
- Establish a two-dimensional (2D) TUFLOW hydraulic model.
- Determine existing case flood conditions for the catchment for a range of flood events 0.2 exceedances per year (EY) up to the 1% AEP event.
- Investigate a series of options for the oval upgrade including costing of those options.

A field bypass option, where drainage capacity underneath the field was provided, was selected as the preliminary design to proceed to concept design. KRGC considered the outcomes of the study and concluded that the Norman Griffiths Oval upgrade project would not be feasible during the development of a concept design. The report was compiled based on five memos that had been issued to conclude the project before the concept design was finalised.

2.1.5. Norman Griffiths Oval - Flood Risk Investigation, BMT, 2020

Following the previous assessment for Norman Griffiths Oval (Reference 4), KRGC investigated the potential for alternative oval upgrade options following strong community support. BMT was engaged by KRGC to undertake further flood risk investigations, particularly considering the current downstream flood risk and changes with potential field upgrades (Reference 5). The TUFLOW model developed by Jacobs (Reference 4) was refined for both the exiting case and proposed upgrade. Changes to flood risk and preliminary costing of options were investigated. The models used for the Norman Griffiths Oval investigation were provided by Council and will be incorporated into the current broader Lane Cove Northern Catchments Flood Study where appropriate.

2.2. Topographic Data

The Digital Elevation Model (DEM) is a representation of the ground topography and one of the primary inputs into a flood model. The study area DEM was developed using the Light Detection and Ranging (LiDAR) data and detailed topographic survey data, as outlined in the following sections.

2.2.1. LiDAR

The majority of the study area DEM was developed using the LiDAR data. This is a form of aerial survey that uses a laser scanner mounted to an aircraft. This data produces a high-resolution model of the ground elevation over large areas. The DEM is derived from a series of points with a typical density of 4 point per square metre. A gridded DEM is produced from filtered ground return points. The NSW Government (Spatial Services) holds this data and it is publicly available through the Elevation Information System (ELVIS, https://elevation.fsdf.org.au/). The 1 m DEM was obtained for the study area, based on data captured in May/June 2020, with a small portion being captured in June 2019. The LiDAR has a reported vertical accuracy of 0.3 m in the vertical (95% confidence interval) and 0.8 m in the horizontal (95% confidence interval). The terrain across the study area is shown in Figure 2. The LiDAR-derived DEM has limitations in accuracy where there is dense vegetation or waterbodies. Due to this limitation, additional survey was obtained, as outlined in Section 2.2.2 below.

WMAwater undertook a comparison of the 2019/2020 LiDAR data with Survey Control Information Management System (SCIMS) survey benchmarks from NSW Spatial Services. The data is available through the NSW Spatial Information Exchange (SIX, http://six.nsw.gov.au/). These are typically located within urban areas on roadways and footpaths. The following filtering of points was applied in order to obtain a reliable SCIMS dataset:

- Removal of points marked as “Destroyed”, “Uncertain” or “Not Found” for their status.
- Removal of points marked as “U” for Vt class (unknown/unreliable survey).
- If the difference between the point and the LiDAR dataset was greater than 1 m.

This yielded a total of 233 reliable points within the study area. A histogram of the difference in level between the 2019/2020 LiDAR and these SCIMS points is shown in Diagram 1. The histogram shows that the differences are primarily within 0.3 m (95% of points), which is considered a reasonable match. There is a slight skew, indicating a bias for the LiDAR data to be slightly higher than the survey marks. The median difference is approximately 0.02 m, with 80% of the points being within ±0.15 m. This demonstrates a high quality dataset that is considered reliable for the purposes of flood modelling.

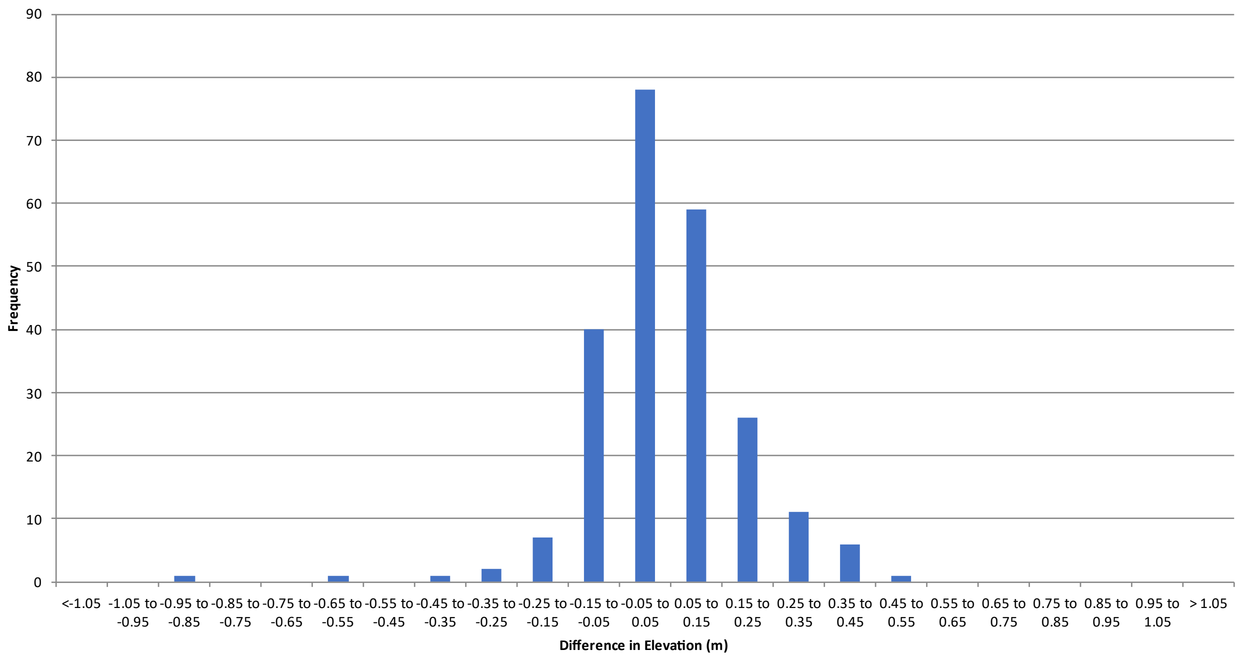


Diagram 1: Histogram of the difference between the 2019/2020 LiDAR dataset and the SCIMS points

For comparison purposes, the same analysis was undertaken for the 2013 LiDAR dataset. The histogram of the differences is shown in Diagram 2. This indicates a similar level of reliability, but a slight skew toward the negative side, indicating a bias for the LiDAR data to be slightly lower than the survey marks. While both datasets appear to be similar in their reliability against the survey marks, the 2019/2020 LiDAR dataset is considered to be more representative of current conditions and development. There are a number of developments that have occurred since 2013, such as the subdivision around Grey Horse Close and several other developments scattered throughout the study area. These would only be represented in the 2019/2020 LiDAR and hence the 2019/2020 LiDAR was adopted for this study.

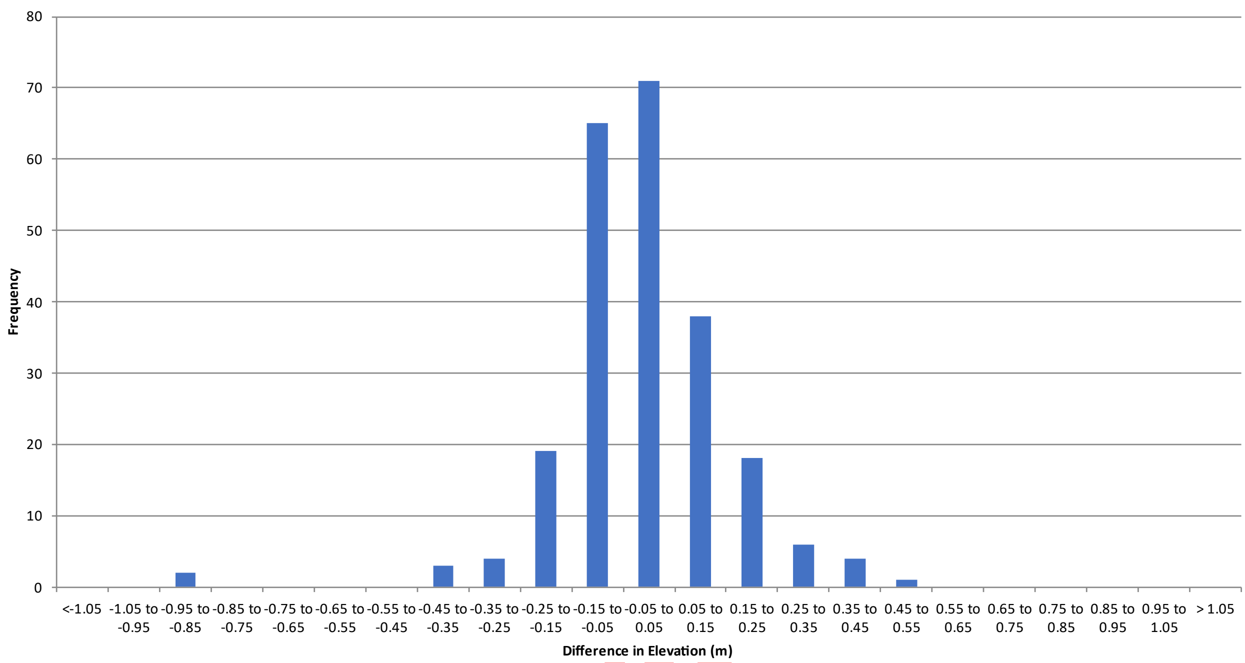


Diagram 2: Histogram of the difference between the 2013 LiDAR dataset and the SCIMS points

2.2.2. Detailed Topographic Survey

Due to the limitations of LiDAR data in heavily vegetated areas, detailed topographic survey of several channels was commissioned. These channels were selected due to their proximity to development, poor quality of the LiDAR data and the requirement for accurate representation of channels. There is a need to accurately represent the channels and their capacity through these residential areas. The location of these channels can be seen in Figure 3. The culverts upstream, downstream and within these reaches were also surveyed. One bridge structure on Troon Place was also surveyed.

The survey was undertaken by Degotardi Smith and Partners in July/August 2023. A copy of the data is provided in Appendix B. It consisted of 37 cross sections, 9 culverts and 1 bridge. The survey covered 7 different reaches.

2.3. Aerial Imagery

Aerial imagery was provided by KRGC in tif format. This dataset consists of 12 cm resolution aerial imagery captured in 2020. A 2011 LPI dataset was also provided by KRGC. Aerial imagery was also available on platforms such as Google Maps (www.maps.google.com.au), Nearmap (www.nearmap.com) and SIX Maps (six.maps.nsw.gov.au).

2.4. Land Use Zoning

The Ku-ring-gai Local Environment Plan (LEP) 2015 applies to the study area. The LEP zoning was provided by KRGC for the study area as well as cadastral boundaries. This information identifies lot boundaries and the zoning (such as residential, commercial, industrial and recreational areas). The study area is mostly composed of R2 Low density residential lots with the lower part of the catchments categorised as E1 – National Park or E2 – Environmental Conservation. The upper eastern part of the study area is R3 - Medium Density Residential or R4 - High Density Residential. Other GIS boundaries, such as catchments, railway corridors, road corridors and easements were also provided.

2.5. Hydraulic Structures

KRGC provided GIS layers of their stormwater database for the study area. A total of 2,634 pipes and 2,902 pits are located within the study area. The dimension of the pits and pits lintel were typically available as well as the pit depth and surface level. The dimension of pipes and culverts were also available. These appear to align with the information in the DRAINS models (References 1 and 2).

2.6. Buildings

Buildings are a major hydraulic feature in the study area. Their presence within overland flow paths can cause flow constrictions and/or divert floodwater. KRGC provided a building layer indicating building footprints. Approximatively 8,000 buildings are within the study area.

2.7. Site Visit

A site visit was conducted on 10th December 2021 by WMAwater staff. The site visit was conducted of the whole study area, with a focus on the major creeks and where these intersect with urban development. The aim of the site visit was to gain an appreciation of the study area including the topography, waterways and urban development. In particular, features that are not readily seen in aerial imagery, LiDAR data or stormwater datasets was able to be observed. Two examples are shown in Photo 2 and Photo 3 below.

Photo 2: Flowpath down a shared driveway upstream of Norman Griffiths Oval on Lofberg Road

Photo 3: Creek downstream of Monteith Street through residential properties

2.8. Historic Rainfall Data

2.8.1. Overview

Rainfall data is recorded either daily (24-hour rainfall totals to 9:00 am) or continuously (pluviometers measuring rainfall in small increments – less than 1 mm). Daily rainfall data has been recorded for over 100 years at many locations within the Sydney basin. However, pluviometers have generally only been installed for widespread use since the 1970s. Together these records provide a picture of when and how often large rainfall events have occurred in the past.

Care must be taken when interpreting historical rainfall measurements. Rainfall records may not provide an accurate representation of past flooding due to a combination of factors including local site conditions, human error or limitations inherent to the type of recording instrument used.

Examples of limitations that may impact the quality of data used for the present study are highlighted in the following:

- Rainfall gauges frequently fail to accurately record the total amount of rainfall. This can occur for a range of reasons including operator error, instrument failure, overtopping and vandalism. In particular, many gauges fail during periods of heavy rainfall and records of large events are often lost or misrepresented.
- Daily read information is usually obtained at 9:00 am in the morning. Thus, if a single storm is experienced both before and after 9:00 am, then the rainfall is "split" between two days of record and a large single day total cannot be identified.
- In the past, rainfall over weekends was often erroneously accumulated and recorded as a combined Monday 9:00 am reading.
- The duration of intense rainfall required to produce overland flooding in the study area is typically less than 6 hours (though this rainfall may be contained within a longer period of rainfall). This is termed the "critical storm duration". For a larger catchment (such as the Parramatta River) the critical storm duration may be greater (say 9 hours). For the study

area a short intense period of rainfall can produce flooding but if the rain starts and stops quickly, the daily rainfall total may not necessarily reflect the magnitude of the intensity and subsequent flooding. Alternatively, the rainfall may be relatively consistent throughout the day, producing a large total but only minor flooding.
- Rainfall records can frequently have “gaps” ranging from a few days to several weeks or even years.
- Pluviometer (continuous) records provide a much greater insight into the intensity (depth vs. time) of rainfall events and have the advantage that the data can generally be analysed electronically. This data has much fewer limitations than daily read data. Pluviometers, however, can also fail during storm events due to the extreme weather conditions.

Intense rainfall events which cause overland flooding in highly urbanised catchments are usually localised and as such are only accurately represented by a nearby gauge, preferably within the catchment. Gauges sited even only a kilometre away can show very different intensities and total rainfall depths.

The rainfall data described in the following sections pertains to information that was used in model calibration.

2.8.2. Daily Rainfall Stations

There are a number of daily rainfall stations available around the study area operated by the Bureau of Meteorology (BoM). Stations within approximately 5 kilometres of the study area were analysed. Only stations with useful data for recent years (from 1991, the earliest calibration event) have been selected. This resulted in a total of 23 stations that were analysed, and these are listed in Table 2 and shown in Figure 4. Diagram 3 also shows the operating period of these stations.

Table 2: Daily rainfall stations within and around the study area

Station Number	Station Name	Start Date	End Date	Length of Record (years)	% Complete
66010	Chatswood Council Depot	1/01/1897	31/12/1993	97.1	60.0
66011	Chatswood Bowling Club	1/07/1951	Open	71.1	44.8
66020	Epping Chester Street	22/01/1886	10/10/2002	116.8	76.5
66028	Hornsby (Pretoria Parade)	1/12/1923	31/05/1995	71.5	84.6
66032	Lindfield West	1/04/1950	30/06/1992	42.3	96.2
66047	Pennant Hills (Yarrara Road)	8/06/1900	1/01/2021	120.7	65.1
66080	Castle Cove (Rosebridge Ave)	1/10/1958	Open	63.8	99.6
66087	Eastwood Bowling Club	1/01/1955	30/04/2004	49.4	80.8
66114	North Turramurra (Dryden Rd)	1/05/2005	31/12/2009	4.7	99.2
66120	Gordon Golf Club	6/08/1906	30/04/2022	115.8	89.1
66156	Macquarie Park (Willandra Village)	1/11/1970	Open	51.7	82.4
66157	Pymble (Canisius College)	1/12/1947	31/07/2011	63.7	98.1
66158	Turramurra (Kissing Point Road)	1/01/1936	31/10/2018	82.9	99.9
66185	Carlingford (Barellan Av)	1/01/1986	30/09/2011	25.8	98.0
66189	West Pymble (Wyuna Road)	1/04/1992	31/05/2011	19.2	90.6
66205	Wahroonga (Boundary Road)	1/03/1998	31/12/2005	7.8	99.2
66206	St Ives (Richmond Avenue)	10/04/1998	16/02/2021	22.9	88.5
66211	Wahroonga (Ada Avenue)	6/03/2010	Open	12.3	99.3
66213	North Ryde Golf Club	10/08/2011	Open	10.9	98.8
67062	Cherrybrook (Casuarina Drive)	1/09/2007	30/09/2015	8.1	99.4
67065	Hornsby (Swimming Pool)	6/02/2008	Open	14.4	79.0
67089	West Pennant Hills (Cumberland State For	1/06/1949	5/02/2015	65.7	89.0
67098	West Pennant Hills (Oratava Ave)	1/01/1943	30/04/2006	63.4	94.5

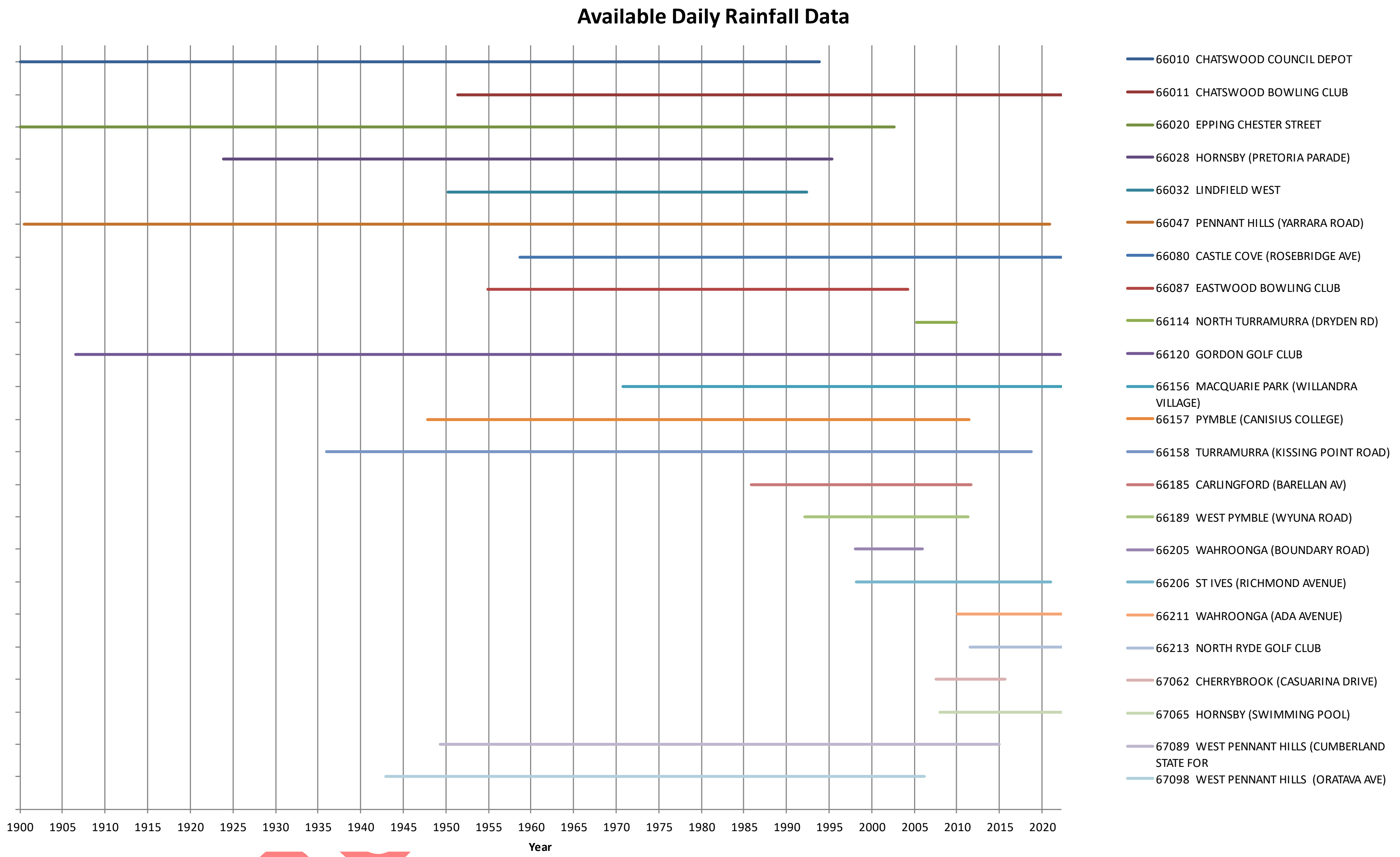


Diagram 3: Available daily rainfall station records for the study area

2.8.3. Sub-Daily Rainfall Stations

A number of continuous rainfall stations (pluviometers) that record data at a sub-daily level are available around the study area. These include stations operated by BoM and Sydney Water (SW). Both Automatic Weather Stations (AWS) that provide rainfall data at a minute resolution and traditional pluviograph stations that provide data at a 6-minute resolution are available from BoM. The only available sub-daily rainfall station operated by the BoM is for Wahroonga Reservoir, which ceased recording sub-daily rainfall in 1973 and the station was closed in January 1991. The next closest stations are more than 8 km away, at Parramatta North and Duffys Forest. No BoM sub-daily rainfall stations were included in the current analysis for this reason.

SW operates eight tipping bucket rainfall stations within approximately 5 km of the study area boundary. These gauges are summarised in Table 3 and shown in Figure 4.

Table 3: Sub-daily rainfall stations within and around the study area

Station Number	Station Name	Start Year	End Year	Operating Authority	Distance from centroid of Study Area (km)
566008	North Ryde Gc	2014	Open	SW	6.3
566017	Chatswood Bowling Club	1980	Open	SW	9.3
566037	Ryde Pumping Station	1980	Open	SW	7.1
566073	Pymble Bowling Club	1987	Open	SW	3.2
566076	Pennant Hills Bowling Club	1989	Open	SW	3.2
566083	North Epping	1990	Open	SW	1.5
566084	North St Ives (Police Driving School)	1990	Open	SW	8.6
566085	East Lindfield Bowling Club	1990	Open	SW	8.5

2.8.4. Calibration Events

The SW rainfall stations of Pymble Bowling Club (566073), Pennant Hills Bowling Club (566076) and North Epping (566083) were selected to analyse the major rainfall events that have occurred in the study area. These stations are the closest gauges with high resolution (sub-daily) rainfall data. The ten largest daily rainfall totals for each of the stations are shown in Table 4.

Table 4: Ten largest rainfall events at selected stations

Pymble Gauge 566073		Pennant Hills Gauge 566076		North Epping Gauge 556083	
Date	Depth of rainfall in 24 hours (mm)	Date	Depth of rainfall in 24 hours (mm)	Date	Depth of rainfall in 24 hours (mm)
3/02/1990	296	3/02/1990	221.5	11/06/1991	230
11/06/1991	218.5	11/06/1991	213.5	10/02/2020	176.5
10/02/2020	196	31/08/1996	166	9/02/1992	171.5
4/02/1990	195.5	21/04/2015	161.5	5/06/2016	154.5
7/02/2010	189	10/02/2020	157.5	21/04/2015	148.5
6/07/1988	184	4/02/1990	156.5	6/06/2016	136
17/01/1988	182	9/02/1992	148	31/08/1996	133.5
9/02/1992	168.5	6/06/2016	145	10/02/1992	124.5
30/04/1988	163	5/06/2016	137	6/01/2016	117
1/05/1988	160.5	22/04/2015	131.5	22/04/2015	116.5

The 11th of June 1991 is in the top two events at all three gauges. There were no specific community responses relating to this event, however, it is likely one of the largest events to have occurred in the catchment.

