NORTHERN BEACHES COUNCIL

GREENDALE CREEK FLOOD STUDY

FINAL

wma water

JULY 2023

Level 2, 160 Clarence Street
Sydney, NSW, 2000

Tel: (02) 9299 2855
Fax: (02) 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

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Project **Greendale Creek Flood Study**	**Project Number** **118094**
Client Northern Beaches Council	**Client's Representative** Valerie Tulk Duncan Howley
Authors Michael Reeves Ben Oakley Rhys Hardwick Jones	**Date** 28 July 2023
Prepared by	**Verified by**

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Northern Beaches Council
425 Pittwater Rd
Dee Why NSW 2099
council@northernbeaches.nsw.gov.au
1300 434 434

GREENDALE CREEK FLOOD STUDY

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FOREWORD

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

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

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

1. ***Flood Study***
   - Determine the nature and extent of the flood problem.
2. ***Floodplain Risk Management Study***
   - Evaluates management options for the floodplain in respect of both existing and proposed development.
3. ***Floodplain Risk Management Plan***
   - Involves formal adoption by Council of a plan of management for the floodplain.
4. ***Implementation of the Plan***
   - Construction of flood mitigation works to protect existing development, use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

The Greendale Creek Flood Study constitutes the first stage of the management process for the catchment. This study has been prepared by WMAwater for Northern Beaches Council and was undertaken to provide the basis for future management of flood liable lands within the study area.

EXECUTIVE SUMMARY

The Greendale Creek Flood Study catchment area is located within the Northern Beaches Council Local Government Area (LGA). The study area includes the suburbs of Beacon Hill, Brookvale, Curl Curl, Freshwater and North Curl Curl, comprising a total area of approximately 482 ha (0.48 km^2). Curl Curl Lagoon is a significant feature of the lower catchment with Greendale Creek and the majority of the study area draining to it, while some of the eastern-most portion of the catchment to the north and south drains directly to the ocean. The surface area of Curl Curl Lagoon is approximately 5.7 ha (0.057 km^2), making up approximately 12% of the catchment area. The urbanised part of the catchment consists of commercial and light industrial development in the lower areas, and residential development in the middle and upper catchment areas. "The Kilns" development is a notable feature of the upper catchment which is bounded to the north and west by steep, natural forested land with sharply incised streams.

The Greendale Creek catchment upstream of Warringah Road (A38) consists of a mix of residential development, sporting fields and natural forested land with drainage infrastructure passing under the road. The catchment downstream of The Kilns is primarily urban residential and industrial areas, with grassed sports fields downstream of Harbord Road (built over a former rubbish dump). The catchment consists of a mix of pervious and impervious surfaces with piped and overland flow drainage systems.

The work undertaken in this study includes:

- preparing suitable models of the catchment and floodplain for use in a subsequent Floodplain Risk Management Study;
- estimating flood behaviour in terms of design flood levels, depths, velocities, flows and flood extents within the study area;
- preparing maps of provisional hydraulic categories and provisional hazard categories;
- determining flood planning levels and the flood planning area;
- preparing information for emergency response planning; and
- assessing the sensitivity of flood behaviour to potential climate change effects such as increases in rainfall intensities and sea level rise.

COMMUNITY CONSULTATION

A questionnaire was distributed to residents in the study area in August 2019. The purpose of the questionnaire was to identify which residents had experienced problems with flooding and to collate historical flood data. A total of 113 responses relating to flooding within the Greendale Creek catchment were received from the distributed questionnaires, via both written and online submissions.

Of the responses received, 34 respondents had experienced flooding due to floodwater or stormwater, with 30 respondents indicating that their home was affected and 4 indicating that their business was affected. One respondent indicated that their main building was affected by above floor level flooding with the remainder indicating that flooding affected their garage, yard or other parts of their property.

A draft version of the report was placed on public exhibition to invite comment from the community. One-on-one sessions with Council and WMAwater staff were also held to provide residents with an opportunity to discuss the study and its implications. A total of 74 one-on-one appointments were attended by residents of the study area and 28 formal submissions were received. The primary concerns for the community were how the study was going to affect insurance premiums, house prices and future development potential. Some changes were made to the flood model and Flood Planning Area based on community feedback.

MODELLING SUMMARY

Estimation of flood behaviour in the catchment was undertaken as a two-stage process consisting of:

1. Hydrologic modelling to convert rainfall estimates to overland flow runoff;
2. Hydraulic modelling to estimate overland flow distributions, flood levels and velocities.