The 10th of February 2020 is in the top five events at all three gauges. There were also 5 community responses relating to this event, the most responses for a single event (see Section 3). The depth of rainfall is above 150 mm at all gauges with almost 200 mm recorded at North Epping and for this reason this event was selected as a calibration event.

The March 2021 event is not in the top 10 events at each site. The storm for this event lasted several days, which resulted in flooding across the study area. There are two community responses related to this event (see Section 3). This was selected as a calibration event.

The 9th of March 2022 is not in the top 10 events at each site. It is respectively the 11th, 17th and 19th storms recorded at each of the gauges 566073 (152.5mm), 566076 (102mm), and 566083 (88.5mm). There are three community responses related to this event (see Section 3). This was selected as a calibration event.

The events selected for the calibration were 11th of June 1990, 10th of February 2020, 21st March 2021 and 9th of March 2022. The depths of rainfall recorded at the closest rainfall stations to the catchment for these events are shown in Table 5.

Table 5: Rainfall depths at the closest rainfall stations for calibration events

Station Number	Station Name	11th June 1991	10th February 2020	21st March 2021	9th March 2022
566076	Pennant Hills Bowling Club	213.5	157.5	110	102
566083	North Epping	230	176.5	106	88.5
566073	Pymble Bowling Club	218.5	196	122.5	152.5
66211	Wahroonga (Ada Avenue)	-	207.2	70.4	137.4
66156	Macquarie Park (Willandra Village)	195.2	201.2	99	69.8

3. COMMUNITY CONSULTATION

At the commencement of the project, the community were informed of the study and provided the opportunity to contribute their observations of flooding within the catchment. Information on the study was provided on KRGC's 'Flooding' webpage, where a 'Survey Monkey' questionnaire could be accessed to share experiences of flooding as well as uploading photos and videos. The 'Loving Living Ku-ring-gai' Facebook page also informed the community of the study and questionnaire. A newsletter was sent to 856 owners and occupiers to inform them of the study and the questionnaire. The community consultation materials can be seen in Appendix C.

The questionnaire was open between 25 January 2022 and 29 March 2022 and was kept open longer than planned due to the March storm events across Sydney, which impacted the study area. 192 responses were lodged with 82 residents indicating they had experienced flooding in the past. Figure 5 shows a graphical summary of the responses received. The responses indicated that:

- The respondents are typically residents of the study area, with an owner-occupied property (92%) and a large proportion having resided in the area for more than 10 years (60% of respondents).
- Approximately 43% of respondents experienced some flooding on or outside their property.
- Of those that experienced flooding, approximately 45% experienced flooding in their backyard, with approximately 30% experiencing flooding in their front yard and on the road. Approximately 10% had experienced flooding within their building.
- Of those that experienced flooding, approximately 60% described the flooding as shallow (up to mid-calf level), indicating the dominant flood mechanism being overland flooding (i.e. not from an open channel or creek).
- Of those that experienced flooding, approximately 35% described the speed of flood waters as running pace, with approximately 40% stating that it was slower than this.
- Of those that experienced flooding, almost half said that it was overflow from neighbouring properties.
- There were 9 responses (from 8 respondents) indicating that a flood mark was available on their property.

The responses and comments submitted were analysed to ascertain information that would be useful for calibrating the flood model. There were 80 responses that contained information that was considered useful. These responses are shown in Figure 6, and a summary of each response is provided in Appendix E, which details the model calibration results.

Apart from residents with a creek at the rear of their property (or front, that is crossed by a driveway bridge/culvert), the majority of respondents described overland flow flood behaviour. A large proportion of these also mention blockage of street drainage pits as contributing to flooding. Many of the respondents did not mention a specific date of flooding. It is considered that the type of overland flooding observed by the community happens relatively frequently, when the capacity of the stormwater system is exceeded (which may be approximately a 20% AEP event).

The flood events mentioned by the community are varied, with some referencing floods in the

1970's and 1980's. Specific events that were mentioned by more than one resident are summarised in Table 6. There were also three respondents that indicated flooding happens frequently, on a yearly basis.

Table 6: Flood events mentioned by the community

Year	Comment
2022	March 2022 was mentioned in 3 responses, with one indicating the 8th. There were also 2 respondents that mentioned January 2022.
2021	March 2021 was mentioned in 2 responses, with one of them indicating the period 14th-23rd. There was also one respondent indicating December 2021. There were an additional 3 respondents indicating '2021', which is most likely to be referring to the March 2021 event.
2020	9 February 2020 was mentioned in 5 responses. December 2020 was also mentioned by 2 respondents, suggesting that there were two storm events that year. There were an additional 2 respondents indicating '2020', which may be referring to either event.
2016	January 2016 was mentioned by 2 respondents, with one indicating the 16th.
2010	January 2010 was mentioned by 2 respondents.

Photographs of flooding were provided by 8 residents, and these are shown in Photo 4 to Photo 14 below. The respondent ID is provided in the caption.

Photo 4: 1984 flooding of a workshop at the rear of a property on Binalong Street, Pymble (ID 076)

Photo 5: 6 February 2010 flood damage of a property on Doncaster Avenue, West Pymble (ID 111)

Photo 6: Photo of a wooden bridge in Sheldon Forest that was washed away in a flood in 2010 – 2012.

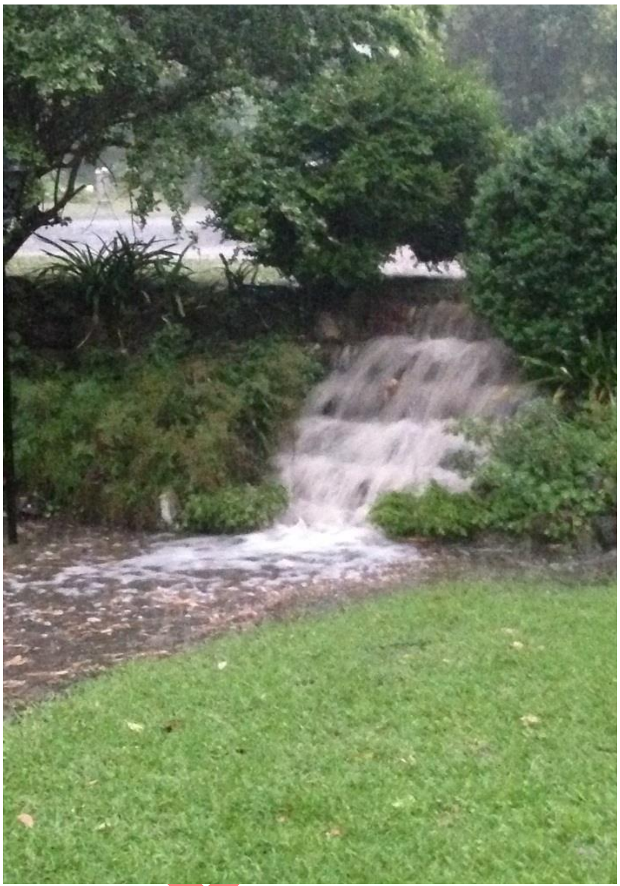


Photo 7: 3 December 2014 flooding through the front yard of a property on Campbell Drive, Wahroonga (ID 082)

Photo 8: 5 June 2016 Coups Creek flooding at the rear of a property on Strone Avenue, Wahroonga (ID 045)

Photo 9: Flooding (date unknown, perhaps 2016?) in the backyard of a property on Troon Place, Pymble (ID 184)

Photo 10: Flooding (date unknown, perhaps 2019?) in the backyard of a property on Troon Place, Pymble (ID 184)

Photo 11: 9 February 2020 flooding in the back yard of a property on Yarrara Road, West Pymble, caused primarily by sewer overflows (ID 061)

Photo 12: 9 February 2020 flooding in the Forwood Avenue sag point, Turramurra (ID 110)

Photo 13: 9 February 2020 Coups Creek flooding at the rear of a property on Strone Avenue, Wahroonga (ID 045)

Photo 14: March 2021 flood damage of a property on Forwood Avenue, Turramurra (ID 161)

4. HYDROLOGIC MODELLING

4.1. Background

A hydrological model is a computer-based software 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) are a means of measuring this information, but they are expensive to setup and maintain. They also require a long record (several decades) and measurements of the velocity of flow during flood events (known as gaugings) to be of most value for flood estimation.

Most of the smaller creeks in NSW are not gauged, and there are no suitable stream gauges within the study area. In this case, using a computer-based hydrologic model is the best practice method for determining how much flow occurs from rainfall information (which is more widely available from rain gauges). 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 Australian Rainfall and Runoff (ARR) 2019 (Reference 6). 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.

DRAINS is widely used throughout Australia to estimate runoff, primarily from urban areas, and was selected for use in this study. DRAINS also includes the ability to simulate the stormwater network, including pits, pipes and channels. While the representation of these elements is simplistic in nature, it can be used for rapid assessments of the stormwater network capacity. Further details regarding the DRAINS software are provided in the DRAINS User Manual (Reference 7).

4.2. Existing DRAINS Models

The existing DRAINS models, developed as part of Reference 1 and 2, were reviewed for their suitability for the Flood Study and were not considered to be adequate in their representation of the catchment conditions. This primarily related to the catchment-wide adoption of a standard pervious/impervious fraction and time of concentration regardless of the sub-catchment size or land use. Sub-catchment boundaries were not available and hence these parameters could not readily be updated. In consultation with Council, it was decided that the existing DRAINS models were to be updated for use in the Lane Cove Northern Catchments Flood Study.

4.3. Updated DRAINS Models

The updates made to the DRAINS models are outlined below. A separate memo describing the updates is also provided in Appendix C, which contains further information.

4.3.1. Sub-catchment Delineation

There are approximately 2,400 sub-catchments in the existing DRAINS models. It was not considered feasible to manually delineate each of these sub-catchments. Instead, a semi-automated approach was adopted using GIS techniques.

The terrain data used to delineate sub-catchments was based on the TUFLOW model. The TUFLOW model included topographic modifications to represent various hydraulic features. This included gutters, which are significant for the capture and directing of flows to individual pits. These gutters are generally not adequately represented even in the 1 m LiDAR grid. Consequently the gutters were enforced as continuous flow paths in the TUFLOW model. The terrain was also 'hydrologically treated' to fill sinks and provide continuous flow paths through the study area.

Pit inlet locations and nodes from the existing DRAINS models were exported into a GIS dataset. The dataset was filtered to leave only those nodes with sub-catchment inflows associated with them (i.e. removing 'junction' pits). This resulted in 2,394 nodes. Sub-catchments to each of these nodes were automatically delineated in GIS software. The sub-catchments were reviewed and node locations adjusted in order to best represent the catchment draining to each point. These sub-catchments were further 'cleaned' and manually adjusted (where required) to obtain a detailed sub-catchment network, as shown in Figure 7.

4.3.2. Sub-catchment Parameters

The 'detailed' catchment data approach was adopted for the updated DRAINS model. Sub-catchment areas were calculated within the GIS software, in addition to the longest flow path and slope. DRAINS allows the land use in each sub-catchment to be modelled in terms of an impervious fraction (termed 'paved', representing directly connected impervious areas), a pervious fraction (termed 'grassed') and a supplementary fraction (representing indirectly connected impervious areas). This approach is described in ARR 2019 (Reference 6). The TUFLOW model surface roughness was used to define the impervious fractions within each sub-catchment. The adopted impervious percentages for each land use type are provided in Table 7 and can be seen in Figure 8.

Table 7: Land use types and impervious fractions

Land Use Type	Paved (%)	Supplementary (%)	Grassed (%)
Road corridors[1]	80	0	20
Residential	40	10	50
Bushland / Vegetated	0	0	100
Grassed (e.g. parks)	0	5	95
Commercial	50	20	30
Waterways[2]	100	0	0

1. Including verges
2. Open water assumed to be 100% impervious

An additional time of 2 minutes was assigned for paved areas (representing concentration time from pervious areas such as roofs) and 1 minute for supplementary and grassed areas. The retardance coefficient (n*) of 0.015 was assigned for paved areas (representing concrete) and 0.1 for supplementary and grassed areas. These values were adopted based on consideration of guidance in the DRAINS manual. It should be noted that the retardance coefficient (n*) is similar to, but not the same as the traditional Manning's 'n' parameter.

The Horton/ILSAX hydrologic model was adopted, with the relevant parameters outlined in Table 8. Depression storage was assumed to be 1 mm for impervious and supplementary areas and 5 mm for pervious areas. A soil type of 3 was adopted, representing a slow infiltration rate.

Table 8: ILSAX Parameters Adopted

Parameter	Value
Paved area (impervious) depression storage (mm)	1
Supplementary area depression storage (mm)	1
Grassed (pervious) area depression storage (mm)	5
Soil Type	3
Overland Flow Approach	Kinematic Wave Equation

5. HYDRAULIC MODEL

5.1. Background

Hydraulic modelling is the simulation of how flow moves across the terrain. A hydraulic model can estimate the flood levels, depths, velocities and extents across the floodplain. It can also 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 flood modelling. It requires high resolution information about the topography, which is available for this study from the LiDAR aerial survey (Section 2.2.1). Various 2D software packages are available (SOBEK, TUFLOW, RMA-2, MIKE FLOOD). The TUFLOW package was adopted as it meets requirements for best practice and is currently the most widely used flood 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 2023-03-AB-w64 (using the finite volume HPC solver in single precision mode). The TUFLOW Heavily Parallelised Compute (HPC) solver can run on a Graphics Processor Unit (GPU) and are significantly faster than TUFLOW Classic models, which rely on a Central Processing Unit (CPU). HPC models can be run across thousands of cores within a GPU. This scheme is also more robust, being a finite volume scheme. Further details regarding TUFLOW software can be found in the User Manual (Reference 8).

In TUFLOW the ground topography is represented as a uniform grid with ground elevations and Mannings 'n' roughness value assigned to each grid cell. The grid size 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. TUFLOW also has the ability to dynamically link to the 1D ESTRY engine, making it useful for simulating both overland flow in the 2D domain, and flows through underground pipes and culverts in the 1D domain.

5.2. Hydraulic Model Extent and Resolution

The hydraulic model covers the entire study area including the suburbs of West Pymble, Pymble, Turramurra, South Turramurra and Wahroonga that are within the Lane Cove River catchment (see Figure 9). The model extends from the catchment boundary (approximately the Pacific Highway and railway line) down to the Lane Cove River. The Lane Cove River itself is not contained in the model. A small portion of the model covers the Coups Creek catchment within Hornsby Shire Council LGA (Normanhurst). The model covers an area of approximately 18.2 km^2 and utilises a 1 m by 1 m grid resolution. This resolution fully utilises the LiDAR data, which is also a 1 m by 1 m grid.

5.3. Model Topography

The hydraulic model reliability is heavily dependent on the underlying terrain data. The 2D topography for the TUFLOW model was based on the 1 m DEM derived from LiDAR data, captured in May/June 2020 (see Section 2.2.1 for details, shown in Figure 2). The LiDAR data is most accurate on hard surfaces exposed to the sky (for example roads). The data is less accurate where there is dense vegetation covering the ground (such as within vegetated creek channels). Flow paths within urban areas are typically along streets, and the vegetated creeks within the study area are typically located downstream of the urban areas. For several areas where vegetated creeks traverse urban areas, detailed topographic survey was obtained (see Section 2.2.2). The cross-section information in this survey was used to modify the terrain in the TUFLOW model to represent these channels as part of the 2D domain. Additional topographic modifications were made to the terrain to ensure correct representation of hydraulic features, as outlined in the following sections.

5.4. Hydraulic Structures

5.4.1. Road Kerbs and Gutters

The road gutter network plays a key role for overland flow in the urbanised parts of the study area. Roadways typically capture the runoff from properties and convey flow within the gutter. Representation of the kerb and gutter system in the model is therefore an important feature to accurately simulate overland flows.

LiDAR typically does not have sufficient resolution to adequately define the kerb and gutter system within roadways. The kerb/gutter feature is of a scale that is smaller than the underlying 1 m LiDAR grid used in the model, and even use of the LiDAR return points does not pick up a continuous line of low points defining the drainage line along the edge of the kerb. Project 15 of ARR 2019 – *Two Dimensional Modelling in Urban and Rural Floodplains* (Reference 9) provides the following guidance:

> *"Stamping a preferred flow path into a model grid/mesh (at the location of the physical kerb/gutter system) may produce more realistic model results, particularly with respect to smaller flood events that are of similar magnitude to the design capacity of the kerb and gutter. Stamping of the kerb/gutter alignment begins by digitising the kerb and gutter interval in a GIS environment. This interval is then used to select the model grid/mesh elements that it overlays in such a way that a connected flow path is selected (i.e. element linkage is orthogonal). These selected elements may then be lowered relative to the remaining grid/mesh."*

In order to model the road drainage system effectively, the gutters were stamped into the mesh using the method described above. The pavement layer provided by KRGC was used to create a gutter line which was reviewed using aerial imagery and modified where required to ensure a reasonable alignment. A total of 186 km of gutter lines were included in the model.

The method used was to inspect the LiDAR by automatically generating points a maximum of every 4 metres along the gutter lines and sampling the lowest elevation value within a 1 metre radius of the point. The elevation of those points was lowered by 0.1 m to simulate a continuous flow path in the gutter. Checks of the resulting gutter inverts were undertaken and it was found that in some locations the GIS algorithm had picked up a minimum elevation that was substantially lower than the road. This occurred in areas where the road was adjacent to steep cliffs. Where this was the case, the elevation of the gutter was revised accordingly.

5.4.2. Pits and Pipes

Urban areas that are developed over natural watercourses typically have a drainage system that consists of pit inlets to capture surface water and a pipe system to convey that water underground to a downstream outlet location, such as a natural creek. The hydraulic model incorporates this stormwater, or 'pit and pipe' network.

The stormwater drainage network was modelled in TUFLOW as a 1D network dynamically linked to the 2D overland flow domain. This stormwater network includes conduits such as pipes / box culverts, and stormwater pits including inlet pits and junction manholes. The schematisation of the stormwater network was undertaken using the stormwater GIS layers supplied by KRGC (see Section 2.5). Pipe sizes were obtained from the GIS layer, along with pit inlet sizes. The majority of pits contained information about the invert level (such as a grate level and pit depth), however, where no invert elevation was available, an estimate was made based on the upstream/downstream invert, the size of the pipe and the LiDAR elevation. For the Quarry Creek catchment, the existing TUFLOW model (Reference 5) stormwater layers were utilised. The alignment of the pipes and location of the stormwater pits were reviewed against the provided aerial imagery and no discrepancies were found. Figure 10 shows the location of pits and pipes included as 1D elements in the hydraulic model and a summary of pipes by size is provided in Table 9. A total of 2,139 pits were included in the model.

Table 9: Stormwater pipes included in the model

Pipe Size (mm)	Number of Pipes in Model
< 375	96
375	1,245
450	473
525	125
600	112
675	41
750	73
825	18
900	187
Total	**2,370**

5.4.3. Culverts

Cross drainage culverts were identified in the KRGC stormwater database (see Section 2.5) and included in the hydraulic model as 1D elements. The alignment, size and structure invert were reviewed against the available aerial imagery and LiDAR data. The location of headwalls was modified to align with the terrain low points. Local terrain modifications were made at culvert inlets and outlets to ensure the transfer of water between the 2D domain and 1D culverts. Culverts included in the existing Quarry Creek model (Reference 5) were used in the current model.

There were nine culverts surveyed as part of the detailed topographic survey (see Section 2.2.2). These culverts were included in the model based on the survey information (dimensions and invert levels). These structures were either not included in Council's asset database or contained different information to that surveyed. An example of one of these culverts is a large box culvert under St Andrews Drive, Pymble (Photo 15). The culverts included in the model are shown in Figure 10.

Photo 15: St Andrews Place Culvert (Degotardi Smith & Partners)

5.4.4. Bridges

There is one substantial bridge structure within the study area, located on Troon Place, Pymble. This bridge was subject to a detailed survey (see Section 2.2.2) and the structure was included in the 2D domain of the hydraulic model. The bridge is a clear spanning structure, and as such form losses were only applied from the soffit of the super structure. Losses were applied in accordance with recent research from TUFLOW (Reference 10). The Troon Place bridge is shown in Photo 16.

Photo 16: Troon Place Bridge (Degotardi Smith & Partners)

There are numerous other bridge structures that span minor waterways throughout the study area. These are typically small single-spanning footbridges, such as those shown in Photo 17. These bridges are likely to only have very localised hydraulic effects and there are too many to quantify and model at the catchment-wide scale. The hydraulic effects would only be initiated when the flow comes into contact with the structure. At this level, the flow breaks out of the main channel and the overbank flow is of more concern rather than the specifics of the flow within the channel and any interaction with small structures.

Photo 17: Example Footbridges (Degotardi Smith & Partners)

There are several areas where dwellings are located at the rear of the property and a creek runs through the front of the property, with driveway access provided over the creek. For these lots, due to the proximity of the creek to the dwelling and the size of the vehicle crossings, they have been included in the model. Three driveway crossings off Quadrant Close were included and seven driveway crossings off Wyomee Avenue were included. An example of these structures is shown in Photo 18.

Photo 18: Example Driveway Bridge

5.4.5. Channels

As outlined in Section 5.3, the topographic survey obtained for channels within urban areas was incorporated into the 2D domain. The cross sections were used to generate topographic modifications to represent the channels. The surveyed points were used to represent the channel invert and width, ensuring that the bank levels aligned with the LiDAR data outside of the channel. Typical channels that were surveyed can be seen in Photo 19.

Photo 19: Typical channels that were surveyed (Degotardi Smith & Partners)

There is one concrete lined channel upstream of Norman Griffiths Oval (east of Lofberg Road) which was represented as a 1D channel in the existing TUFLOW model (Reference 5). This channel was retained in the current model.

For the remaining natural creek channels within the study area, breaklines were used to represent the channel inverts. The lowest elevations from the LiDAR data were sampled at a 20 m interval to generate a breakline to ensure a continuous flow path is represented along the creek. This was done for all major natural creek channels in the study area that were not subject to detailed survey. For Quarry Creek, the channel topography downstream of Norman Griffiths Oval developed as part of the existing TUFLOW model (Reference 5) was used. The channels included in the model are shown in Figure 10.

5.4.6. Buildings

Buildings in overland flow paths can have a significant influence on surrounding flood levels and can redirect floodwater. Buildings and other significant features likely to obstruct flow were incorporated into the model based on a building footprint layer provided by KRGC (see Section 2.6). The building footprints were reviewed using aerial imagery and Google Street View in key overland flow areas, and modified where required. Approximately 8,000 buildings were included in the model and are shown in Figure 11.

5.4.7. Fences

Smaller localised obstructions (such as fences) can be represented in TUFLOW in several ways including as impermeable obstructions, a percentage blockage or as an energy loss. The obstructions may also be approximated generally by increasing Mannings roughness for certain land use areas (such as residential) to represent the typical type of fencing used in such areas.

Individual fences in the catchment were not explicitly modelled, as they are difficult to identify and relatively impermanent (since people can change their fences without Council approval). Fences in urbanised areas were therefore accounted for by applying a slightly higher Mannings roughness for the residential land-use type to simulate the obstruction to flow.

5.4.8. Avondale Dam

Avondale Dam is a small dam located on Avondale Creek, upstream of the Comenarra Parkway in bushland adjacent to the Avondale Golf Club. It is assumed that this was an old water supply dam, however, it is no longer operational. No details of the dam were able to be obtained. For the purpose of flood modelling, a dam wall was included in the 2D topography and the dam was assumed to be full, such that any inflow into the dam would overtop the dam wall and continue to flow along Avondale Creek.

5.4.9. Norman Griffiths Oval

It is understood that the Normal Griffiths Oval at West Pymble is currently being upgraded. The upgrades include a new synthetic surface, lighting, pathways and an improved drainage system. The existing oval configuration was retained for the calibration events, however, for design flood events, the proposed oval upgrades were included in the model, as per information provided in Reference 11. For the purpose of flood modelling, these upgrades consisted of:

- Diversion structure to take flow from the 4 x 750 mm diameter pipes under Lofberg Road into new twin 600 mm diameter pipes that run underneath the oval, with the existing 1050 mm pipe under the oval also being retained;
- An underground 'detention basin' structure consisting of porous chambers in an aggregate fill that allows water to fill the chambers and void space in the aggregate fill. The stage-storage relationship was implemented based on DRAINS modelling that was undertaken for the design (Appendix 9 in Reference 11); and
- An outlet structure that consists of a 375 mm diameter pipe that discharges into the existing 1050 mm diameter pipe, noting that the pipe is actually 825 mm in diameter at the point it exits from under the field, with this constriction included in the model.

The configuration of the proposed system is shown in Diagram 5 and Diagram 5, with the existing pipe constriction shown in Diagram 6.

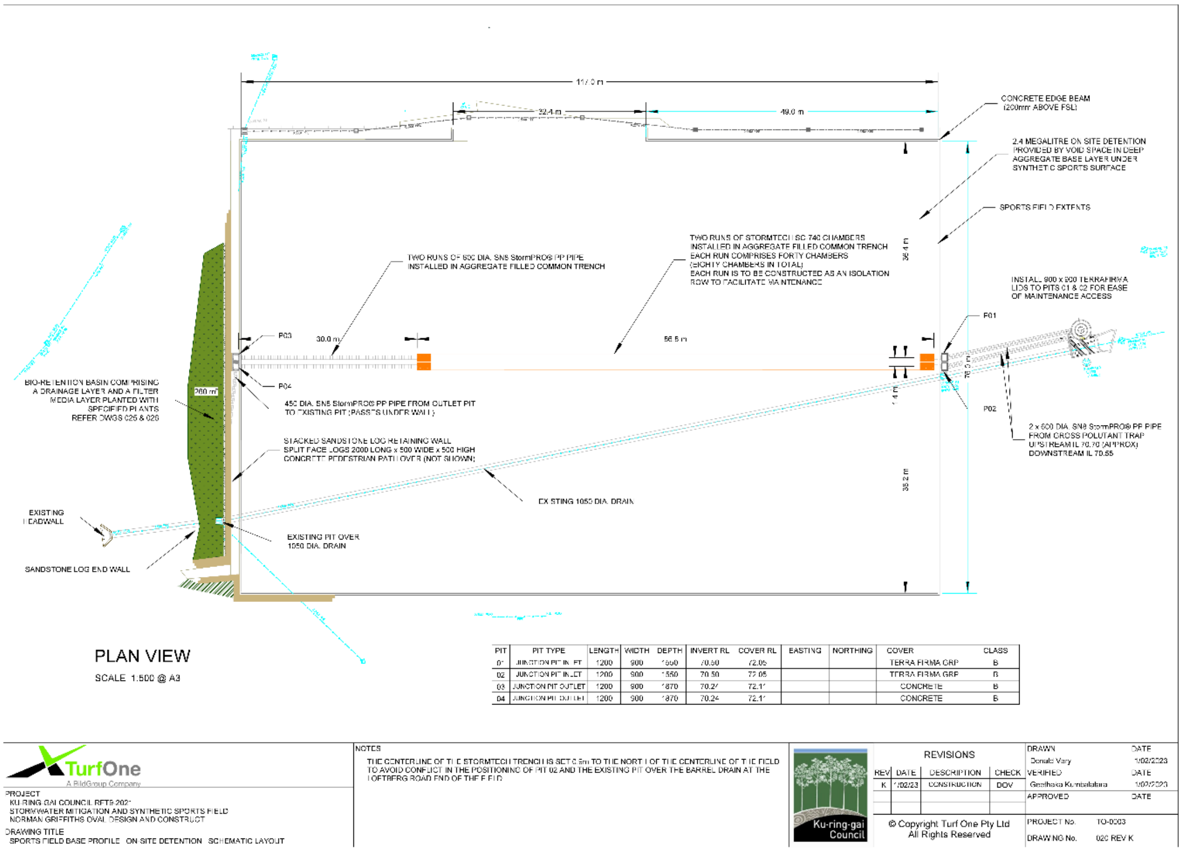


Diagram 4: Norman Griffiths Oval Stormwater Upgrades – Plan View (Reference 11)

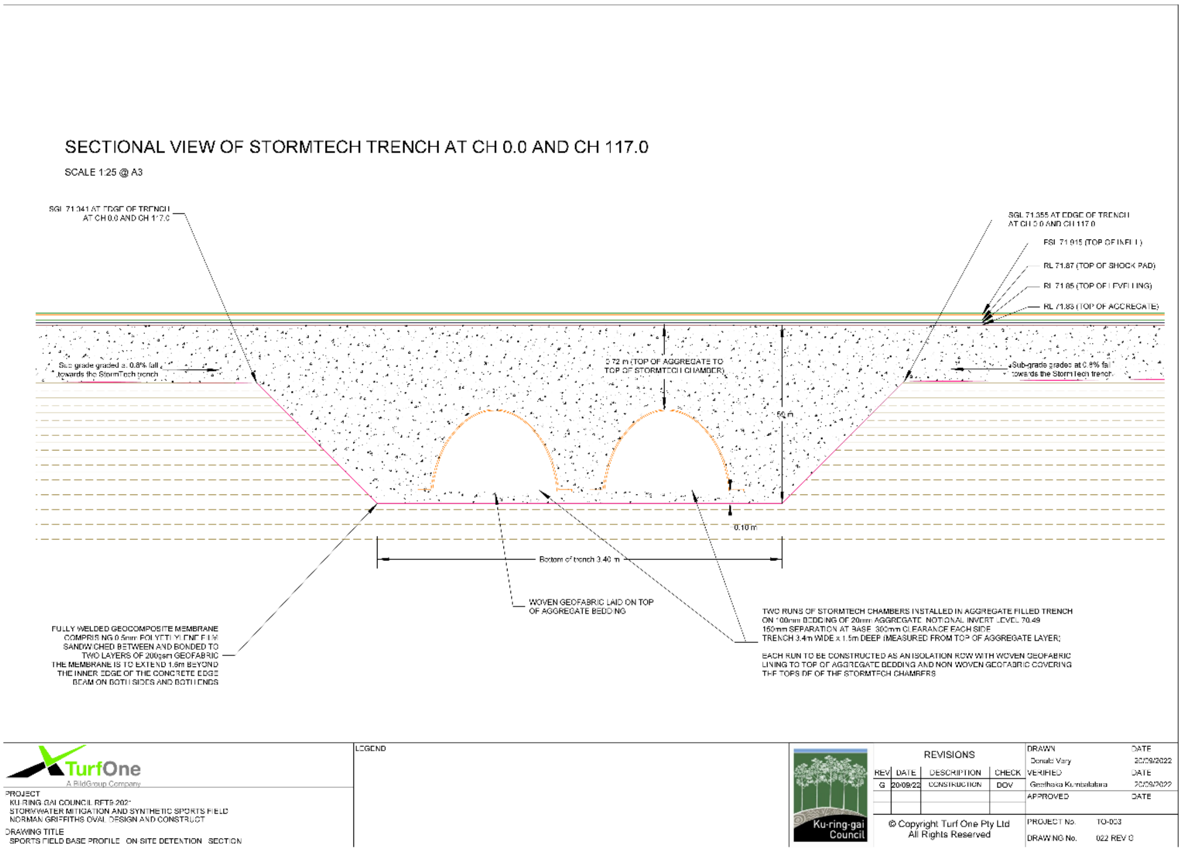


Diagram 5: Norman Griffiths Oval Stormwater Upgrades – Section View (Reference 11)

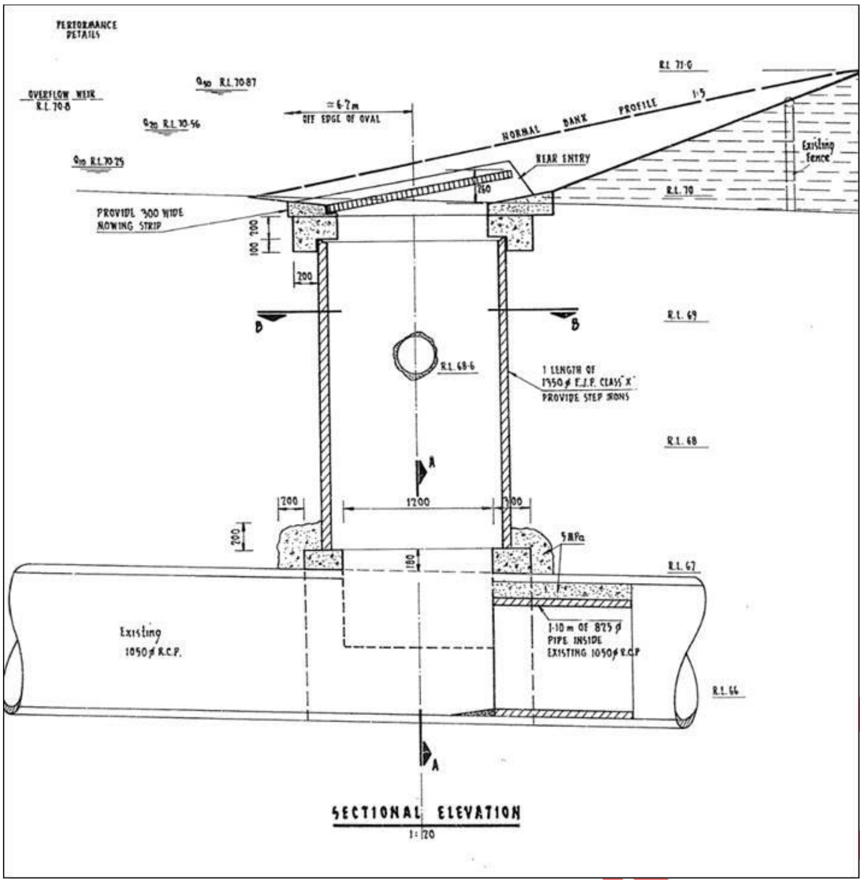


Diagram 6: Existing Norman Griffiths Oval 1050 mm diameter pipe with 825 mm constriction

5.5. Hydraulic Roughness

Roughness, represented by the Mannings 'n' coefficient, is a key parameter in hydraulic modelling. It models the resistance that floodwaters experience when flowing over a surface. For example, floodwater flows more easily over a concrete carpark surface than through dense vegetation in a natural creek channel. As part of the calibration process, roughness values are adjusted within the ranges defined in the literature so that the model better matches observed peak flood levels at a variety of locations. The typical ranges of the Mannings 'n' coefficient for different surface types are discussed in Project 15 of ARR 2019 – *Two Dimensional Modelling Urban and Rural Floodplains* (Reference 9). Chow (Reference 12) also provides some information with regards to the setting of the roughness values for hydraulic calculations.

The Mannings 'n' values adopted for the study area are shown in Table 10. These values have been adopted based on the site inspection, past experience in similar floodplain environments, consideration of the above references and the model calibration process. The spatial variation in Mannings 'n' is shown in Figure 11. The land use planning layer was used to determine the land use type across the study area and was reviewed against the available aerial imagery. The creek channel extents were derived using preliminary TUFLOW results, which indicated the areas of conveyance and relatively deep floodwaters (over a metre) that is associated with creek channels.

Table 10: Mannings 'n' values adopted in the TUFLOW model

Land Type	Mannings 'n'
Grass and open space	0.04
Dense vegetation and bushland	0.15
Creek channel	0.05
Road corridor	0.03
Residential areas	0.06
Commercial and Industrial	0.025

5.6. Boundary Conditions

5.6.1. Inflows

The DRAINS hydrologic model (Section 4) simulates the runoff that occurs for a particular rainfall event. Local runoff hydrographs are produced for each sub-catchment area. These hydrographs are applied at the downstream end of each sub-catchment, within the TUFLOW 2D domain (see Figure 9). These inflow locations correspond with stormwater inlet pits (1,712 inflow points) or drainage reserves and open watercourses (686 inflow points). These inflow points typically receive inter-allotment drainage and sheet runoff flows from upstream catchment areas. Flows for three sub-catchments were distributed across seven inflow points, to obtain better resolution of flooding within the sub-catchment. The flows were assigned based on the proportion of the sub-catchment area that drains to each inflow location.

5.6.2. Downstream Boundary Condition

The western edge of the study area is located along the Lane Cove River. This is located within the Lane Cove National Park or other heavily forested areas of the Ku-ring-gai LGA. The boundaries for each of the creeks discharging into the Lane Cove River were based on a stage-discharge curve, adopting a 1% slope. This allows water to flow out of the model, assuming a normal flow depth. There is no development in this lower section of the creeks and the flooding conditions on private properties are not considered to be sensitive to the assumed tailwater condition. These downstream boundaries are shown in Figure 9.

6. MODEL CALIBRATION

6.1. Approach

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.

The choice of calibration events for flood modelling depends on a combination of the severity of the flood event and the quality of the data available. Ideally, data would be available from streamflow and rainfall gauges in addition to records of flood marks or inundation extents. Typically, in urban catchments both gauge records and reliable calibration information is lacking. The following limitations prevent a comprehensive calibration of the hydrologic and hydraulic models for this study.

- There is only a limited amount of historical flood information available for the study area. There are no stream flow gauges and much of the flood information is based on flood observations and estimates of flood depths, etc. rather than surveyed flood marks.
- Rainfall records and particularly pluviometer records for past floods within the catchment are limited. Rain gauges are sparsely distributed and may not accurately capture the spatial and temporal distribution of rainfall during the storm event.
- Changes to the catchment over time due to urban development may result in significant changes to land uses and drainage structures. The models have been developed for current conditions, and the simulation of historic events too far into the past may not be accurate due to changes in the catchment.

These limitations are typical of the majority of urban catchments across Sydney and the calibration exercise undertaken here constitutes recommended practice as outlined in Reference 9. This involves a 'validation' of the models based on what data is available, rather than a detailed 'calibration'. In lieu of this, a detailed sensitivity analysis (Section 9) is undertaken to understand how variations to the adopted model parameters influence the modelled flood behaviour.

6.2. Summary of Historical Event Rainfall Data

6.2.1. The 10th of June 1991 Storm Event

The storm of 10th of June 1991 impacted the study area and a spatial analysis of the daily rainfall gauges was undertaken, using daily and sub-daily rainfall recorded at 9am on the 11th of June 1991. This is shown in Figure 12. This indicates that the storm was more intense across the northern half of the study area (Wahroonga and Turramurra) with over 220 mm of rain falling within a 24-hour period, while the southern part of the study area received between 195 mm and 220 mm of rain. An analysis of the nearby sub-daily rainfall data was also undertaken, and indicated a reasonably consistent rainfall from 9 am on the 10th to 3 am on the 11th, as shown in Figure 13. The peak rainfall burst occurred at approximately 7 pm on the 10th. The sub-daily rainfall records were compared to the ARR 2019 design Intensity Frequency Durations (IFDs) at the study area centroid with the comparison shown in Figure 14. This indicates that the rainfall burst for durations above approximately 6 hours was between a 5% AEP and 2% AEP.

6.2.2. The 9th of February 2020 Storm Event

An analysis of the community consultation responses (see Section 3), showed that the February 2020 storm was the most commonly identified by the community. The 24-hour rainfall totals for the 2020 event (recorded at 9am on 10th of February) are fairly consistent across the study area as shown in Figure 15. The 24-hour rainfall depth is between 170 mm and 210 mm across the study area, with the eastern portion of the site receiving the highest rainfalls, reducing to a minimum of 170 mm at the Lane Cove River on the western boundary. The sub-daily rainfall records show a similar temporal pattern for all three nearby gauges, with Pymble Bowling Club recording a rainfall total close to 200 mm while the Pennant Hills Bowling Club recorded approximately 160 mm, as shown in Figure 16. Approximately 65% of the rainfall volume fell in the first 6 hours of the storm. A comparison of the sub-daily rainfall records with ARR 2019 IFD at the catchment centroid shows a similar peak burst intensity at 6 hours as a 2% AEP event at the Pymble gauge, while the other two gauges are between a 10% and 2% AEP for the same duration, as shown in Figure 17.

6.2.3. The 20th of March 2021 Storm Event

The March 2021 storm event was mentioned by 2 residents, with one indicating the 14th – 23rd as the dates. Daily rainfall totals were analysed and the daily rainfall recorded to 9 am on the 21st March 2021 was the largest daily total. The 24-hour rainfall totals for this event vary across the study area, with Pymble recording over 120 mm while just 70 mm was recorded at Wahroonga, as shown in Figure 18. The sub-daily rainfall records show a similar temporal pattern for all three nearby gauges, with Pymble Bowling Club recording the highest rainfall total and the Pennant Hills Bowling Club and North Epping gauges recording lower (and very similar) rainfalls, as shown in Figure 19. A comparison of the sub-daily rainfall records with ARR 2019 IFD at the catchment centroid shows the rainfalls for this event were less than a 20% AEP event, as shown in Figure 20.

6.2.4. The 8th of March 2022 Storm Event

During the community consultation phase of this study, the 8th of March 2022 storm hit Sydney. There were widespread reports of flash flooding, although only 3 responses for the community consultation for this study identified this storm as a particular issue. The rainfall total to 9 am on 9th March 2022 was used to analyse the spatial distribution of the rainfall event, with the results shown in Figure 21. The rainfall varies across the study area, following the gradient of the land. The highest rainfall totals were around Pymble (approximately 150 mm), reducing to around 90 mm in the vicinity of the Lane Cove River. The distribution of this rainfall over time was fairly consistent across the nearby sub-daily rainfall gauges, as shown in Figure 22, although the rainfall totals varied significantly. A comparison of the sub-daily rainfall records and the ARR 2019 IFD at the study area centroid indicated that the Pymble Bowling Club gauge may have reached between a 5% AEP and 2% AEP storm for durations between 2 hours and 7 hours, while the other two gauges, with much lower rainfall totals, were more frequent than the 20% AEP. This is shown in Figure 23.

6.3. Recorded Flood Observations

As part of the community consultation phase of this project (see Section 3), residents shared their knowledge and photos of flooding in the study area. Of the 192 responses, 80 were considered to be useful for the purposes of calibration, with descriptions of flood behaviour and/or photographs.

The responses selected for the calibration process contained information related to flooding that could be verified in the model. This information related to flood behaviour observations such as depths, extents and the source of the water. These observations are not considered to be of an accuracy that would warrant detailed model calibration, however, can be used to validate the modelled flood behaviour. The location of these responses is shown in Figure 6. Residents were also able to upload photographs of flooding in their questionnaire response, and these were also used to validate the model.

6.4. Hydraulic Model Validation

The historic rainfall events were simulated in the DRAINS hydrologic model using the spatial distributions of rainfall presented in Figure 12, Figure 15, Figure 18 and Figure 21 for the June 1991, February 2020, March 2021 and March 2022 storm events. The rainfall total for each of the DRAINS sub-catchments was sampled from the rainfall grid produced. A rainfall multiplier was used within DRAINS for each sub-catchment to scale the total rainfall.

This rainfall depth was distributed temporally using the available sub-daily rainfall data, with the Pymble Bowling Club gauge being adopted for each of the calibration events. It was observed that the temporal pattern did not change significantly between the three nearby gauges for the calibration events. There was not sufficient data to calibrate the hydrologic model or rainfall losses. As such, along with the parameters outlined in Section 4.3.2, an antecedent moisture condition of 3 was adopted for each of the calibration events, representing 'rather wet' conditions, or approximately 12.5 mm to 25 mm of rainfall in the 5 days preceding the storm (Reference 7).

The resultant runoff hydrographs produced by the DRAINS model were then applied to the TUFLOW hydraulic model to simulate the flood behaviour for the duration of each event (24 hours). In the TUFLOW model, it was assumed that stormwater pits were blocked by 20% for on-grade pits and 50% for sag pits. Culverts with headwalls were assumed to be blocked by 20%. These blockage factors were applied due to the high number of residents that mentioned blockage as a factor in recent flood observations. The model results were then compared to the respondent's photographs and comments in order to validate the model. The modelled flood depths for the 1991, 2020, 2021 and 2022 events can be seen in Figure 24, Figure 25, Figure 26 and Figure 27, respectively.

6.4.1. Validation to Photographs

The photography received as part of the community consultation were compared to the model results, where the photographed flood event was simulated. The modelled depth and flood extent in Image 1 to Image 3 match the observed flood behaviour in Photo 20 to Photo 22. The colour scale used for the modelled depth is shown in Diagram 7.

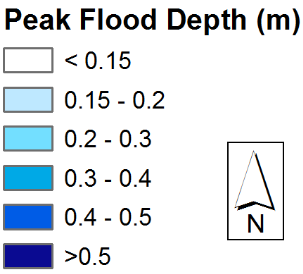


Diagram 7: Colour scale used for flood depth images

Photo 20: 9 February 2020 flooding in the back yard of a property on Yarrara Road, West Pymble, caused primarily by sewer overflows (ID 061)

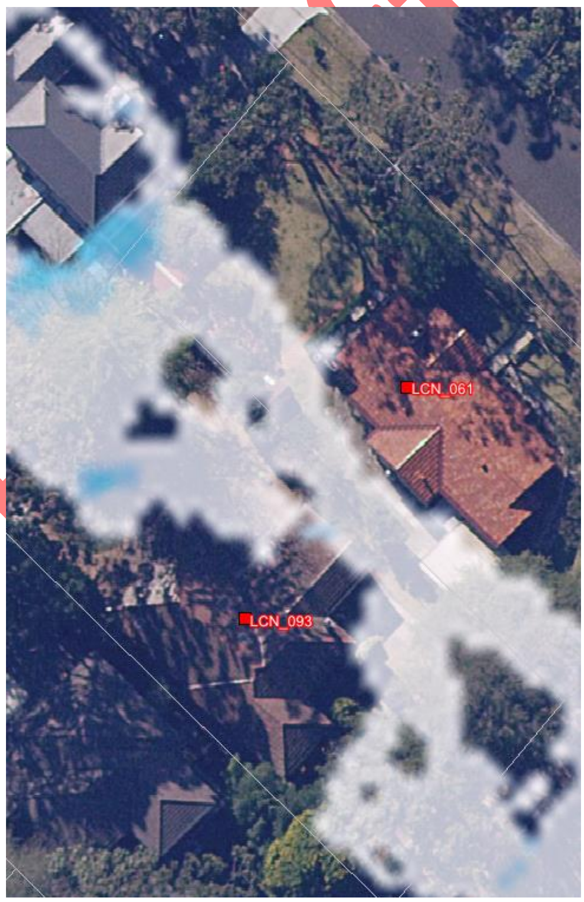


Image 1: Hydraulic model results at Yarrara Road, West Pymble in February 2020 event.

Photo 21: 9 February 2020 flooding in the Forwood Avenue sag point, Turramurra (ID 110)

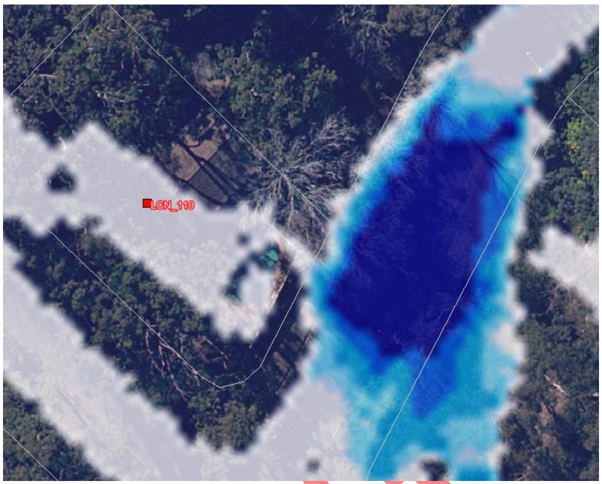


Image 2: Hydraulic model results at Forwood Avenue, Turramurra in February 2020 event.

Photo 22: 9 February 2020 Coups Creek flooding at the rear of a property on Strone Avenue, Wahroonga (ID 045)

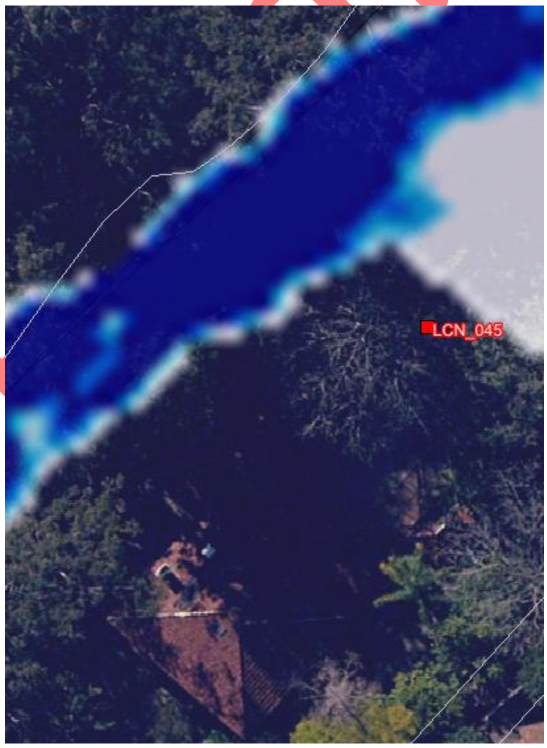


Image 3: Hydraulic model results at Strone Avenue, Wahroonga in February 2020 event.

6.4.2. Validation to Observations

A comparison between flood observations reported by the community and the modelled flood behaviour is provided in Table E1 in Appendix E. The modelled peak flood depth on the property (or road, if the road is the subject of the response) is provided for each of the historic events, with a comparison to the modelled flood behaviour. In most cases, the modelled peak flood depth on a property is recorded in an isolated area, such as adjacent to a building where high velocity flows may be stopped by the building and result in higher flood depths as water builds up before flowing around the building. Where this is the case, it was noted in the comments as the modelled peak flood depths across the property may be more representative of the observed flood depths than immediately adjacent to a building or some other isolated location.

The modelled flood behaviour's match to the observed flood behaviour is given along with a comment. The following categories are used to indicate how good the match is:

- No: This indicates that no match was obtained. This is primarily due to the observations relating to local catchment runoff that have not been modelled, and hence a match is not possible. In some cases, judgement has been used to assess whether the observed flooding is local runoff or not.
- Poor: This indicates that a poor match was achieved to observed flood depths or extents. The difference between the modelled and observed flood depths may be ±0.3 m or more, or flooding on the property may not be modelled at all. Possible reasons for the poor match are provided.
- Fair: This indicates that a fair match was achieved. The observed flood behaviour was generally replicated, however, the modelled flood depths may not match the observed depths (although typically within ±0.2 m) and further reasoning is provided.
- Good: This indicates that a good match was achieved. The modelled flood behaviour matches the observation well, with flood depths typically within ±0.1 m of the observed depths and the area of affectation replicated.

Overall, the comparisons indicate that generally a good match was achieved. In some cases, where a fair match was obtained, the observations were not detailed enough to identify exactly why the flood behaviour was not replicated exactly. In some cases the topography did not support the observations or it was thought that 'local drainage' runoff may be a factor in observed inundation. There are also highly localised features (such as private bridge crossings) that have not been modelled that may have a localised influence on flood behaviour. Overall, the model was considered to represent flood behaviour across the study area at the catchment-scale that the model is intended to represent. Major flow paths, creeks and flood problems areas have all been represented in the model.

7. DESIGN FLOOD EVENT MODELLING

7.1. Overview

ARR 2019 guidelines (Reference 6) for design flood modelling were adopted for this study. The new guidelines were first published in 2016, finalised in 2019 and present a significant update on the previous version published in 1987 (Reference 13). Since 1987, there have been numerous advances in the understanding of rainfall-runoff processes, technological advances and a larger set of recorded rainfall data available. This additional 30 years of data (from approximately 1985 to 2015), particularly for continuously recorded rainfall (pluviometers), allows for Australia-specific techniques and regionalised information to be used across the country. Specifically related to design flood modelling there is updated IFD information, design temporal patterns, areal reduction factors and rainfall losses to consider.

ARR 2019 guidelines were used to estimate the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP events. The PMF flows were derived using the BoM's Generalised Short Duration Method (GSDM, Reference 14) to estimate the probable maximum precipitation (PMP). A detailed critical duration analysis was undertaken to determine the most representative duration and temporal pattern across the catchment. The selected storm for each AEP event was then used to simulate the design flood behaviour.

This section outlines the design flood inputs and parameters that were used and the critical duration analysis.

7.2. IFD

7.2.1. Design Rainfall Depths

IFD information was obtained from the BoM using the 2016 rainfall data. It was noted that the IFD varies slightly across the study area, with higher design rainfalls occurring along the north eastern boundary (around Wahroonga, Warrawee and Turramurra) and lower rainfalls occurring along the western boundary (Lane Cove River). The difference between the maximum and minimum rainfall depths is approximately 10% across a range of AEPs and durations inspected. Due to the gridding of the IFD data, the very northern portion of the catchment (near Wahroonga) has the highest rainfalls and a small portion of the catchment (near South Turramurra) has the lowest rainfalls. The majority of the catchment, however, has a similar rainfall. A summary of rainfall depths at the catchment centroid (near the boundary of Turramurra and South Turramurra, at the Comenarra Parkway) is provided in Table 11, noting that the rainfall depth for any particular sub-catchment may vary within ±5% of this. The design rainfall at the centroid is representative of rainfalls across most of the catchment and was adopted for all the sub-catchments across the study area in the DRAINS model.

Table 11: Design rainfall depths (mm) at the centroid of the Lane Cove Northern Catchments study area

Duration (min)	AEP						
	20%	10%	5%	2%	1%	0.5%	0.2%
10	19.1	22.7	26.4	31.3	35.2	37.7	42.6
20	27.4	32.5	37.8	44.8	50.4	54.0	61.1
30	32.3	38.5	44.7	53.1	59.8	64.4	72.8
45	37.4	44.6	51.8	61.8	69.9	75.4	85.3
60	41.3	49.2	57.2	68.5	77.6	83.8	94.9
90	47.3	56.5	65.9	79.2	90.1	97.2	110
120	52.3	62.5	73.1	88.1	100	108	122
180	60.9	73.0	85.6	104	118	127	143
270	72.0	86.6	102	123	141	151	170
360	81.9	98.7	116	141	162	172	194

7.2.2. PMP Rainfall Depths

The design rainfalls for the PMP were derived using the BoM's GSDM (Reference 14). The catchment terrain was estimated to be 'rough' with an elevation adjustment factor of 1 and a moisture adjustment factor of 0.7. The GSDM requires rainfall to be distributed spatially using ellipses. Ellipse 'A', at the centre, has an area of 2.6 km^2 and represents the region of highest rainfall. Given the nature of the study area and the focus on overland flow paths through urban areas, it was assumed that all the flow paths and creeks of interest would have an upstream catchment area less than this, and as such the ellipse 'A' rainfall was applied to all sub-catchments. Even the largest catchment draining to the Comenarra Parkway (Avondale Creek) was estimated to be approximately 2.4 km^2, less than the Ellipse A area of 2.6 km^2. It would only be in the downstream forested reaches of the major creeks that this approach may overestimate the rainfall runoff using this approach.

7.2.3. At-site Rainfall Analysis

An at-site rainfall frequency analysis was undertaken to compare with the BoM's design rainfall data. This was undertaken to assess if there was any significant bias between them. It is not unexpected that individual gauges will show some bias as the approach used to pool data for the 2016 IFD estimates will generally be more correct than any one individual gauge, since it draws on information from surrounding gauges as well. While this process increases the certainty of estimates, it can cause over-smoothing of site data and introduce bias in locations where there are steep rainfall gradients. Based on past experience, it is only those areas with steep escarpments close to the coast that the 2016 IFD's have been shown to vary from the information at local rainfall gauges.

This analysis was undertaken for the closest three pluviometer gauges (see Section 2.8.3): Pymble Bowling Club (566073), Pennant Hills Bowling Club (566076) and North Epping (566083). These gauges are owned by SW and are not located within the study area, however, are close to the boundary of the study area. The annual maximum rainfall for a range of durations from 5 minutes to 2 hours was extracted from the available data from each gauge to produce an annual maximum series (AMS) for each duration. The derived AMS was subject to some filtering, however, it is difficult to undertake a rigorous filtering of sub-daily rainfall data. In particular, the AMS should:

- Contain the maximum recorded rainfall in each year (for the duration of interest). Due to the nature of sub-daily rainfall records, it is difficult to determine if the maximum was actually captured without cross-checking with other nearby gauges (Reference 15).
- Sub-daily rainfall records are also difficult to quality control, meaning that there may be erroneously high rainfall recorded over short periods of time. Continuous rainfall stations are more susceptible to malfunction than the standard daily rain gauge (Reference 15). Erroneous rainfalls, which are difficult to detect, may enter the AMS.
- Have a long enough record for extraction of statistically significant rainfall maximums. The BoM adopted rainfall stations with more than 8 years of data (Reference 15).

The following rainfall records were adopted for the AMS:

- For Pymble Bowling Club (566073), the rainfall record from 1988 to 2021 (inclusive) was adopted (34 years).
- For Pennant Hills Bowling Club (566076), the rainfall record from 1990 to 2021 (inclusive) was adopted, removing the years 2001 to 2013 (inclusive) due to nil data being recorded. This resulted in 19 years of data in the AMS from a discontinuous record.
- For North Epping (566083), the rainfall record from 1991 to 2021 (inclusive) was adopted, removing the years 2001 to 2013 (inclusive) due to nil data being recorded. This resulted in 18 years of data in the AMS from a discontinuous record.

The Cunnane plotting position was applied to the AMS, which is typically undertaken to fit a probability distribution to a dataset. The AMS was then plotted against the 2016 IFD data, obtained for the location of the rainfall gauge. The results are shown in Figure 28, Figure 29 and Figure 30 for the Pymble Bowling Club, Pennant Hills Bowling Club and North Epping Gauges, respectively. The results indicate the 2016 IFDs are generally higher than the at-site record for a 5 minute duration at each of the gauges. At the 15 minute duration, the at-site data is very similar to the IFD's, with no bias present at the two bowling club gauges. Above the 15 minute duration there is a slight bias for the at-site data to be slightly higher than the IFDs at these gauges. For the North Epping gauge, the bias for the at-site data to be slightly under the IFD data persists through to the 90 minute duration, where the at-site analysis aligns with the IFD.

It is noted that the critical duration for the study area is dominated by the 10 minute event (see Section 7.8), and hence there is no significant bias at this duration for the bowling club gauges, and the North Epping gauge only shows a slight bias for the at-site data to be lower than the IFD. For the adopted duration of 45 minutes, the at-site data is slightly higher than the IFD for the Pymble Bowling Club, and slightly lower for the Pennant Hills Bowling Club and North Epping gauges. The study area sits in between these gauges. The bias present is minimal and not consistent between gauges. Based on this analysis and consideration of the limitations of the available data, it was considered appropriate to adopt the ARR 2016 IFDs. Reference 16 provides further reasoning as to why the regionalised approach of the 2016 IFDs generates a more robust estimate of rainfall frequency than using a single site:

Although at-site frequency analysis of the Annual Maximum Series (AMS) of observed rainfall was an integral part of the method adopted for the 2016 design rainfalls, it was only one of many steps used to produce the new gridded, regional design rainfall estimates.

A regionalisation method was applied to give more weight to longer record stations within each region. This improved the estimates of rare (less frequent) events. A spline interpolation method was then applied to the regionalised rainfall data from across Australia to estimate gridded values for the whole country. Factors including latitude, longitude, elevation and consistency with neighbouring sites were used, in addition to rainfall characteristics at recording sites, thus allowing more reliable interpolation of rainfall depths in data sparse areas.

Rainfall values from a Generalised Extreme Value (GEV) distribution fitted to the AMS at a specific duration for a particular site will vary from the point values extracted from the grid of design rainfall values. Although each independent event in the AMS is a record of the actual rainfall recorded by a rain gauge, these measured rainfall values are effectively point samples of the rainfall distribution across Australia. Each point sample has its own uncertainty and does not represent completely the underlying population of rainfall values. The extracted grid values, created from the regionalised rainfall inputs, will generally fall within the 95% confidence limits of the GEV distribution for the specific duration at each location.

The length and period of record at a site makes a significant difference in the level of uncertainty of any at-site comparisons. Regionalisation was applied to the measured rainfall data to effectively smooth out the effects of sampling uncertainty.

7.3. Temporal Patterns

Temporal patterns are a hydrologic tool that describe how rain falls over time and are used in hydrograph estimation. Previously, with ARR 1987 guidelines (Reference 13), a single temporal pattern was adopted for each rainfall event duration. However, ARR 2019 (Reference 6) discusses the potential inaccuracies with adopting a single temporal pattern and recommends an approach where an ensemble of different temporal patterns is investigated.

Temporal patterns for this study were obtained from the ARR 2019 data hub (Reference 17, http://data.arr-software.org/). A summary of the data hub information at the catchment centroid is presented in Attachment A. The revised ARR 2019 temporal patterns attempt to address the key concerns practitioners found with the ARR 1987 temporal pattern approach. 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 and duration. Given the rainfall-runoff response can be quite catchment specific, using an ensemble of temporal patterns attempts to produce the median catchment response.

As hydrologic modelling has advanced, it is becoming increasingly important to use realistic temporal patterns. The ARR 1987 temporal patterns only provided a pattern of the most intense burst within a storm, whereas the ARR 2019 temporal patterns look at the entirety of the storm including pre-burst rainfall, the burst and post-burst rainfall. There can be significant variability in the burst loading distribution (i.e. depending on where 50% of the burst rainfall occurs an event can be defined as front, middle or back loaded). The ARR 2019 method divides Australia into 12 temporal pattern regions, with the Lane Cove Northern Catchments falling within the East Coast South region.

ARR 2019 provides 30 temporal patterns for each duration which are sub-divided into three temporal pattern bins based on the frequency of the events. Diagram 8 shows the three categories of bins (frequent, intermediate and rare) and corresponding AEP groups. The "very rare" bin is in the experimental stage and was not used in this flood study. There are ten temporal patterns for each AEP/duration in ARR 2019 that have been utilised in this study for the 20% AEP to 0.2% AEP events.

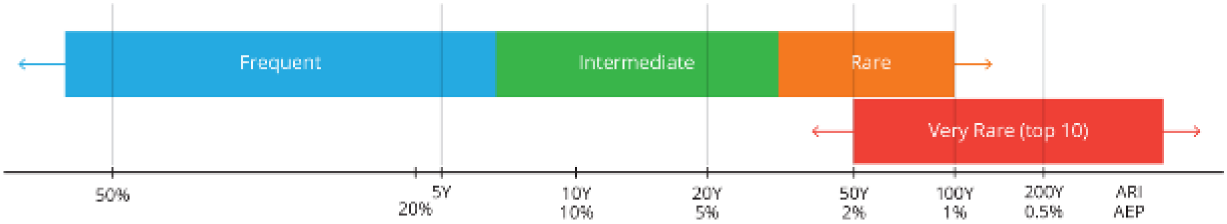


Diagram 8: Temporal Pattern Bins

The method employed to estimate the PMP utilises a single temporal pattern (Reference 14).

7.4. Rainfall Losses

For design flood modelling, the DRAINS model adopted the ILSAX parameters previously outlined in Table 8. This includes the depression storage values for each of the land coverage types and the soil type. Each of the design storms in DRAINS adopted an antecedent moisture condition of 3, representing a 'rather wet' catchment, or 12.5 mm to 25 mm of rain in the preceding 5 days. These parameters together represent the way that rainfall losses (both initial and continuing) are accounted for. Considering the adopted parameters, no pre-burst rainfall was included in the design storms and only the storm burst was simulated.

7.5. Areal Reduction Factors

The design rainfall estimates are based on point rainfalls and in reality, the catchment-average rainfall depth will be less. Areal Reduction Factors (ARFs) allow for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms simultaneously over the whole catchment area. Given the nature of the study area and the focus on overland flow paths through urban areas, ARFs were not applied in the DRAINS model. In accordance with ARR 2019 (Reference 6), catchments with an area up to 1 km^2 should not apply ARFs, and there is limited research on the applicability of ARFs to catchments that are less than 10 km^2. The largest catchment to an urban area was assessed and found to be less than 1 km^2. Even the largest catchment draining to the Comenarra Parkway (Avondale Creek, downstream of the urban area) was estimated to be approximately 2.4 km^2. It was therefore reasonable to not apply ARFs for the study area based on the size of the catchments draining to the areas of interest.

7.6. Downstream Boundary and Initial Conditions

As outlined in Section 5.6.2, a stage-discharge curve was adopted for the downstream boundaries where the creeks and tributaries discharge to the Lane Cove River. This simulates a normal flow depth condition and was adopted for all design flood events.

The study area was assumed to be dry at the start of the storm, with the exception of Avondale Dam, which was assumed to be full to the assumed crest level of the dam.

7.7. Blockage

ARR 2019 (Reference 6) recommends applying blockage to hydraulic structures, and outlines a methodology to determine inlet blockage factors by considering debris availability, debris mobility, debris transportability and waterway opening of the structure. This assessment was undertaken considering the typical culvert structures found in the study area. These structures fell into two categories, for which AEP dependent blockages were estimated in accordance with ARR 2019 procedures. The blockage factors from this analysis can be seen in Table 12.

Table 12: Blockage assessment results of key hydraulic structures

AEP	Blockage of small[1] structures	Blockage of large[2] structures
More frequent than 5%	25%	10%
5% to 0.5%	25%	20%
Rarer than 0.5%	50%	20%

[1] Generally smaller structures with a diameter or width less than 1.2 m (the assumed L_{10}), with typically a low 1% AEP debris potential as they are located in the upper catchment (e.g. culverts under the railway line).

[2] Generally larger structures with a diameter or width greater than or equal to 1.2 m (the assumed L_{10}), with typically a high 1% AEP debris potential as they are located in the lower catchment (e.g. culverts under The Comenarra Parkway at major creek crossings).

A single blockage factor was considered appropriate across the range of design flood events, with the adopted blockage factors outlined in Table 13. This includes the culverts discussed above, in addition to pit inlets.

Table 13: Adopted Blockage Factors

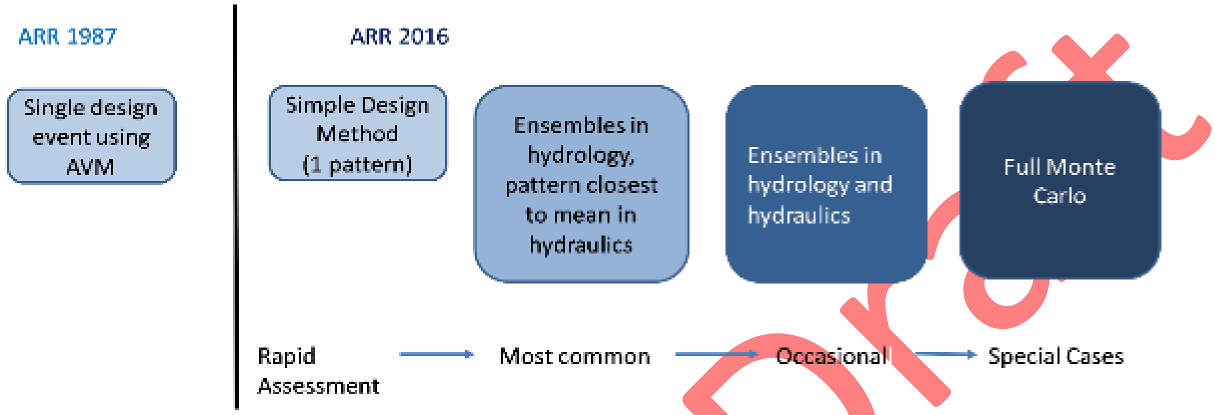
Structure	Design Blockage (%)
Culverts with headwalls	20%
Sag Pit	50%
On-grade Pit	20%

7.8. Critical Duration Assessment

7.8.1. Approach

ARR 2019 requires an ensemble of temporal patterns to be run for each AEP and duration combination, and the 'occasional' approach was adopted for Lane Cove Northern Catchments, as shown in Diagram 9.

Diagram 9: Design modelling techniques for an ensemble of temporal patterns (Reference 6)



This approach requires the ensemble of temporal patterns to be run in both the hydrologic and hydraulic models. This approach was adopted due to the complex nature of the shallow overland flow paths through the urban areas, which is of interest to the study. Total flows at key locations cannot readily be extracted from the DRAINS hydrologic model due to the nature of these flow paths.

7.8.2. Critical Duration

The critical duration is the storm duration that best represents the flood behaviour (e.g. flow or level) for a specific design magnitude at a particular location. 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 such as land use, shape, stream characteristics, etc.

With ARR 2019 methodology, the mean flow (or level) is computed from the ensemble of temporal patterns for each duration. The critical storm duration for a location of interest is then the design storm duration that produces the highest mean flow (or level). Where there are multiple locations of interest with different contributing catchment sizes, there can be multiple critical durations that need to be considered.

7.8.3. Representative Storm Burst

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. This is typically the storm that produces the next highest flow (or level) above the average (from the ensemble of temporal patterns) for the critical duration. In most cases, however, a representative storm does not necessarily need to be of the same duration as the critical duration, and there may be a number of storms that can represent the critical duration behaviour, potentially at multiple locations and even where the critical duration varies.

Adopting a range of critical durations across a catchment can complicate future analysis and the use of modelling tools if multiple storms need to be simulated to obtain the design flood behaviour for a particular event. Thus, it is preferable to adopt a single representative storm that is similar to the critical duration behaviour across the entire catchment for each event where possible.

7.8.4. Representative Storm Selection

To select the representative storm for each AEP for the Lane Cove Northern Catchments study area, the DRAINS hydrologic model was run for durations from 20 minutes to 4.5 hours, with the ensemble of temporal patterns for the 20% AEP, 5% AEP and 1% AEP events (representative of each temporal pattern bin). Each of these storms was then simulated in the TUFLOW model. For each duration, a grid of the mean peak level at each grid cell was calculated. A maximum envelope grid was then calculated taking the highest mean peak level for each grid cell. This shows the critical duration mean peak level at all flooded cells across the study area. The source of the peak mean level for each grid cell was mapped to show the variation in critical duration across the catchment. The critical duration maps are shown in Figure 31, Figure 33 and Figure 35 for the 20% AEP, 5% AEP and 1% AEP events, respectively. The majority of overland flow areas in the upper catchment had a critical duration of 10 minutes, which transitions to 20 minutes when creeks begin to form. At the downstream end of the major creeks the critical duration is 30 to 45 minutes. There are several small flood storage areas that have a longer critical duration although typically still less than 2 hours.

A histogram of the number of cells (frequency) for each duration is shown in Diagram 10 for each of the events simulated. This indicates that the majority of the study area is dominated by the 10 minute storm, due to the large area that shallow overland flows cover in the upper catchment areas compared to the confined creek channels. Durations above 45 minutes only cover a very small portion of the study area, with the critical duration primarily being represented by the 10 minute to 45 minute storms.

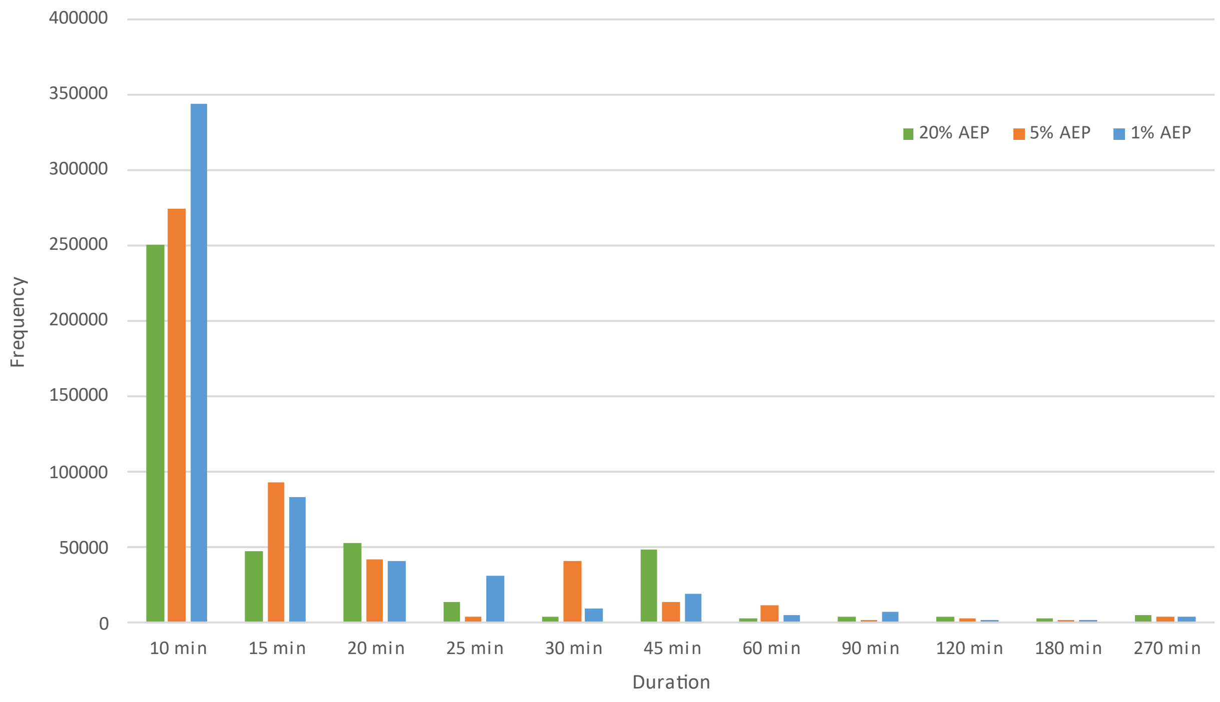


Diagram 10: Histogram of critical durations across the study area

Through a comparison of the peak flood level grid for each storm with the critical duration mean peak level across the entire study area, a representative storm was selected for each AEP event simulated. This was a temporal pattern from the 45 minute storm duration and was able to represent the critical duration in downstream areas (such as The Comenarra Parkway crossings), while also being representative of flooding in upstream areas. Although the critical duration in the upstream areas was 10 minutes, the shallow overland nature of flooding in these areas means that there is very little change in peak flood levels across different durations or even different AEPs. The selection process focussed on accurately representing the critical duration behaviour in the urban areas, while getting as close as possible to the critical duration behaviour through the creeks down to The Comenarra Parkway. The selected storms result in minimal variation in peak water level from the critical duration mean peak level. The selected storm typically results in slightly higher levels by up to 0.05 m, however this is primarily in the creeks through forested areas. This difference is shown in Figure 32, Figure 34 and Figure 36 for the 20% AEP, 5% AEP and 1% AEP events, respectively.

A similar, but simplified approach was undertaken for the PMF event, whereby a single storm was run for durations from 15 minutes to 1 hour. The results indicated that the 15 minute storm was critical across the majority of the urbanised study area, with the 30 minute and 45 minute storms dominating in the downstream forested creek areas, as shown in Figure 37. For the purpose of this study, the 15 minute and 45 minute storms were selected as being representative of flooding across the study area. The maximum envelope of these two durations was taken to produce the PMF results across the study area.

The selected storms were considered representative for all design events within that temporal pattern bin (Diagram 8). The selected storms were adopted for modelling of the design flood events and processing of flood results (as described in Section 8). The adopted representative design storms are summarised in Table 14.

Table 14: Adopted Representative Design Storms

Temporal Pattern Bin	Events	Duration (mins)	Temporal Pattern ID (Ensemble No.)
Frequent	20% AEP	45	4554 (10)
Intermediate	10% AEP 5% AEP	45	4536 (2)
Rare (2% AEP to 0.2% AEP)	2% AEP 1% AEP 0.5% AEP 0.2% AEP	45	4535 (10)
N/A	PMP	15, 45	GSDM

8. DESIGN FLOOD RESULTS

8.1. Introduction

The 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP events were simulated using the adopted representative design storms. The PMF event was also simulated using the 15 minute and 45 minute PMP storms. The storms were run in the hydrologic model and the resulting flows were input into the hydraulic model to simulate flood behaviour across the study area. The results for the design flood events are presented in the following maps:

- Peak flood depths and levels in Figure F1 to Figure F8;
- Peak flood velocities in Figure F9 to Figure F16;
- Hydraulic hazard in Figure F17 to Figure F19; and
- Hydraulic categories in Figure F20 to Figure F22.

These results are available in electronic GIS and tabular format. The digital data should be used in preference to the figures in this report as they provide more detail. The figures are intended to provide an overview of the results and should not be relied upon for detailed information at individual properties.

Additional results are presented in the following tables and graphs:

- Stage hydrographs at road crossings in Figure G1 to Figure G37; and
- Peak flood levels, depths and flows at road crossings and key locations in Table G1, Table G2 and Table G3, respectively.

A discussion of these results is provided in the following sections.

8.2. Summary of Results

The flood behaviour across the Lane Cove Northern Catchments study area can be seen in the peak flood depth / level maps (Figure F1 to Figure F8) and peak velocity maps (Figure F9 to Figure F16). These results are presented for the range of design flood events modelled from the 20% AEP to the PMF event. A tabulated summary of peak flood levels, depths and flows at selected locations, as shown in Figure 38, are detailed in Table G1, Table G2 and Table G3, respectively.

In frequent events, flow is generally shallow (<0.15 m) and contained within the gutters and dedicated drainage reserves in the urban areas. There are some areas, however, where shallow overland flow paths form through properties. Within the major creeks and tributaries, flow is typically contained within the channel. In rarer events, more of the overland flow paths form through urban areas as the stormwater network and kerb and gutter system reach capacity. Given the steep nature of the study area, many of these flow paths remain shallow. Along water courses and key flow paths, affectation of property becomes more evident in rarer events such as around Tanderra Street in Wahroonga, Cynthia Street and Hesperus Street in Pymble, and between Binalong Street and Lofberg Road in West Pymble. Ponding at sag points becomes more prominent in these events as well as creeks overtopping road crossings, with key locations including crossings of Yanko Road, Doncaster Avenue and The Comenarra Parkway at major creek crossings, in addition to local roads such as Exeter Road in Wahroonga, Holmes Street in Turramurra and Forwood Avenue in Turramurra.

In the PMF event a large portion of the study area is inundated, although much of this is still shallow overland flow, with deeper areas restricted to channels, concentrated flow paths and sag points. Development, however, has occurred on some of these flow paths.

8.3. Hydraulic Hazard Categorisation

Hydraulic hazard is a measure of the potential risk to life and property damage from flooding. Hydraulic hazard is typically determined by considering the depth and velocity of floodwaters. In recent years, there have been several developments in the classification of flood hazard. Research has been undertaken to assess the hazard to people, vehicles and buildings based on flood depth, velocity and velocity depth product. ARR 2019 (Reference 6) contains updated recommendations regarding the categorisation of flood hazard. A summary of this categorisation is provided in Diagram 11. This categorisation is based on an extensive literature review and laboratory testing. It considers hazard to people, vehicles and buildings to develop 6 categories of flood hazard based on flood depth, velocity and depth-velocity product.

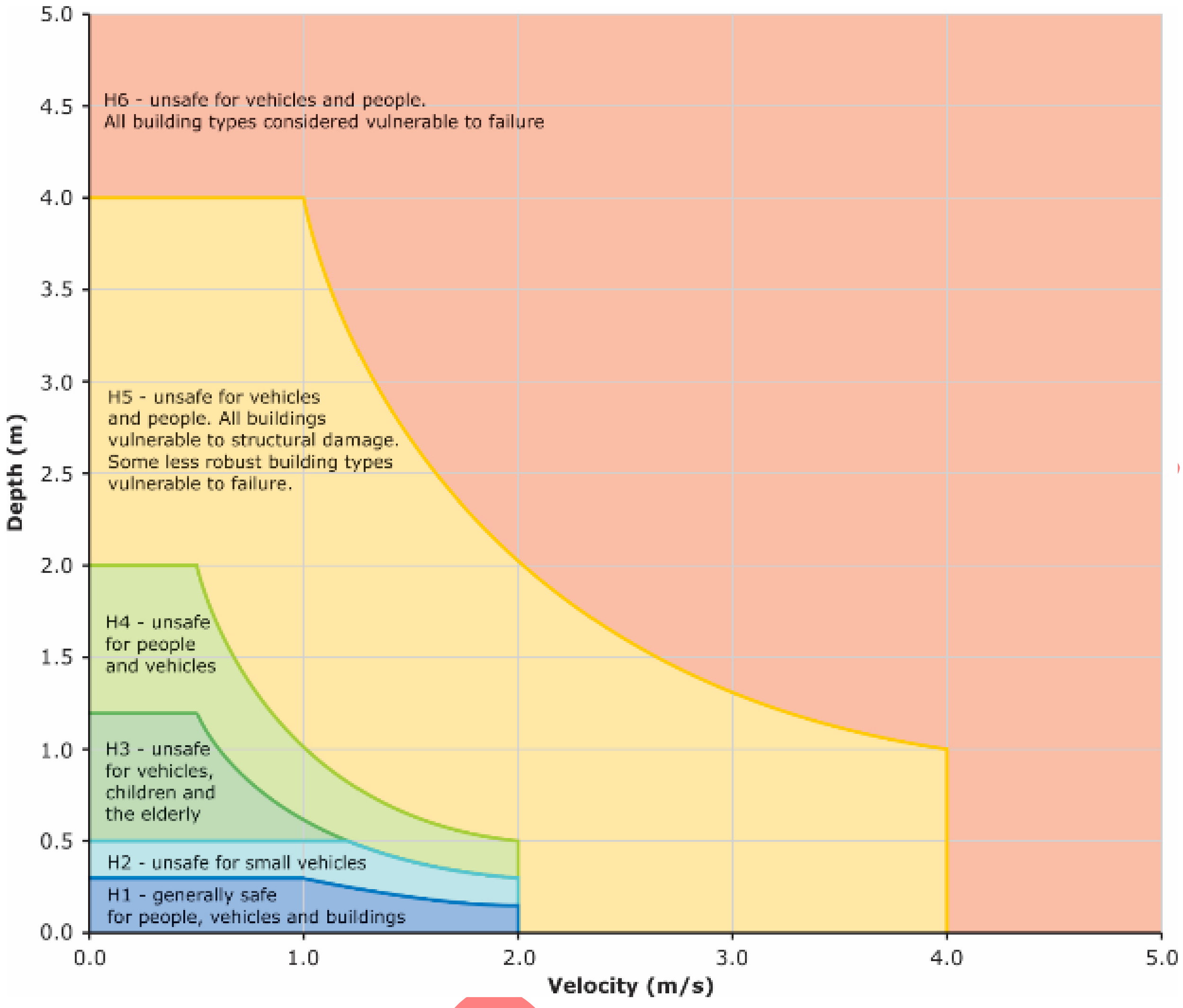


Diagram 11: General flood hazard vulnerability curves (Source: Reference 6)

The following 6 classes of hazard are defined:

- H1 – Generally safe for vehicles, people and buildings;
- H2 – Unsafe for small vehicles;
- H3 – Unsafe for vehicles, children and the elderly;
- H4 – Unsafe for vehicles and people;
- H5 – Unsafe for vehicles and people. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure; and
- H6 – Unsafe for vehicles and people. All building types considered vulnerable to failure.

The hazard categories using the ARR 2019 classification are mapped in Figure F17, Figure F18 and Figure F19 for the 1% AEP, 0.2% AEP and PMF events. In the 1% and 0.2% AEP events, much of the urban area is affected by H1 hazard, with areas of higher hazard (H3 and above) generally restricted to creek channels. There are some areas of higher hazard where creeks overtop roads or where high velocity water flows down streets. In the PMF event many of the roads and flow paths that convey a substantial amount of water are classified as H5, with H6 being common within the creek channels.

8.4. Hydraulic Categorisation

Hydraulic categorisation involves mapping the floodplain to indicate which areas are most important for the conveyance of floodwaters and the temporary storage of floodwaters. This can help in planning decisions about which parts of the floodplain are suitable for development, and which areas need to be left as-is to ensure that flooding impacts are not worsened compared to existing conditions.

The NSW Government's 2005 Floodplain Development Manual (Reference 18) 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, such as that of Howells *et al* (Reference 19), rely on combinations of velocity and depth criteria to define the floodway.

For this study, hydraulic categories were defined by the following criteria and is considered to be a reasonable representation of the flood function of this catchment:

- Floodway is defined as areas where:
  - the peak value of velocity multiplied by depth (V x D) > 0.3 m^2/s, **AND** peak velocity > 0.3 m/s, **OR**
  - peak velocity > 1.0 m/s **AND** peak depth > 0.2 m;

  The remainder of the floodplain is either Flood Storage or Flood Fringe;
- Flood Storage comprises areas outside the floodway where peak depth > 0.3 m; and
- Flood Fringe comprises areas outside the Floodway where peak depth < 0.3 m.

The hydraulic categories have been mapped in Figure F20, Figure F21 and Figure F22 for the 1% AEP, 0.2% AEP and PMF events, respectively. As expected, the creeks and major flow paths are classified as floodways in the 1% AEP and 0.2% AEP events, with flood storage areas where there is ponding on road sag points, on the upstream side of buildings and other isolated areas. In the PMF event, the majority of flow paths are floodways, with only shallow overland flow remaining as flood fringe.

8.5. Flood Emergency Response Planning

8.5.1. Property Inundation

Due to the nature of the catchment, it is difficult to determine if properties are inundated in certain events. With the steep topography present, many houses interface with high side of the terrain and are elevated above lower areas including waterways. However, in plan view, the creek often appears to impact the building. An example is provided in Photo 3. There are also numerous properties with creeks that flow through them. In some cases, the dwelling is located at the rear of the property, with the driveway crossing the creek to access the road, such as those located on the western side of Wyomee Avenue.

There are two key areas where development has taken place on flow paths and is likely to be at risk of inundation in a flood event due to the depth of flooding at the buildings. These areas are:

- Gilda Avenue, Ada Avenue and Tanderra Street, Wahroonga;
- Cynthia Street and Hesperus Street, Pymble.

In other parts of the catchment there are individual properties (for example those located directly on a flow path) that may also be at risk of inundation. There are also several areas subject to shallow overland flows (less than 0.15 m deep), however, the risk of building inundation is typically lower, due to the shallow nature of flows.

8.5.2. Road Inundation

There are numerous local roads throughout the study area that are subject to inundation. The inundation is typically shallow as overland flows are conveyed along road corridors. In some areas, where major flow paths cross roads, however, the depth of flow can be significant. At some of the major road crossings (typically where the 1% AEP depth reaches over 0.3 m), water level hydrographs were provided in Figure G1 to Figure G37 of Appendix G. Peak flood levels, depths and flows at key locations are also provided in Table G1, Table G2 and Table G3 respectively. The locations of these road crossings are shown in Figure 38.

Some of the deepest flooding experienced is where major creeks cross local roads. The 1% AEP flood depth reaches over 1 m at Exeter Road, Wahroonga; Homes Street, Turramurra and Forwood Avenue, Turramurra. Flood depths on other roads reported on throughout the study area range between 0.3 m and 1 m in the 1% AEP event. While there are some road crossings not inundated in the 20% AEP event, most road crossings were inundated by the 5% AEP event (with the exception of The Comenarra Parkway at Peppermint Creek and Water Dragon Creek, St Andrews Drive and Troon Place, which have a higher level of flood immunity).

The rate of rise at each of the road crossings is very quick, typically within 20 to 30 minutes of the onset of rainfall. This is driven by the quick catchment response and the selected critical duration storm (45 minutes). The rate of rise may be quicker than this, noting much of the urban areas had a critical duration of just 10 minutes. While the rate of rise is rapid, the duration of inundation is also short, with flooding typically lasting less than 1 hour. Again, the duration of inundation may be longer for a longer storm duration, although likely to reach a lower peak level.

8.5.3. Flood Emergency Response Classification

The Floodplain Development Manual (Reference 18) requires flood studies to address the management of continuing flood risk to both existing and future development areas. As continuing flood risk varies across the floodplain, so does the type and scale of the emergency response problem and therefore the information necessary for effective Emergency Response Planning (ERP). Classification provides an indication of the vulnerability of the community in flood emergency response and identifies the type and scale of information needed by the NSW State Emergency Service (SES) to assist in ERP.

The Flood Emergency Response Classification (FERC) for the study area was undertaken in accordance with the *Australian Disaster Resilience Handbook 7 Managing the Floodplain: A guide to best practice flood risk management in Australia* (Reference 20). FERC classifications consider flood affected communities as those in which the normal functioning of services is altered, either directly or indirectly, and 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.

The ERP classification for urban regions within the hydraulic model extent have been defined using the PMF flood event and can be seen in Figure F23. The classification has been undertaken on a precinct basis rather than lot-by-lot and is targeted at highlighting those areas which may require evacuation or assistance during a flood event. However, these classifications may vary depending on local flood characteristics and resultant flood behaviour, i.e. in flash flooding or overland flood areas. These categories are described in Diagram 12 below.

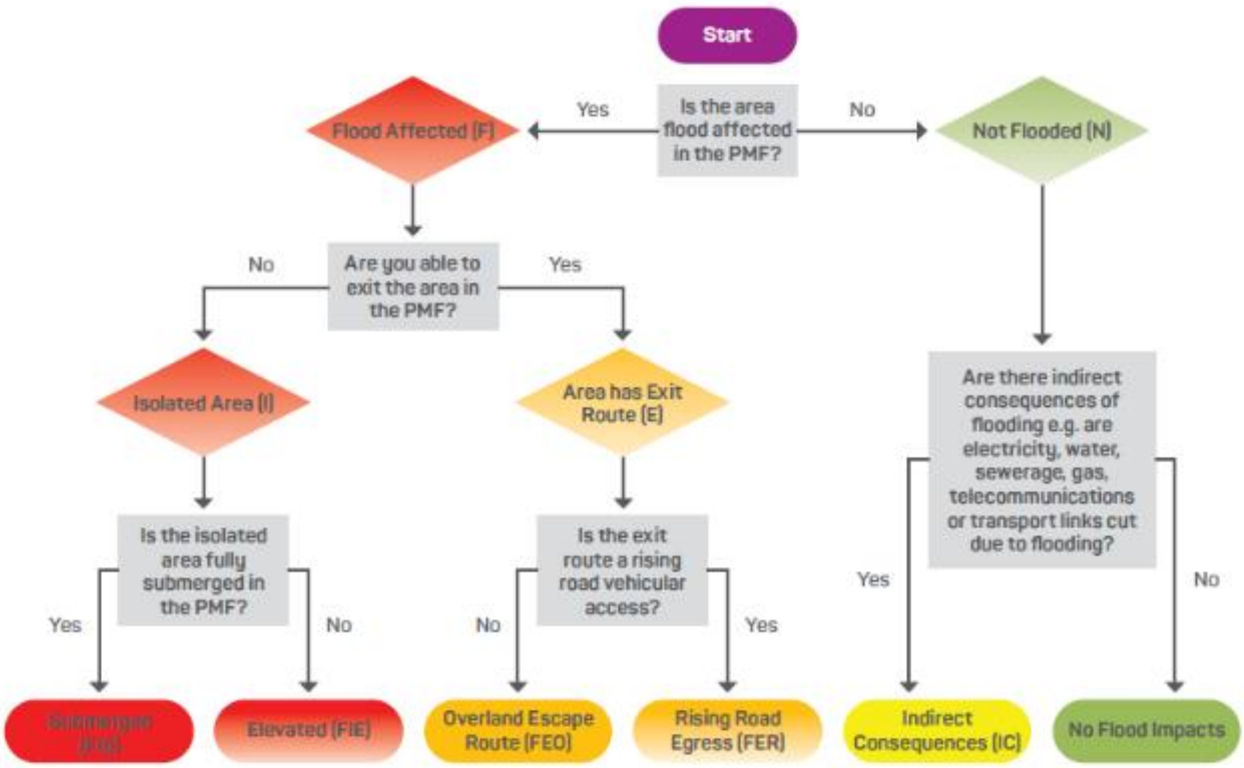


Diagram 12: Flow Chart for Determining Flood Emergency Response Classifications (Reference 20)

Some of the key areas with evacuation difficulties (flooded isolated and submerged (FIS) and flooded isolated elevated (FIE) areas) include the following:

- Campbell Drive, Wahroonga;
- Jordan Road, Wahroonga;
- Howson Avenue, Turramurra;
- Warragal Road, Turramurra;
- Yamba Street, Forwood Avenue and Hudson Close, Turramurra and South Turramurra;
- Quadrant Close, St Andrews Drive and Troon Place, Pymble;
- Around Barwon Avenue, South Turramurra;
- The southern end of Kissing Point Road, South Turramurra;
- Avon Road, Pymble;
- Around Greenway Drive, Wyomee Avenue and Yanko Road, Pymble and West Pymble;
- Gloucester Avenue, West Pymble.

8.6. Flood Planning Area

8.6.1. Background

Land use planning is an effective means of minimising flood risk and damages from flooding. Land use planning for flooding can be achieved through the use of:

- A Flood Planning Area (FPA), which identifies land that is subject to flood related development controls; and
- A Flood Planning Level (FPL), which identifies the minimum floor level applied to development proposals within the FPA.

Defining FPAs and FPLs in urban areas can be complicated by the variability of flow conditions between mainstream and local overland flow. Traditional approaches developed for riverine or "mainstream" flow areas often cannot be applied in steeper urban overland flow areas. Additionally, defining the area of flood affectation due to overland flow (which by its nature includes shallow flow) involves determining at which point flow is significant enough to be classified as "flooding" rather than just a drainage or local runoff issue. In some areas of overland flow, the difference in peak flood level between events of varying magnitude can be so minor that applying the typical freeboard can result in an FPL greater than the PMF level.

The FPA should include properties where development would result in impacts on flood behaviour in the surrounding area and in areas of high hazard where there is a risk to safety or life. The FPL is determined in addition to this with the purpose of decreasing the likelihood of damage such as over-floor flooding of buildings.

The Floodplain Development Manual (Reference 18) suggests that the FPL generally be based on the 1% AEP event plus an appropriate freeboard (typically 0.5 m). However, it also recognises that different freeboards may be deemed appropriate due to local conditions provided adequate justification is provided.

Further consideration of flood planning areas and levels is typically undertaken as part of the Floodplain Risk Management Study to determine what should be included in the Floodplain Risk Management Plan.

8.6.2. Methodology

The methodology used for defining the flood planning area is consistent with that adopted in a number of similar studies throughout the Sydney metropolitan area. It divides the flood area between “mainstream” and “overland” flooding areas using the following criteria:

- Mainstream flooding: Areas along the main creeks or trunk drainage alignment, where flow is sufficiently deep and there is sufficient relief that freeboard can be added to the flood surface and the extent then “stretched” to include adjacent land. The mainstream part of the study was defined as those creeks and flow paths where water concentrates into a defined water course. The FPA along this reach was defined as the 1% AEP peak flood level plus 0.5 m freeboard, with the level extended perpendicular to the flow direction either side of the flow path.
- Overland flooding: For overland flow areas, addition of freeboard and stretching generally produces an over-estimate of the land subject to flood risk. This is because the stretching extends across land in a way that would not actually occur even with significant additional flow from a much larger storm. It may even extend beyond the modelled PMF extent. It is therefore appropriate to not apply freeboard for the purpose of defining the FPA for overland flooding. The 1% AEP event was adopted with the following filtering applied (to remove shallow overland flows not associated with a continuous flow path):
    - **Depth Filter** – Exclude results below 150 mm depth; and
    - **Small Pond Filter** – Remove isolated ‘puddles’ or ‘orphans’ smaller than 150 m^2.

The resultant extent of both the mainstream FPA and overland FPA can be seen in Figure F24.

8.6.3. PMF Event

In July 2021, the NSW Government implemented updates to the Flood Prone Land Package. The purpose of the package is to increase flood resilience in New South Wales, reduce loss of life and property damage. The package provides Councils additional land use planning tools to manage flood risk beyond the 1% AEP flood event and strengthen evacuation consideration in land use planning. The updates included amendments to Schedule 4 of Environmental Planning and Assessment Regulation including changes to Clause 7A(1), Clause 7A(2). These amendments (now contained in Clause 9 of Schedule 2) now require Councils to note on Section 10.7 certificates if the land is within the FPA (Clause 9(1)) or between the FPA and PMF (Clause 9(2)) and subject to flood related development controls.

For the purpose of identification of those properties that are outside the FPA, but within the PMF and subject to flood related development controls (for development comprising hazardous materials / industry, sensitive, vulnerable or critical uses), the PMF extent was filtered in a similar manner to the derivation of the overland FPA. The following filtering was applied:

- **Depth Filter** – Exclude results below 300 mm depth; and

- **Small Pond Filter** – Remove isolated 'puddles' or 'orphans' smaller than 300 m^2.

The resultant extent of the PMF for flood planning controls can be seen in Figure F25.

8.7. Flooding Hotspots

8.7.1. Tanderra Street, Wahroonga

This area is located on a tributary flow path of Coups Creek in Wahroonga. Water flows down past Rhonda Close and then through several private properties on Gilda Avenue and Walpole Place. Water can overtop Ada Avenue and runs through several properties on Tanderra Street. Water then flows in a southwest direction and crosses Amaroo Avenue, where it then becomes more channelised before joining Coups Creek immediately upstream of Mahratta Avenue.

The peak flood depths and hydraulic hazard for the 1% AEP event can be seen in Figure F26. Although the flow path commences as shallow overland flow, it becomes deeper due to a trapped depression between Gilda Avenue and Ada Avenue, with the 1% AEP flood depth reaching 0.8 m upstream of Ada Avenue, triggering H3 hazard. Downstream of Tanderra Street, the primary flow path is through private property, rather than along Tanderra Street, which is reasonably flat (Photo 23). At the local sag point on Tanderra Street, the 1% AEP flood depth reaches approximately 0.5 m with the flood hazard being H2. Flood depths on the flow path through properties (on the right of Photo 23) can reach higher than this, with up to H3 hazard. The 1200 mm diameter pipe that conveys flow under this flow path is at capacity in events as frequent as the 20% AEP.

Photo 23: Tanderra Street sag point, looking east (*Source: Google Street View*)

8.7.1. Monteith Street, Turramurra

The confluence of two major flow paths occurs at the Holmes Street cul-de-sac in Turramurra. The cul-de-sac is located within a local depression, which, once filled, spills through private properties to the south. This water flows overland to the Monteith Street sag point (Photo 24). On the downstream (southern) side of Monteith Street, water is channelised between buildings (see Photo 3) and is conveyed to Rothwell Road. Water Dragon Creek officially forms through the bushland area downstream of Rothwell Road.

Photo 24: Monteith Street sag point, looking southeast (*Source: Google Street View*)

The peak flood depths and hydraulic hazard for the 1% AEP event can be seen in Figure F27. The water ponds in the Holmes Street cul-de-sac with depths of over 1.5 m in the 1% AEP event. Due to depth and velocity the flow path downstream of Holmes Street, the floodwater is primarily categorised as H5 with sections of H6. The 1800 mm diameter culvert at Monteith Street and the triple 1350 mm diameter culverts at Rothwell Road reach capacity in approximately a 10% AEP event. The depths during the 1% AEP event are modelled to be approximately 0.8 m and 0.6 m for the Monteith Street and Rothwell Road crossings, respectively.

8.7.2. Cynthia Street, Pymble

The confluence of two flow paths between Cynthia Street and Hesperus Street in Pymble. One flow path comes from Golfers Parade and crosses through private properties to Cynthia Street. A smaller flow path arrives at Cynthia Street from Ward Street and Yarrara Road that also crosses through private property. There are two sag points on Cynthia Street that receive flows from the two flow paths, although the road is relatively flat and these sag points are not well defined (Photo 25).

Photo 25: Cynthia Street sag point, looking southeast (*Source: Google Street View*)

From Cynthia Street, floodwater in excess of the stormwater network can be conveyed overland through private property. Dispersed overflows from the two sag points (approximately 150 m wide) flow through private property and converge at a single sag point on Hesperus Street. Downstream of Hesperus Street an open channel takes flow from the stormwater network (the main culvert under Hesperus Street is a 1050 mm diameter pipe) and overland flows. This channel only runs the length of one property. Flow is then conveyed via a 2000 mm (W) x 1200 mm (H) box culvert and then twin 1050 mm diameter pipes as it runs around the Latona Street cul-de-sac. It then discharges into another open channel at the end of Latona Street and then into a 1500 mm diameter pipe under Greenway Drive at the sag point (Photo 26). There is then another portion of open channel before being conveyed under Warrowa Avenue via a 1800 mm diameter pipe. Downstream of Warrowa Avenue, flow is conveyed in an open channel adjacent to Wyomee Avenue. Along this flow path, between Cynthia Street and Warrowa Avenue as water enters into and out of culverts and open channels, overland flow occurs that affects properties.

Photo 26: Greenway Drive sag point, looking southwest (*Source: Google Street View*)

The peak flood depths and hydraulic hazard for the 1% AEP event can be seen in Figure F28. There is widespread inundation of properties on Cynthia Street, with flood depths reaching up to 0.5 m in the 1% AEP event. The flood hazard is typically H2 for the disperse and shallow flow path, although can reach higher around structures where there are larger depths and/or velocities. Downstream of Hesperus Street there are properties affected, with overland flows the deepest between the two sections of open channel on the northern side of the Latona Street cul-de-sac, with flood depths also typically reaching 0.6 m in the 1% AEP event and H5 hazard. At Greenway Drive, the peak flood depth in the 1% AEP event reaches 0.7 m with H4 and H5 hazard over the road sag point near the intersection with Par Close. The Greenway Drive culvert capacity is exceeded in the 20% AEP event.

8.7.3. Kendall Street, West Pymble

This flow path is a tributary of Quarry Creek in West Pymble. Stormwater drainage and overland flows upstream of Grayling Road discharge into a channel through Grayling Street Reserve on the downstream side of the road. From this channel, water is conveyed through a 900 mm diameter culvert, with flows in excess of this overtopping Binalong Street and running through private property. The flow path crosses the Kendall Street sag point (the culvert now a 1200 mm diameter pipe at this location) and runs through Our Lady of Perpetual Succour Catholic Primary School (Photo 27). Overland flows downstream of the school are conveyed along a private shared driveway (see Photo 2) before crossing Lofberg Road (the culvert now a 1350 mm diameter pipe at this location). Overland flow from the Lofberg sag point is conveyed through an area with cricket nets adjacent to Lofberg Oval within Bicentennial Park and discharges into a creek channel within the park. At the southern corner of Bicentennial Park, it is joined by flow from Norman Griffiths Oval and forms Quarry Creek.

Photo 27: Kendall Street sag point and school, looking southwest (*Source: Google Street View*)

The peak flood depths and hydraulic hazard for the 1% AEP event can be seen in Figure F29. Although the flow path remains relatively narrow, the peak depth occurs directly adjacent to the buildings downstream of Binalong Street, including the primary school. At Kendall Street the peak flood depth reaches 0.7 m with H3 hazard in the 1% AEP event. Through the school, the hazard reaches H5 and H6. Flooding over Lofberg Road reaches 0.5 m depth with H3 hazard in the 1% AEP event. The capacity of the drainage pipe along this flow path is exceeded in the 10% AEP event.

8.1. Advice on Land-Use Planning Considering Flooding

It is considered good practice to permit land use and development that is compatible with the nature of flooding in a particular area. For example, it is wise to limit use and development of land that is classified as floodway, since these are areas of conveyance and not only pose significant risks to humans, but any development in these areas can shift flood risks to other areas.

8.1.1. Existing Flood Planning Controls

KRGC implements flood-related planning controls in the study area via the Ku-ring-gai Local Environmental Plan 2015 (LEP, Reference 21) and Ku-ring-gai Development Control Plan 2024 (DCP, Reference 22). The LEP specifies that land is subject to flood-related restrictions if it is within the flood planning area for any type of development (Clause 5.21).

The LEP outlines the overall objectives and nature of these restrictions, the DCP specifies flood-related development controls that apply to land affected by flooding (DCP 2024, Section C, 24D.2 and 24R.3), or land where a catchment flood study has not been completed.

The flood-related development controls specified in the DCP cover flood impacts (not making flooding worse for neighbouring properties), building components and structural soundness (to ensure buildings can withstand flood forces), minimum floor levels and consideration of safety for people and vehicles. Council considers flood risk based on the FPA extent (where there is an existing Flood Study) or where council deems the development could influence a nearby drainage system (where there is no existing Flood Study).

The DCP controls are adequate, allowing for:

- Consideration of flood affectation of the site or development, as 'overland' or 'mainstream', including the application of a variable freeboard.
- Application of key floodplain management principles with regard to land use planning and development. This includes flood impacts and flood resilience of the proposed development.
- Fencing is a key consideration for overland flow, and controls for fencing are outlined in 24D.7.

However, the DCP does not allow for further consideration of flood risk beyond 'overland' and 'mainstream' classifications (for example low, medium or high risk) or consideration of the development type (for example an industrial development versus a residential development). The DCP only specifies minimum floor levels for habitable floors and garages. The breakdown of land use categories would further consider different land use vulnerability to flooding and the breakdown of the floodplain would further consider flood constraints on land. A matrix approach of flood-related development controls is considered current best practice that factors in land use and flood risk.

The flood study requirements (24R.3) appear to be geared toward smaller developments where flood studies do not exist. For example, it discusses catchment sizes and flow rates, use of the rational method, use of Mannings equation, use of HEC-RAS modelling and references to cross-sections, etc. Given KRGC now has detailed 2D overland flood modelling across much of the LGA, the approach to a flood study for a development is very different. The consideration of using the flood study model to assess a proposed development is absent from this section of the DCP.

The DCP does not make provisions for:

- Design guidelines for basement car parking in flood affected areas.
- Control of development in high hazard or floodway areas.
- Consideration of the latest research for vehicle stability for car parking criteria. The DCP refers to velocity x depth product, whereas considering a hydraulic hazard category (see Section 8.3) provides a simpler way to map and visualise vehicle stability criteria and demonstrate compliance.
- Emergency response considerations including reliable access, evacuation, flood warning, rate of rise and duration of inundation, emergency response strategies or shelter-in-place requirements.
- Ongoing management of flood risk such as storage of hazardous materials, flood management plans or subdivisions with potential future development.

The current DCP provides some breakdown of flood risk and constraints, however, does not clearly highlight all flood related development constraints described in the flood planning constraint category (FPCC) approach, which is outlined below. An approach such as the FPCCs is considered current best practice for land use planning.

8.1.2. Flood Planning Constraint Categories

Guideline 7-5 of the Australian Disaster Resilience Handbook Collection (Reference 20) recommends using FPCCs to better inform land use planning activities. These categories condense the wealth of flood information produced in a flood study and classify the floodplain into areas with similar degrees of constraint. These FPCCs can be used in high level assessments of land use planning to inform and support decisions. For detailed land use planning activities, it is recommended that the flood behaviour across the range of flood events be considered, depending on the level of constraint.

The Australian Disaster Resilience Handbook Collection (Reference 20) recommends the use of four constraint categories. It is recommended that isolation potential also be considered for the high constraint category. This could include areas classified as 'isolated' (see Section 8.5 for details). Isolation has not been considered in the FPCCs defined for the study area, since it is not considered to be a significant constraint in this catchment due to the short duration of flooding. In areas that are already developed, the isolation potential has been defined using Flood Emergency Response Classifications (see Section 8.5), and land use planning activities should consider these in addition to the FPCCs.

The constraints have been adapted to suit the Lane Cove Northern catchments and are outlined in Table 15. The associated FPCC map is provided in Figure F30.

Table 15: Flood Planning Constraint Categories for the Lane Cove Northern catchments

FPCC	Constraints	Implications	Considerations
FPCC 1	Floodway and flood storage areas in the 1% AEP event H6 hazard in the 1% AEP event	Any development is likely to affect flood behaviour in the 1% AEP event and cause impacts elsewhere. Hazardous conditions considered unsafe for vehicles and people, all types of buildings considered vulnerable to structural failure.	Majority of developments and uses have adverse impacts on flood behaviour or are vulnerable. Consider limiting uses and developments to those that are compatible with flood function and hazard.
FPCC 2	Floodway in the 0.2% AEP event H5 flood hazard in the 1% AEP event H6 flood hazard in the 0.2% AEP event	People and buildings in these areas may be affected by dangerous floodwaters in rarer events. Hazardous conditions considered unsafe for vehicles and people, and all buildings vulnerable to structural damage. Hazardous conditions develop in rare events which may have implications for the development and its occupants.	Many uses and developments will be vulnerable. Consider limiting new uses to those compatible with flood function and hazard (including rarer flood flows) or consider treatments to reduce the hazard (such as filling). Consider the need for additional development control conditions to reduce the effect of flooding on the development and its occupants.
FPCC 3	Within the FPA	Hazardous conditions may exist creating issues for vehicles and people. Structural damage to buildings is unlikely.	Standard land use and development controls aimed at reducing damage and the exposure of the development to flooding are likely to be suitable. Consider additional conditions for emergency response facilities, key community infrastructure and land uses with vulnerable users.
FPCC 4	Within the PMF extent	Emergency response may rely on key community facilities such as emergency hospitals, emergency management headquarters and evacuation centres operating during an event. Recovery may rely on key utility services being able to be readily re-established after an event.	Consider the need for conditions for emergency response facilities, key community infrastructure and land uses with vulnerable users.

9. SENSITIVITY ANALYSIS

9.1. Overview

A number of sensitivity analyses were undertaken to establish the variation in design flood levels and flows that may occur if different parameter assumptions were made. These sensitivity scenarios are summarised in Table 16.

Table 16: Overview of Sensitivity Analyses

Scenario	Condition 1	Condition 2
Antecedent moisture condition (AMC)	Rather dry (AMC 2)	Saturated (AMC 4)
Rainfall loss	Reduced 50%	Increased 50%
Mannings “n” roughness	Reduced 25%	Increased 25%
Structure Blockage	Unblocked	Increased 50%
Pit Blockage	Increased 50%	-
Downstream boundary condition (curve slope)	x 0.1	x 10
Climate Change (for the 1% AEP event)	0.5% AEP	0.2% AEP

The change in flood level across the study area for each scenario compared to the adopted design 5% or 1% AEP flood events are provided in Appendix H.

9.2. Antecedent Moisture Condition (AMC)

DRAINS uses the AMC parameter, in conjunction with the soil type parameter, to determine the initial infiltration capacity of the soil. AMC is the representation of the soil conditions due to rainfall in the 5 days preceding the modelled event. The design flood events utilised a ‘Rather Wet’ condition, representing 12.5-25 mm of rainfall in the preceding 5 days. Two conditions were modelled to check the sensitivity to the soil moisture conditions, ‘Rather Dry’ (0-12.5 mm) and ‘Saturated’ (>25 mm). The change in peak flood levels is shown in Figure H1 to Figure H4.

The peak flood level moderately decreases for drier conditions. For both the 5% and 1% AEP events, the peak flood level decreases by typically up to 0.05 m throughout the urban areas where overland flow begins to concentrate. In the downstream creek areas, the decrease is greater, typically in the order of 0.2 m, although can be up to 1 m upstream of hydraulic structures.

The increase in soil saturation results in the inverse, with an increase in peak flood levels. The increases throughout the urban areas are typically up to 0.05 m, with higher increases in the downstream areas. Flood levels in the creeks increases by approximately 0.2 m and up to 0.4 m upstream of hydraulic structures.

9.3. Rainfall Losses

Initial rainfall losses in DRAINS are modelled as depression storages across each of the areas within a sub-catchment. They have been altered to simulate both an increase and decrease in rainfall losses by 50%. For both impervious and pervious areas, the change in storage depression will affect the quantity and timing of run-off generated. Continuing rainfall losses (infiltration into the soil) in DRAINS are determined by the soil type, with four different types to choose from, ranging from high infiltration rates (sands and gravel) to very slow infiltration rates (clays with a high water table). The soil type was also varied to simulate the change in rainfall infiltration. These scenarios and the corresponding parameter values are shown in Table 17.

Table 17: Rainfall Loss Sensitivity

Parameter	Adopted Design	-50%	+50%
Paved area (impervious) depression storage (mm)	1	0.5	1.5
Supplementary area depression storage (mm)	1	0.5	1.5
Grassed (pervious) area depression storage (mm)	5	2.5	7.5
Soil Type (infiltration rate)	3 (slow)	4 (very slow)	2 (moderate)

The change in peak flood levels is shown in Figure H5 to Figure H8, for both the 5% AEP and 1% AEP events. The results from modelling the variations in rainfall losses indicate a similar outcome to the AMC scenarios, although the magnitude of the respective increases and decreases in flood depth were slightly larger than for the AMC parameter. The decrease in rainfall losses was comparable to saturated soils (AMC 4) and the increase in rainfall losses was comparable to rather dry conditions (AMC 2).

9.4. Mannings 'n' Roughness

The Mannings 'n' roughness coefficient (see Section 5.5) was increased and decreased by 25% respectively for all land types across the study area. The change in adopted values is provided in Table 18.

Table 18: Mannings 'n' Roughness Sensitivity

Land Type	Adopted Mannings 'n'	-25%	+25%
Grass and open space	0.04	0.03	0.05
Dense vegetation and bushland	0.15	0.1125	0.1875
Creek channel	0.05	0.0375	0.0625
Road corridor	0.03	0.0225	0.0375
Residential areas	0.06	0.045	0.075
Commercial and Industrial	0.025	0.01875	0.03125

The changes in peak flood levels with decreasing and increasing the Mannings 'n' roughness values for the 5% AEP and 1% AEP events are shown in Figure H9 to Figure H12.

The results indicate that increasing the surface roughness results in higher peak flood levels across much of the study area. However, the increase in velocity of the runoff in the upper catchment areas results in more water ponding upstream of hydraulic structures that constrict these flows, such as road culverts. Increases in peak flood levels across the urban areas are typically up to 0.05 m, and in the order of 0.1 m in the downstream creek areas. The higher surface roughness means that runoff from the upstream areas is delayed, resulting in a lower volume of water arriving at key constriction points such as culverts at any one time. This reduces the volume of ponding upstream of these structures, with peak flood levels lower by up to 0.05 m.

Conversely, reducing the surface roughness results in lower peak flood levels across much of the study area. This is up to approximately 0.05 m within the urban areas and in the order of 0.2 m within the downstream creek areas. At hydraulic structures there can be an increase in peak flood level up to 0.05 m. The increase in the velocity of the runoff (with lower roughness) in the upper catchment areas results in water arriving quicker, and resulting in more ponding upstream of hydraulic structures that constrict these flows, such as road culverts.

9.5. Structure Blockage

The design flood events adopted a blockage factor of 20% for all cross-drainage culvert structures (see Section 7.7). Two blockage scenarios were modelled and applied to all culvert structures – one with 0% blockage and one with 50% blockage. The change in peak flood level with these scenarios is shown in Figure H13 to Figure H16 for the 5% AEP and 1% AEP events.

The unblocked scenario resulted in a reduction in peak flood level of up to 0.1 m immediately upstream of a structure, with an increase in the downstream reaches. The magnitude of increase on the downstream side was typically about half the magnitude of the increase on the upstream side. The changes in flood level are highly localised, although the most extensive impacts are seen along The Comenarra Parkway crossings.

The increase in structure blockage results in a corresponding increase in peak flood level of typically up to 0.1 m immediately upstream of a structure. For major crossings on The Comenarra Parkway, the increase can be up to 0.8 m in the 5% AEP and 0.3 m in the 1% AEP event. There are corresponding decreases in peak flood level in the downstream reaches, although typically of a much lower magnitude than the increases.

The study area is considered to have low sensitivity to the changes in structure blockage as most changes to flood levels are highly localised.

9.6. Pit Blockage

The influence of the adopted stormwater pit blockage factor was investigated. The design flood events adopted a blockage factor of 20% for on-grade pits and 50% for sag pits (see Section 7.7). These blockage factors were reduced to 0% for all pits and increased to 60% and 75% for on-grade and sag pits, respectively (representing an additional 50% blockage of inlet area that was clear). The change in peak flood level for these two scenarios is provided in Figure H17 to Figure H20 for the 5% AEP and 1% AEP events.

For the unblocked scenario, there is very little change to peak flood levels, even throughout the urban areas where pits are located. The change in peak flood level across the catchment is within ±0.05 m and the changes are not extensive. Typically, the reductions are found at the upstream ends of the stormwater network, where slightly more water can enter the system. Increases can be found at the lower portions of the network, where the capacity of the pipes is reached quicker due to the additional upstream inflows.

For the blocked scenario, it is not anticipated that all pits in a catchment would be simultaneously blocked to this degree, but rather the impact of blocked structures at individual locations can be investigated. Again, there are only small changes in peak flood level, typically within ±0.05 m. There are minor changes within the major creeks in the downstream areas, with the 5% AEP event displaying an increase within the creeks (up to 0.02 m), and the 1% AEP event showing a decrease within the creeks (up to 0.02 m).

The study area is relatively insensitive to pit blockage assumptions.

9.7. Downstream Boundary Conditions

For the design flood events, a stage-discharge relationship was applied to the outlet of the creeks where they discharge to the Lane Cove River. This relationship is determined within the TUFLOW software based on an adopted slope of 1%. To test the sensitivity of this stage-discharge relationship, the slope was decreased by a factor of 10 and increased by a factor of 10. The change in peak flood level for these two scenarios was negligible, and hence these scenarios were not mapped. Thus, the model was insensitive to downstream boundary conditions.

9.8. Climate Change

Climate change is expected to increase sea levels and also short duration rainfall intensities from east coast convective storm events. It is typical practice in catchment flood studies under the NSW flood program to model scenarios incorporating the effects of these impacts from climate change to understand the potential changes in flood behaviour.

Various projections of the likely increases to sea levels are available, however, receiving waters of the Lane Cove River are not influenced by sea level in this area. The Lane Cove River is only tidal up to Lane Cove Weir, located downstream of the study area. As such, sea level rise will not affect the study area. Any increase in design flood rainfall intensities will increase the frequency, depth, and extent of inundation across the catchment. The design rainfall information currently provided by the BoM is based on historical climate data and does not currently include any allowance for likely increases to convective storm rainfall intensity in the future. ARR 2019 (Reference 9, Book 1 Chapter 6) provides some guidance about consideration of the impacts of climate change on design rainfall intensities. It suggests assuming that rainfall intensities can be assumed to scale up by about 5% per degree of average surface warming.

The current NSW State Government's advice recommends sensitivity analysis on flood modelling should be undertaken to develop an understanding of the effect of various levels of change in the hydrologic regime on the project at hand (Reference 23). To understand potential changes to flood behaviour due to increased intensity of rainfall, the 0.5% AEP and 0.2% AEP events were compared with the 1% AEP event (per the relevant guideline, Reference 23). These events provide an indication of how 1% AEP flood levels would change if the rainfall intensity increased to the point that it matches either the current 0.5% AEP (a 7.9% increase in intensity for the 45-minute critical storm event) or 0.2% AEP (a 22% increase in intensity for the 45-minute critical storm event). The change in peak flood level, comparing the 0.5% AEP event and 0.2% AEP with the 1% AEP event can be seen in Figure H21 and Figure H22, respectively.

Comparing the 0.5% AEP event with the 1% AEP event, flood levels are typically increased by up to 0.05 m throughout the urban areas, particularly on overland flow paths where concentration of flow occurs. Peak flood levels increase by approximately 0.1 m to 0.2 m in the downstream creeks through forested areas, with impacts in the range of 0.2 m to 0.4 m upstream of key crossings of The Comenarra Parkway. Comparing the 0.2% AEP event with the 1% AEP event, flood levels are typically increased by approximately 0.1 m to 0.2 m on overland flow paths. In the downstream creeks, peak flood levels typically increase by 0.2 m to 0.4 m, with impacts upstream of key crossings of The Comenarra Parkway in the order of 0.5 m to 1 m.

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ATTACHMENT A: ARR 2019 Datahub Metadata

Attachment A

FIGURES

Figures

Appendix A

APPENDIX A. GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. 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.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of 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).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before 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 diagrammetic 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 Aflood 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 Astandard 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.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual 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 (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or

- major overland 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.

mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State=s rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **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 Awater 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.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

APPENDIX B. TOPOGRAPHIC SURVEY

Appendix B

APPENDIX C. COMMUNITY CONSULTATION MATERIALS

APPENDIX D. MEMORANDUM: DRAINS MODELLING FOR LANE COVE NORTHERN CATCHMENTS FLOOD STUDY

APPENDIX E. CALIBRATION RESULTS

Appendix E

Table E1: Comparison of observed and modelled flood behaviour based on community questionnarie responses

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
002	8 March 2022. Very shallow ankle depth water, in front yard due to water ponding.	0.35	0.34	0.30	0.38	Good	Depth is typically less than 0.1 m deep across front yard. Deeper water simulated on downstream side due to ponding on neighbour's property.
003	March 2022. Shallow calf deep water, caused by overland flow from neighbouring properties and Roland Avenue down both sides of property.	0.29	0.28	0.27	0.33	Good	Shallow overland flows simulated from neighbours and Roland Avenue. Flooding on both sides of the property is evident. Water depths typically shallow, maximum of 0.3 m deep at rear in channel.
004	Deep (greater than knee depth) flooding occurring yearly. Caused by blocked gutters and strong overland flow to water course at boundary.	1.59	1.55	0.83	2.01	Good	Overland flows modelled from Strone Avenue through to Coups Creek. Maximum depths in Coups Creek at the rear of the property.
014	2013. Very shallow water depth in backyard. Caused by inadequate drainage in neighbouring properties.	-	-	-	-	No	Property address not provided. No properties inundated in the vicinity of Iona Avenue and Yarrara Road intersection. Based on description it could be a 'local drainage' issue rather than 'flooding'.
024	March 2022. Very shallow water depth in backyard due to runoff from neighbouring properties.	0.00	0.00	0.00	0.00	No	Only shallow inundation in the gutter of Ashburton Avenue. Observed inundation likely due to very localised catchment not modelled in this study. This is likely to be a 'local drainage' issue rather than 'flooding'.
026	First half of 2012. Shallow water depth (mid-calf height) through backyards of multiple properties in Albion Avenue and Jubilee Avenue. Creek at rear of properties overflowed into these properties.	0.22	0.21	0.15	0.28	Good	Flow path modelled along the rear of properties on Albion Avenue, with depths in the order of 0.1 m to 0.3 m.
027	14-23 March 2021. 10cm deep water cut off access to property and neighbouring properties. Caused by blocked stormwater drains from tree roots and leaves.	0.62	0.61	0.42	0.61	Good	Modelled peak flood depth is against the building. Typical flood depths are in the order of 0.1 m on property (and neighbouring properties) from overflows from Lucinda Avenue South.
037	Evans Street collects and channels water onto the footpath of The Comenarra Parkway, causing erosion.	-	-	-	-	Good	Flood behaviour replicated in the model – water is modelled on Evans Street and this is directed down to the Commenara Parkway. From the Comenarra Parkway it then flows into

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
							Avondale Creek.
039	Ponding in low lying residential pockets adjacent to creeks in Bicentennial Park.	-	-	-	-	Good	While no specific locations were provided, the model simulates inundation on a flow path between Kendall Street and Lofberg Road, upstream of Bicentennial Park.
044	Stormwater drain running through front yard increases its depth but has never reached capacity.	1.45	1.41	1.21	1.59	Fair	Unclear whether this refers to on-site stormwater or the street drainage. The street drainage is estimated to overtop the kerb, with shallow flows through this property. Maximum flood depths are recorded in the upper reaches of Water Dragon Creek that traverses the rear of the lot. Photos provided of the creek (eg. Photo 13).
045	9 February 2020. Shallow depth within immediate vicinity to house. River which runs adjacent to property broke its banks. Blocked drains and overflowing river caused significant overland flow.	2.18	2.14	1.55	2.51	Fair	Overland flow simulated to flow through the property, from both Strone Avenue and Cyrus Avenue. Coups Creek, which runs along the rear of the property reaches considerable depths (as shown in Photo 8).
048	No impact on property. Stormwater drains do not seem to cope with some heavy rainfall events.	0.10	0.08	0.04	0.13	Good	Flow path is evident at the rear of the property, but remains shallow and does not affect dwelling or any substantial amount of land.
051	Flooding occurring on a frequent basis. Shallow depth water through backyard caused by water diversion and runoff from neighbouring golf course.	1.17	1.03	0.53	1.56	No	Flooding only simulated at the front of the property due to watercourse – contained within banks. Runoff from the golf course is captured by another minor creek and does not reach the property. Any inundation of the back yard may be a 'local drainage' issue rather than 'flooding'.
057	No date provided. Deep water flow over road and through neighbouring parks. Roads cut off due to blocked drains.	0.03	0.03	0.03	0.03	Fair	Shallow inundation modelled on Solander Close and at the front of the property. May be referring to deep ponding on Forwood Avenue sag point, which reaches up to 1 m deep in the 2022 event. Shallow flooding also modelled through Comenarra Reserve.
061	9 February 2020. Very shallow water depth in backyard. Caused by tree roots blocking drains and	0.27	0.23	0.00	0.34	Good	Shallow flow modelled from Yarrara Road through property, affecting front and back yards in 2020. Peak depths are at the

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
	forcing overland flow.						rear of the property, where flow is obstructed by neighbouring building. Modelled flooding aligns with photograph provided of the 2020 event (Photo 20).
062	Creek drains well, no flooding issues.	0.90	0.81	0.44	1.00	Good	Modelled flooding remains within the channel through the property in all events. There are shallow overland flows simulated from Timaru Street, however, this is on the opposite side of the creek to the dwelling and unlikely to be observed.
064	No date provided. Flooding frequently occurring. Front yard shallow water depth, backyard very shallow. Flooding caused by run off from neighbouring properties.	0.19	0.19	0.00	0.23	Good	There is a minor flow path through this property, from Chisholm Street to Barwon Avenue. Shallow depths modelled to affect the southern side of this property. Water would appear to come from neighbouring properties.
066	No date provided. Very shallow water depth caused by blocked stormwater drains in Mitchell Crescent.	0.00	0.00	0.00	0.00	Fair	Overflow from Mitchell Crescent is modelled to affect the neighbouring property to the south – it is not modelled to affect this property. There may be local features which may divert water onto this property, however, it is at a higher elevation than Mitchell Crescent.
069	2021. Very shallow water depth in backyard caused by leaves blocking water courses and drains.	-	-	-	-	Fair	House number not provided. There are a number of properties on Finlay Road with shallow flooding in yards. These are primarily in the vicinity of Mildred Street.
070	Frequent flooding in backyard to a very shallow water depth caused by inadequate/old stormwater drains and overland flow from neighbouring properties.	0.15	0.15	0.14	0.16	Good	Shallow inundation modelled across property, flowing from neighbouring lots along the eastern side of the property to Acacia Close.
073	January 2022. Flooding to a very shallow depth caused by overland flow from neighbouring properties (especially Turramurra Shops). Blocked drains and an inadequately sized overland flow easement contribute to the problem.	0.29	0.27	0.18	0.36	Good	Reasonably shallow inundation modelled across the front of this property. The flow path comes from upstream properties, including Turramurra shops on the Pacific Highway. Depths are typically less than 0.2 m along overland flow path.
074	No date provided. Deep water caused by overflow of creek, and from neighbouring properties and roads.	1.22	1.10	0.37	1.45	Good	A major tributary flows through property, with deep water modelled as described. Water flows across Exeter Road

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
	Water overtopping road where drainage is located underneath roadway. Water below driveway bridge, but overtops neighbour's bridge.						(overtopping road as described) and then through properties. Specific bridge details not in model to check, but modelled depths appear to reasonably align with observations.
076	1984. Shallow flooding occurred in backyard where easement is located. Caused by runoff from neighbouring properties and roadway.	0.48	0.23	0.00	0.66	Good	Water modelled to affect property from both neighbouring property to the east and overflows from Binalong Street. These are typically shallow, although deeper water modelled at dwelling itself. Aligns with Photo 4.
077	Water remains within creek banks at rear of property.	0.95	0.85	0.31	1.71	Good	Water modelled to remain within creek banks for all events simulated.
080	No date provided. Shallow flooding caused by overflow from neighbouring properties and inadequate street stormwater drainage capacity.	0.02	0.00	0.00	0.03	Good	Shallow flooding modelled to affect property in the 1991 and 2022 events. This water is from an overland flow path from Boronia Avenue to Yeramba Street. Water would appear to come from neighbouring properties and just affects the south eastern side of the lot.
081	No date provided. Shallow flooding through garage. Caused by overflowing street gutters and water flowing down driveway.	0.24	0.19	0.12	0.26	No	Water on Fox Valley Road is modelled to be contained within the gutters. There is a shallow overland flow path at the rear of the property, from Seymour Close through to the bushland, with shallow depths over the property. Given the steep nature of Fox Valley Road at this location, inundation of the garage is likely to be from 'local drainage' rather than from the road.
082	3 December 2014. Shallow flooding in front yard caused by overflow from blocked street drainage pits. Drains nearly 90% blocked.	0.42	0.41	0.38	0.45	Fair	The property receives overflow from Campbell Drive when street drainage is at capacity, and also overflows from the channel located in the median of Cooper Crescent. Reasonably deep water modelled to pond at the upstream side of the dwelling likely due to trapped floodwaters. Aligns with Photo 7.
083	No date provided. Water to knee depth affecting the back yard (NW corner) and inside home - depth of 300-400mm in basement. Debris from adjacent golf	0.25	0.26	0.15	0.32	Good	House number not provided. Description of flooding aligns with model – flow originates from property at the end of Warrowa Avenue and down eastern boundary of 3 Avondale Place and

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
	course blocking drains and forcing water into property; flow from neighbouring property also causing flooding.						then onto Avondale Place itself. Flood depths given on Avondale Place. Likely to be property opposite this, in which water from Avondale Place flows through.
084	9 February 2020. Very shallow water depth affecting the inside of home and front yard. Caused by pooling of water in front yard and blocked drainage pits. Location estimated (Campbell Drive).	-	-	-	-	Fair	House number not provided. There are several properties located on Campbell Drive that experience shallow inundation in the front yards in the vicinity of Cooper Crescent.
085	December 2021. Blocked stormwater drains on street causing water to pool and spill out of roadway containment. Drains blocked to between 70% and 90%.	1.34	1.23	0.52	1.57	Good	A major tributary flows through property, with deep water modelled overtopping Exeter Road. This water then flows through properties downstream.
087	No date provided. Very shallow flooding in backyard caused by overland flow from neighbouring properties.	0.10	0.00	0.00	0.40	Good	Very shallow flows modelled affecting back yard of property (from neighbouring lots) in the 1991 and 2022 events. Deeper ponding in 2022 is modelled around the house itself.
093	No date provided. Water to a depth of 30cm flows through both front and backyards during heavy downpours. Boundary stormwater pipe reaches capacity and forces water through property.	0.23	0.22	0.00	0.28	Good	Shallow flow modelled from Yarrara Road through property, affecting front and back yards in all events except 2021. Peak depths are at the rear of the property, where flow is obstructed by neighbouring building.
094	April 2018. Very shallow flooding underneath house. Cause by flow from higher neighbouring properties.	0.24	0.13	0.04	0.33	Good	Overflows from Barwon Avenue sag point modelled to flow through properties to Canoon Road. Typically very shallow flows, although water ponds against building in some events.
097	No specific date provided. Water depths between 10cm and 20cm in backyard are caused by ponding in low areas.	0.20	0.18	0.11	0.26	Good	Flow path modelled along the rear of properties on Albion Avenue, with depths in the order of 0.1 m to 0.3 m.
101	1984. Water to a depth of 1m to be flowing across access road to property.	0.61	0.58	0.30	0.57	Fair	Flow path modelled across front of property to Kimbarra Road low point.
102	18 Jan 2012 & 21 Jan 2016. Very shallow flow along boundary of property caused by blocked drains and flow from neighbouring properties.	0.74	0.61	0.24	0.84	Fair	Creek just intersects rear corner of property (maximum flood depths recorded for this location), however, the front of the property also experiences shallow inundation from

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
							Blytheswood Avenue along the northern boundary.
105	1983. Water above knee depth in front yard caused by overflowing of river which runs through front of property, and leaves blocking street drainage.	1.88	1.85	1.62	2.01	Good	A major tributary flows through property at the front, with deep water modelled as described.
109	Multiple flooding events in the 1990s. Ankle deep water through front yard and house caused by inadequate street drainage forcing water down driveway.	0.43	0.37	0.21	0.57	Good	There is a flow path through the rear of the property (where maximum flood depths are recorded), however, shallow inundation (less than 0.1 m deep) from Allawah Road is also modelled down the southern side of the lot.
110	9 February 2020. Knee deep water flowing through front yard and ponding on road caused by inadequate road drainage at bottom of hill. Water coming from neighbouring properties and road.	0.31	0.18	0.12	0.46	Good	Water modelled to break out of Nimbrin Street and flow through property toward Forwood Avenue. This would inundate the front yard. Substantial ponding on Forwood Avenue modelled in the 2020 event (to 1 m deep), aligning with photograph provided (Photo 12).
111	6 February 2010. Shallow water flowing through front and back garden. Caused by diversion of water into inadequate drainage pits. Location estimated.	2.07	2.03	1.03	2.40	Fair	House number not provided. Estimated to be at Doncaster Avenue low point. Shallow overflow from Doncaster Avenue through front of property and inundation from creek in back yard as well.
112	No date provided. Creek at bottom of backyard reported to be blocked with debris.	0.29	0.26	0.13	0.36	Good	Creek runs along rear boundary of property – generally reasonably shallow flows.
113	1986. Shallow water depth in front and back yards of property caused by inadequate drainage at the street. Water flowing down driveway/side of house.	0.04	0.02	0.01	0.04	Good	There is a minor flow path through this property, from Barwon Avenue through to the Lane Cove River. Shallow depths modelled to affect the southern side of this property – along driveway and side of house.
114	1994 and 2001. Shallow water depths flowing through property (and all neighbouring properties) caused by overland flow from Barwon Road toward Cove Street. Blocked drains were reported.	0.17	0.16	0.13	0.18	Good	There is a minor flow path through this property, from Barwon Avenue through to the Lane Cove River. It affects the rear of this property and neighbours on Cove Street. Typically shallow flows less than 0.1 m deep.
115	May 2009. Water levels in creek at front property boundary rose significantly during downpour,	0.86	0.77	0.35	0.97	Fair	Creek modelled at front of property, with peak depths up to 1 m in the events modelled.

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
	estimated to be 1.4m deep.						
118	No date provided. Shallow water depth on road caused by inadequate capacity of street drainage.	0.47	0.43	0.24	0.61	Fair	Shallow inundation modelled on Duff Street, with depths at the sag point reaching 0.45 m in the 2022 event. Water overflows from the sag point through the property. This is typically shallow, however peak depths are within a channel along the northern boundary of the property.
122	Frequent flooding in the last 4 years. Shallow water depth through front and backyard and through home caused by overland flow from newly developed neighbouring properties.	0.15	0.15	0.00	0.17	Good	There is a minor flow path through this property, from Chisholm Street to Barwon Avenue. Shallow depths modelled to affect the southern side of this property. Water would appear to come from neighbouring properties. Water not simulated near dwelling – this may be a 'local drainage' issue rather than 'flooding'.
125	No date provided. Knee deep water flowing through backyard, caused by overflowing of river at property boundary.	2.15	2.11	1.51	2.59	Good	Coups Creek runs along the rear boundary of the property. Water is modelled to be contained within the channel, but may still be part of the 'backyard' that is referred to.
129	March 2019. Ankle deep water flow through front yard caused by blocked street drainage pits and overflow from neighbouring properties.	0.47	0.08	0.06	0.48	Good	Shallow inundation through front yard (less than 0.1 m) due to overflow from Lucinda Avenue, although at the property it may appear to come from neighbouring lots. There is a flow path through the back yard as well, with an area of ponding where peak flood depths are modelled to occur.
130	Pre 2010. Knee deep water flowing through front and backyard (up to 10cm in areas). Caused by blocked drainage pipe at property boundary, and inadequate stormwater capacity.	0.70	0.61	0.15	0.88	Fair	Water modelled to affect property from neighbouring property to the rear. Flow modelled in back and front yards, with flow along the eastern side of the house. This is typically shallow, however, deeper water is modelled at the garage location. Water flows from garage out to Kendall Street sag point with depths in the range of 0.1 m to 0.4 m.
132	January 2022. Calf level water depth over road between #20 and #21 due to inadequate street drainage capacity.	0.16	0.13	0.00	0.31	Good	Sag point on Amaroo Avenue is located between numbers 20 and 21. Peak flood depths on the road in the 2022 event reach approximately 0.4 m, aligning with observations of calf-level

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
							water depth. Peak flood depths are provided for the property, which is modelled to experience shallow overland flows through the front yard from Tandema Street to Amaroo Avenue.
133	~1970. Calf level water depth contained within road boundary, caused by inadequate drainage and overflow from school oval.	0.15	0.23	0.00	0.16	Fair	Shallow inundation modelled on the property. Flooding also modelled on Diana Avenue due to runoff from school fields. Due to the gradient of the road, this remains shallow (less than 0.1 m) and within the road corridor.
134	No date provided. Ankle depth water in front yard due to runoff from neighbouring properties and street. Creek once ran through front of property, now altered course.	0.05	0.05	0.04	0.11	Good	Water modelled to just break out of Marshall Avenue at the location of the property and follow the old creek line. Only a small portion of the front yard is affected, with shallow depths simulated. ‘Local drainage’ issues may also contribute to inundation of front yard.
135	No date provided. Calf depth flooding over driveway caused by overtopping of water course/creek on boundary of property.	1.32	1.23	0.75	1.38	Good	Driveway access subject to flood depths of up to 1.4 m (likely under bridge structure). Could easily be calf depth on top of bridge structure.
138	December 2020. Ankle depth flooding in front yard caused by overtopping of drainage channel due to blocked culvert. Culvert 90% blocked.	1.08	1.00	0.73	1.18	Good	Shallow inundation of yard (less than 0.1 m) modelled adjacent to watercourse. Blockage of culvert under Forwood Avenue would also contribute to inundation on lot.
139	2011. Flooding to 2cm depth in home and front yard caused by incorrect placement of drainage pit on street. Water overtopped gutters and spilled down driveway.	0.18	0.15	0.08	0.24	Good	Water from Antoinette Close modelled to break out and flow through the front yard and around south eastern side of the house toward a creek channel. Shallow inundation modelled (less than 0.1 m), although peak flood depths occur at the dwelling (where the garage is located).
140	No date specified. Calf depth water affecting both front and back yard, caused by blocked drains, ponding within road and overflow from neighbouring properties.	0.80	0.78	0.70	0.86	Good	Flow path is modelled at the rear of the property only, with peak flood depths due to the upstream ponding location. Flows are typically shallow (0.1 m to 0.3 m deep) downstream of this where the respondent noted flooding issues in front yards and back yards. This flow path traverses front and back yards

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
							along the property numbers mentioned.
143	No date provided. Knee deep water affecting backyard and roadway. Caused by floodwater diversion infrastructure from neighbouring golf course, forcing water inter properties.	0.31	0.34	0.00	0.46	Good	Description of flooding aligns with model – flow originates from property at the end of Warrowa Avenue (from golf course) and down eastern boundary of 3 Avondale Place and then onto Avondale Place itself.
144	No date provided. 5cm water depth across driveway and front lawn. Caused by inadequate capacity of street stormwater drainage, forcing water down driveway.	0.00	0.00	0.00	0.00	No	No flooding modelled on Finlay Road at this location, and no inundation on property. May be due to 'local drainage' rather than 'flooding', given the steep nature of Finlay Road and the very small catchment area that drains past the property.
145	No date provided. No significant issues, although blocked street drainage pits noted.	0.05	0.05	0.04	0.11	Fair	Water modelled to just break out of Marshall Avenue near the property and follow the old creek line. Only a small portion of the front yard is affected, with shallow depths simulated.
146	2021. Knee deep water through backyard and road, caused by overgrowth of vegetation surrounding drainage easements, blocking water passage.	0.00	0.00	0.00	0.00	Fair	Water modelled to only affect Avondale Place at this location. There may be additional runoff from the golf course at the rear that is not part of the modelling ('local drainage' rather than 'flooding'). Flooding on Avondale Place itself is in the range of 0.2 m to 0.5 m.
149	No dates provided. Nearby creek rises significantly during downpours, but has always been contained water within banks.	1.29	1.20	0.49	1.61	Good	Creek at the rear of the property has significant depths simulated (up to 1.6 m in 2022), however is contained within the banks.
150	Frequent flooding. Knee deep water affecting access driveway due to blocked drains from debris flowing from Turramurra Plaza.	0.17	0.16	0.09	0.23	Fair	Flow path is modelled at the front of the property. Flows are typically shallow (0.1 m to 0.3 m deep) in the vicinity of the driveway that is noted by the respondent. Wate originates from Turramurra Plaza upstream of this location.
152	January 2010. Calf deep water flowing through backyard and garage due to inability for street drainage and easement to cope with water redirected from neighbouring golf course.	0.00	0.00	0.00	0.20	Fair	Water modelled to only affect Avondale Place and creek adjacent to property at this location. There may be additional runoff from the golf course at the rear that is not part of the modelling ('local drainage' rather than 'flooding'). Flooding on Avondale Place itself is in the range of 0.2 m to 0.5 m.

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
153	2020. Calf level water over road caused by blocked street drains (including Doncaster Avenue culvert) and excess water being diverted from a neighbouring golf course.	1.65	1.58	0.46	2.78	Good	Creek flows along eastern boundary of the property where peak flood depths are modelled. Property is also affected by shallow overland flows from Coventry Place in the 2022 event. Flows generally remain within the channel, although a small portion of the back yard is modelled to be impacted by shallow overflows. On Doncaster Road, peak flood depths reach approximately 0.35 m in the 2020 event, aligning with observations. Blockage of the Doncaster Avenue culvert would also contribute to flooding on Doncaster Avenue.
154	February 2020. Ankle deep water affecting the front yard and the inside of the home. Caused by blocked drains, which led to overflow down the driveway, ponding and runoff from neighbouring properties.	-	-	-	-	Fair	House number not provided. Near the corner of Campbell Drive and Rainforest Close there is shallow inundation (less than 0.1 m) of lots due to overflows from neighbouring properties.
157	No date provided. During heavy downpours, nearby creek rises significantly due to blocked drains and water courses.	2.09	1.90	0.42	2.37	Good	Creek at the rear of the property has significant depths simulated (up to 2.4 m in 2022), however, appears to just inundate areas outside the channel in the 2020 event, with 1991 and 2022 showing inundation beyond the main creek channel, but still largely contained and not impacting the dwelling.
160	December 2020. Knee deep water flowing across driveway due to water flowing from roadway and neighbouring properties. Drains reported to be 50% blocked in some cases.	0.13	0.12	0.09	0.15	Fair	Depth is typically less than 0.1 m deep across front yard.
161	March 2021. Ankle deep water flowing and ponding from neighbouring properties (back left), flowing through house and garage.	0.02	0.02	0.02	0.03	No	Water on neighbouring lot modelled to flow from Konda Place through to Forwood Avenue, not impacting this property. The very northern top of the lot is affected by shallow overland flows from the neighbouring property, however this does not affect the dwelling. Above floor flooding experienced likely to be from 'local drainage' rather than 'flooding'.

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
166	No date provided. Frequent, calf level flooding across road caused by blocked drains. Deep ponding within roadway.	0.00	0.00	0.00	0.00	Fair	No flooding modelled on property, however, the respondent notes flooding on roads and mentions Congham Creek. The creek commences downstream of Wallalong Crescent, and flooding on upstream roads (such as Wallalong Crescent and Diana Avenue) are modelled to be shallow (less than 0.1 m).
167	During 2018 and 2019. Ankle deep water flowing down driveway and front yard. Caused by blocked street drains, forcing water out of roadway and into property.	0.05	0.05	0.05	0.25	Fair	Flow is modelled on properties mentioned by the respondent due to overflows from Avon Road. Modelled to continue along overland flow path toward a creek channel rather than impact front yard. This may be contributed to by 'local drainage' inundation on the property.
171	January 2016. Calf level flooding in front yard, back yard and inside home. Caused by inadequate capacity of street drainage and blocked drainage pits forcing water into properties.	0.03	0.02	0.01	0.04	No	No water modelled on Roland Avenue outside property. Roland Avenue is on a constant grade. The first drain on the western side of the road is located downstream of the property entrance. Water not modelled to break out of the road until the sag point on Roland Avenue is reached. This may be regarded as 'local drainage' inundation rather than 'flooding'. There is shallow inundation at the rear of the property modelled as a creek line runs along the back property boundary.
177	No date provided. Creek at boundary of property rises very rapidly during heavy rain events. 2m wide, 1m deep water through 'creek' in backyard. Caused by blocked street drainage and runoff from neighbouring properties.	0.91	0.84	0.54	1.14	Good	Creek line runs through backyard of property, modelled to be up to 1.1 m deep in the 2022 event.
182	No date provided. Rapid rising of creek in backyard during heavy downpours.	0.22	0.22	0.22	0.25	Good	There is a flow path that crosses the property, with depths between 0.2 m and 0.3 m. This is the upper reaches of Peppermint Creek.
184	3 times since 1997 (incl 25 April 2019). Ankle deep water through backyard (from Beechworth Road), and knee deep water ponding across Troon Place. Caused	0.00	0.00	0.00	0.00	No	No inundation modelled on property. Photo 9 and Photo 10 clearly indicate flows entering the back yard. Overflows from Beechworth Road are modelled to flow west from the cul-de-

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
	by significant blockages and inadequate street drainage.						sac and not impact Troon Place. There is, however, still a reasonable area between Beechworth Road and Troon Place, in which local runoff could be generated and flow rapidly down the steep slopes, impacting this property. This would be considered to be 'local drainage' issues rather than 'flooding'. The Troon Place bridge is not estimated to have been overtopped in the modelled events, ponding on the road remains shallow (less than 0.1 m).
185	No date provided. Nearby Coups Creek rises rapidly during heavy downpours, but does not reach house - passes under Lucinda Avenue.	3.62	3.49	0.89	3.92	Good	Coups Creek crosses the front of this property. The Lucinda Avenue culvert modelled to be at capacity in the 1991 and 2022 events, with water flowing over Lucinda Avenue in these events. The dwelling, however, remains flood free.
186	No date provided. Calf deep water ponding in backyard due to blocked stormwater drains at rear of property.	0.12	0.10	0.04	0.22	Good	Overland flow path modelled from Jubilee Avenue to Kimbarra Road. A second flow path along the rear of the property joins this one immediately downstream of the property.
187	No date provided. Frequent flooding across Campbell Drive at Rainforest Crescent due to blocked and inadequate capacity drains.	0.00	0.00	0.00	0.00	Good	No flooding modelled on property, however, substantial flooding on Campbell Drive at Rainforest Crescent is modelled. Shallow inundation (less than 0.1 m) is modelled in all events, except for the 2022 event where the culvert capacity under Campbell Drive is exceeded and water overtops the road, to a maximum depth of approximately 0.5 m.
190	No date provided. Knee deep water through front yard and garage, due to water being forced down driveway from blocked street drainage and water coming from neighbouring properties.	0.14	0.13	0.11	0.18	Fair	Flow modelled from neighbouring lots through backyard toward Iona Avenue sag point. This travels along the southern boundary of the site, likely to affect the garage at ground level on that side of the building. The modelled depths remain shallow (less than 0.2 m).
191	Late 1980s/early 90s. Knee deep water surrounding house, and ankle deep water through garage. Caused by severe street drain blockages and new	0.21	0.16	0.08	0.25	Fair	Street name appears to be a typo – estimated as Ulm Avenue. Shallow overland flows affect this lot at the rear. Potential to affect garage at ground level. Overflows from Barwon Avenue

ID[1]	Summary of Observation	Modelled Peak Flood Depths at Property or Road[2] (m)				Comparison with Modelled Flooding[3]	
		1991	2020	2021	2022	Match	Comments
	developments built over a previous natural water course. Location estimated.						sag point modelled to flow through properties to Canoon Road, although does not typically reach knee deep.
192	No date provided. Knee deep water flowing through backyard and roadway. Caused by inadequate drain capacity during heavy downpours. Flows from Auluba Reserve into Chisholm Street sag, down battle axe driveway, through rear of properties and Down Cove St. New pipe from Chisholm Street to Cove Street helps.	0.11	0.11	0.00	0.14	Fair	There is a minor flow path through this property, from Chisholm Street to Barwon Avenue. Shallow depths modelled to affect the southern side of this property. Water would appear to come from neighbouring properties.

1. Locations shown in Figure 6.
2. Maximum depth over entire property or on road
3. The following depth ranges were adopted based on respondent descriptions:
   Ankle Depth = 0 m to 0.2 m deep
   Calf Depth = 0.2 m to 0.4 m deep
   Knee Depth = 0.4 m to 0.7 m deep
   Above knee depth = greater than 0.7 m deep

APPENDIX F. DESIGN FLOOD MAPS

APPENDIX G. DESIGN FLOOD RESULTS

Appendix G

Table G1: Peak Flood Levels (mAHD) at Key Locations

ID[1]	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
CC01	Exeter Road	159.4	159.5	159.5	159.6	159.6	159.6	159.7	160.3
CC02	Koora Avenue	177.1	177.2	177.2	177.2	177.3	177.3	177.3	177.7
CC03	Ada Avenue	171.6	171.7	171.8	171.8	171.8	171.9	171.9	172.5
CC04	Tanderra Street	169.2	169.3	169.4	169.4	169.5	169.5	169.5	169.9
CC05	Amaroo Avenue	165.6	165.7	165.8	165.9	165.9	165.9	166.0	166.5
CC06	Mahratta Avenue	161.9	162.0	162.0	162.1	162.2	162.2	162.3	163.6
CC07	Lucinda Avenue	158.9	158.9	159.0	159.1	159.1	159.2	159.2	160.0
CC08	The Comenarra Parkway at Coups Creek	119.0	119.0	119.1	119.2	119.3	119.4	119.5	121.1
PC01	Cooper Crescent North	83.0	83.0	83.2	83.4	83.5	83.6	83.7	84.9
PC02	Cooper Crescent South	83.1	83.1	83.2	83.4	83.5	83.6	83.7	84.9
PC03	Campbell Drive at Peppermint Creek	83.1	83.1	83.2	83.4	83.4	83.5	83.6	84.9
PC04	Campbell Drive at Clyde Place	92.3	92.5	92.5	92.5	92.5	92.5	92.6	92.9
PC05	Campbell Drive at Rainforest Close	74.5	74.6	74.7	74.8	74.9	74.9	75.0	75.7
PC06	The Comenarra Parkway at Peppermint Creek	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.9
WC01	Cornwall Avenue	124.1	124.2	124.2	124.2	124.3	124.3	124.3	124.7
WC02	Duff Street	115.4	115.4	115.4	115.5	115.5	115.5	115.5	115.9
WC03	Holmes Street	98.5	98.6	98.8	98.9	99.0	99.0	99.1	101.1
WC04	Monteith Street	96.7	96.8	96.9	96.9	97.0	97.0	97.1	100.5
WC05	Rothwell Road	93.2	93.3	93.5	93.6	93.7	93.8	93.9	94.8
WC06	The Comenarra Parkway at Water Dragon Creek	73.0	73.0	73.0	73.0	73.0	73.1	73.1	74.5
AC01	Forwood Avenue	66.8	66.9	67.0	67.0	67.1	67.1	67.2	68.2
AC02	The Comenarra Parkway downstream of Forwood Avenue	59.0	59.1	59.2	59.2	59.3	59.3	59.4	60.1
AC03	Warragal Road	123.7	123.8	123.8	123.9	123.9	123.9	123.9	124.4
AC04	Quadrant Close	77.9	78.0	78.0	78.0	78.0	78.1	78.1	78.3
AC05	St Andrews Drive	72.9	72.9	72.9	72.9	73.0	73.0	73.1	73.7

ID[1]	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
AC06	Troon Place	68.9	69.0	69.1	69.3	69.4	69.5	69.6	71.3
AC07	The Comenarra Parkway at Avondale Creek	36.9	36.9	36.9	36.9	37.6	37.8	38.1	39.3
AC08	Greenway Drive	78.1	78.1	78.4	78.6	78.7	78.7	78.8	79.5
AC09	Warrowa Avenue	74.3	74.3	74.4	74.5	74.6	74.6	74.7	75.4
AC10	Avondale Place	71.7	71.8	71.8	71.8	71.8	71.9	71.9	72.3
AC11	Doncaster Avenue	58.0	58.1	58.2	58.4	58.5	58.6	58.7	60.3
HC01	Patterson Avenue	66.5	66.5	66.7	67.0	67.0	67.1	67.1	67.6
QC01	Kendall Street	71.2	71.3	71.4	71.5	71.5	71.6	71.6	72.5
QC02	Lofberg Road west	66.2	66.4	66.5	66.7	66.8	66.8	66.9	67.4
QC03	Lofberg Road upstream Norman Griffiths Oval	73.6	73.6	73.7	73.7	73.7	73.7	73.7	74.1
QC04	Ryde Road	78.0	78.1	78.2	78.2	78.3	78.3	78.3	78.5
QC05	Yanko Road	53.7	53.8	53.9	54.0	54.1	54.2	54.2	55.1

1. Locations shown on Figure 38

Table G2: Peak Flood Depths (m) at Key Locations

ID[1]	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
CC01	Exeter Road	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.7
CC02	Koora Avenue	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.7
CC03	Ada Avenue	0.1	0.1	0.2	0.3	0.3	0.3	0.4	1.0
CC04	Tanderra Street	0.3	0.4	0.4	0.5	0.5	0.5	0.5	1.0
CC05	Amaroo Avenue	0.3	0.4	0.5	0.5	0.6	0.6	0.6	1.2
CC06	Mahratta Avenue	0.4	0.4	0.5	0.6	0.7	0.7	0.8	2.1
CC07	Lucinda Avenue	0.3	0.3	0.4	0.5	0.5	0.5	0.6	1.4
CC08	The Comenarra Parkway at Coups Creek	-	-	0.1	0.2	0.3	0.4	0.5	2.0
PC01	Cooper Crescent North	-	-	0.3	0.5	0.6	0.6	0.7	2.0
PC02	Cooper Crescent South	-	-	0.2	0.3	0.4	0.5	0.6	1.8
PC03	Campbell Drive at Peppermint Creek	-	-	0.2	0.3	0.4	0.5	0.6	1.8
PC04	Campbell Drive at Clyde Place	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.8
PC05	Campbell Drive at Rainforest Close	0.3	0.5	0.6	0.7	0.7	0.8	0.8	1.6
PC06	The Comenarra Parkway at Peppermint Creek	-	-	-	-	-	-	-	0.9
WC01	Cornwall Avenue	0.5	0.5	0.6	0.6	0.6	0.6	0.7	1.0
WC02	Duff Street	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.8
WC03	Holmes Street	1.3	1.4	1.5	1.6	1.7	1.7	1.8	3.8
WC04	Monteith Street	0.6	0.6	0.7	0.8	0.8	0.9	1.0	4.4
WC05	Rothwell Road	-	0.2	0.3	0.5	0.6	0.6	0.7	1.7
WC06	The Comenarra Parkway at Water Dragon Creek	-	-	-	-	-	-	-	1.5
AC01	Forwood Avenue	1.2	1.3	1.4	1.5	1.5	1.6	1.6	2.6
AC02	The Comenarra Parkway downstream of Forwood Avenue	0.3	0.4	0.5	0.5	0.6	0.6	0.6	1.4
AC03	Warragal Road	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.8
AC04	Quadrant Close	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.7
AC05	St Andrews Drive	-	-	-	-	0.1	0.1	0.2	0.8

ID[1]	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
AC06	Troon Place	-	-	-	-	-	-	-	0.8
AC07	The Comenarra Parkway at Avondale Creek	-	-	-	0.1	0.7	0.9	1.2	2.4
AC08	Greenway Drive	0.2	0.2	0.5	0.7	0.7	0.8	0.9	1.6
AC09	Warrowa Avenue	0.1	0.2	0.3	0.4	0.5	0.5	0.6	1.3
AC10	Avondale Place	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.9
AC11	Doncaster Avenue	0.3	0.4	0.5	0.7	0.8	0.9	1.0	2.6
HC01	Patterson Avenue	-	-	0.2	0.4	0.5	0.5	0.6	1.1
QC01	Kendall Street	0.3	0.5	0.6	0.6	0.7	0.7	0.8	1.6
QC02	Lofberg Road west	-	0.1	0.3	0.4	0.5	0.6	0.6	1.2
QC03	Lofberg Road upstream Norman Griffiths Oval	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.7
QC04	Ryde Road	0.2	0.2	0.3	0.3	0.4	0.4	0.4	0.6
QC05	Yanko Road	0.4	0.5	0.6	0.7	0.8	0.8	0.9	1.8

1. Locations shown on Figure 38

Table G3: Peak Flows (m^3/s) at Key Locations

ID[1]	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
CC01	Exeter Road Culverts	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.3
	Exter Road Overtopping	5.3	6.9	8.5	10.2	11.8	12.9	15.0	65.9
CC02	Koora Avenue Overtopping	0.6	1.0	1.3	1.6	1.9	2.1	2.4	15.6
CC03	Ada Avenue Culvert	2.9	3.1	3.1	3.1	3.1	3.2	3.1	3.0
	Ada Avenue Overtopping	0.5	1.4	2.7	4.3	5.6	6.7	8.0	44.5
CC05	Amaroo Avenue Culvert	4.2	4.4	5.1	5.2	5.2	5.2	5.3	5.1
	Amaroo Avenue Overtopping	0.5	0.8	1.9	4.0	5.6	6.7	8.5	50.9
CC06	Mahratta Avenue Culverts	2.9	3.0	3.1	3.2	3.2	3.2	3.2	3.3
	Mahratta Avenue Overtopping	0.5	0.8	1.9	4.0	5.6	6.7	8.5	50.9
CC07	Lucinda Avenue Culvert	6.8	7.5	7.4	7.4	7.4	7.5	7.6	7.5
	Lucinda Avenue Overtopping	3.1	5.0	7.6	12.1	15.7	18.2	22.4	120.7
CC08	The Comenarra Parkway at Coups Creek Culverts	8.8	11.6	14.9	21.2	27.1	30.9	32.4	37.1
	The Comenarra Parkway at Coups Creek Overtopping	0.3	0.4	0.7	5.2	9.4	12.2	21.8	286.1
PC03	Campbell Drive at Peppermint Creek Culvert	6.0	6.9	7.6	7.8	7.9	7.9	8.0	8.9
	Campbell Drive at Peppermint Creek Overtopping	-	-	1.1	5.1	7.7	9.5	12.2	80.1
PC04	Campbell Drive at Clyde Place Culverts	0.7	1.4	1.2	1.2	1.1	1.2	1.4	1.9
	Campbell Drive at Clyde Place Overtopping	0.3	0.8	1.4	1.7	2.0	2.3	2.9	21.6
PC05	Campbell Drive at Rainforest Close Culvert	3.6	3.5	3.7	3.6	3.6	3.7	3.6	3.6
	Campbell Drive at Rainforest Close Overtopping	0.1	0.9	2.1	3.7	5.2	6.3	8.2	57.6
PC06	The Comenarra Parkway at Peppermint Creek Culvert	15.2	17.6	20.0	23.5	25.4	26.6	28.4	32.4
	The Comenarra Parkway at Peppermint	0.5	0.7	1.0	1.5	1.8	2.0	2.5	124.3

ID[1]	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
	Creek Overtopping								
WC01	Cornwall Avenue Culvert	0.3	0.4	0.5	0.6	0.7	0.8	0.8	0.9
	Cornwall Avenue Overtopping	0.8	1.0	1.3	1.4	1.6	1.8	2.1	11.8
WC02	Duff Street Culvert	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.7
	Duff Street Overtopping	1.3	1.8	2.3	2.7	3.1	3.4	4.0	18.0
WC03	Holmes Street Culverts	2.5	3.9	3.9	3.9	3.9	3.9	3.9	3.9
	Holmes Street Overtopping	5.7	8.3	12.4	16.7	20.8	23.7	29.2	160.0
WC04	Monteith Street Culvert	3.1	3.4	3.7	4.0	4.2	4.4	4.6	5.1
	Monteith Street Overtopping	10.7	13.8	17.2	21.2	25.1	27.8	33.0	161.1
WC05	Rothwell Road Culvert	13.6	14.8	14.9	14.9	14.9	14.9	14.9	15.2
	Rothwell Road Overtopping	-	2.4	6.7	11.6	16.3	19.8	25.7	156.2
WC06	The Comenarra Parkway at Water Dragon Creek Culverts	7.9	8.5	9.0	9.8	10.2	10.4	10.9	12.5
	The Comenarra Parkway at Water Dragon Creek Overtopping	-	-	-	-	-	-	-	171.5
AC02	The Comenarra Parkway downstream of Forwood Avenue Culverts	8.0	8.9	9.5	10.1	10.4	10.6	11.0	12.9
	The Comenarra Parkway downstream of Forwood Avenue Overtopping	2.3	4.4	7.8	11.4	14.7	16.9	20.9	120.4
AC03	Warragal Road Culvert	1.4	1.5	1.6	1.7	1.8	1.8	1.9	3.2
	Warragal Road Overtopping	2.5	3.6	4.8	5.6	6.6	7.3	8.6	41.7
AC04	Quadrant Close Culvert	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2
	Quadrant Close Overtopping	0.2	0.3	0.4	0.7	0.7	0.8	1.0	5.3
AC05	St Andrews Drive Culvert	6.1	7.2	9.8	13.0	15.0	16.5	19.0	27.1
	St Andrews Drive Overtopping	-	-	0.1	0.2	0.3	0.3	0.4	56.0
AC06	Troon Place Bridge	6.5	7.6	10.5	14.2	16.4	18.0	20.9	90.6
AC07	The Comenarra Parkway at Avondale Creek Culvert	18.2	20.7	23.8	28.4	30.1	30.3	30.3	30.3
	The Comenarra Parkway at Avondale Creek Overtopping	-	-	-	-	5.9	20.2	45.1	355.2

ID[1]	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
AC08	Greenway Drive Culvert	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	Greenway Drive Overtopping	2.2	4.1	7.6	10.4	13.0	14.2	17.5	75.2
AC09	Warrowa Avenue Culvert	6.8	8.0	8.4	8.5	8.5	8.5	8.5	8.5
	Warrowa Avenue Overtopping	0.1	0.3	1.7	4.5	7.3	8.6	12.1	77.0
AC10	Avondale Place Culvert	0.7	0.7	0.8	1.0	1.0	1.1	1.2	2.0
	Avondale Place Overtopping	1.4	2.0	2.7	3.8	4.6	5.1	6.2	34.6
AC11	Doncaster Avenue Culvert	11.1	12.9	12.9	14.0	13.0	14.0	13.0	12.8
	Doncaster Avenue Overtopping	-	0.1	1.1	5.8	11.1	14.0	20.5	144.4
HC01	Patterson Avenue Culvert	0.9	1.1	1.1	1.1	1.1	1.1	1.1	1.2
	Patterson Avenue Overtopping	-	-	-	0.1	0.5	0.7	1.1	9.4
QC01	Kendall Street Culvert	2.3	2.6	2.8	3.0	3.2	3.3	3.5	4.1
	Kendall Street Overtopping	0.5	1.2	2.1	3.1	4.0	4.6	6.0	37.0
QC02	Lofberg Road west Culverts	5.3	6.4	7.6	7.8	7.8	7.9	8.1	8.2
	Lofberg Road west Overtopping	-	0.2	1.3	2.9	4.4	5.5	7.5	55.5
QC03	Lofberg Road upstream Norman Griffiths Oval Culvert	1.5	1.7	1.8	1.9	1.9	1.9	1.9	2.3
	Lofberg Road upstream Norman Griffiths Oval Overtopping	1.0	1.5	2.1	2.8	3.3	3.7	4.6	26.4
QC05	Yanko Road Culvert	5.0	5.4	5.6	6.0	6.2	6.4	6.6	9.2
	Yanko Road Overtopping	4.8	6.8	9.6	14.0	17.4	20.3	25.1	120.5

1. Locations shown on Figure 38

APPENDIX H. SENSITIVITY RESULTS