MODEL CALIBRATION

There is only limited data for model calibration. While there are good records of the water level within the lagoon, the records of overland flow and flooding throughout the catchment are sparse and generally qualitative descriptions, rather than recorded flood levels for specific events. The November 2018 event was chosen for model calibration and the modelled flood behaviour was compared to the gauged water level hydrograph at the Curl Curl gauge (213426), and qualitative descriptions of flooding, obtained from Council's customer complaints database, local flood investigations and community consultation responses.

The accuracy of the resulting design flood information is largely dependent upon the ability of the modelling system to accurately replicate historical flood data. As relatively few historical records are available for calibration the accuracy of the design flood levels is likely to be around +/- 0.3m to +/- 0.5m. This level of accuracy is typical of systems like Greendale Creek in the Sydney basin. The accuracy can be significantly improved upon if all flood related information (peak levels, velocity estimates, flood extents, photographs, videos etc) in future events is recorded and used for model calibration.

DESIGN FLOOD ESTIMATION

Design flood information was obtained by including design rainfalls from the Bureau of Meteorology into the hydrologic model and then inputting the flows into the hydraulic model. The critical storm duration (duration that produces the highest flood level) was determined based on the mean of 10 temporal patterns for each duration and varied across the catchment from 30 to 180 minutes, although the variation was relatively minor and the 45 minute design storm burst was found to adequately represent the typical behaviour. Design flood information is provided in Appendix D, E and F.

SENSITIVITY ANALYSIS

Sensitivity analysis was undertaken to test the impact of varying the following key parameters:

- catchment lag factor (C);
- rainfall losses;
- roughness;
- structure blockage;
- energy losses;
- initial water level;
- downstream ocean levels; and
- increases to rainfall intensity and sea level resulting from climate change.

The input having the most widespread influence on flood levels was rainfall intensity.

INFORMATION TO SUPPORT PLANNING AND MANAGING FLOOD RISK

The following information in regard to the above has been provided:

- depth and velocity of floodwaters across roads;
- capacity of stormwater network (i.e. in what event will overland flow occur above a stormwater pipe);
- description of "hot spots";
- mapping of flood planning constraints categories; and
- mapping of the flood planning area.

A discussion of the AEP terminology and a glossary of other flood-related terms is provided in Section 14.

1. INTRODUCTION

1.1. Background

The Greendale Creek Flood Study covers the Greendale Creek catchment, including Curl Curl Lagoon, which is located in the northern coastal suburbs of Sydney. The study area includes the suburbs of Beacon Hill, Brookvale, Curl Curl, Freshwater and North Curl Curl. The catchment lies within the Local Government Area (LGA) of Northern Beaches Council.

This flood study provides information about existing flood risk in the catchment. Flood modelling tools were developed that can be used by Council for decision-making about land-use planning, and in future studies to assess the effectiveness of potential measures to reduce flood risk. The models were calibrated using observations from historical floods, and used to estimate the impacts of flooding for a range of standardised "design" flood probabilities. This modelling was completed in accordance with the guidelines in Australian Rainfall and Runoff (Reference 1).

Flooding in the catchment can occur when intense local rainfall generates runoff exceeding the capacity of drainage channels and creeks, or from flooding of Curl Curl Lagoon leading to inundation of low lying areas in the lower Greendale Creek catchment. Previous flood studies and floodplain risk management studies (References 2, 3 and 4) of Curl Curl Lagoon were undertaken in 2004 and 2005, respectively. However no detailed catchment study considering overland flooding of the Greendale Creek catchment upstream of the influence of lagoon flooding has previously been undertaken.

Northern Beaches Council is responsible for managing development in accordance with flood risk, as per the NSW Floodplain Development Manual (FDM, Reference 5). This study will provide Council with relevant flood information for strategic planning and development assessment.

1.2. Scope of Study

The Flood Study defines design flood behaviour for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% Annual Exceedance Probability (AEP) design storms and the Probable Maximum Flood (PMF) in the Greendale Creek catchment. This report documents the data, methodology and outputs from the flood modelling exercise, including the following specific tasks:

- the collection and collation of existing information relevant to the study which includes the data already held by Council as well as other information, such as rainfall data;
- the preparation of hydrologic and hydraulic models capable of defining the flood behaviour for the study area for a wide range of design flood probabilities;
- undertaking sensitivity analysis;
- assessing the impacts of projected future changes to rainfall intensity and sea level rise
- the interpretation and presentation of model results to describe and categorise flood behaviour and hazard for a range of design storm events for the existing catchment conditions;
- determining the Preliminary Flood Planning Area extent.

2. BACKGROUND

2.1. Study Area

The study area covers approximately 470 hectares (4.7 km^2), comprising the Greendale Creek catchment, including Curl Curl Lagoon. The Greendale Creek catchment is situated within the northern beaches of Sydney and includes parts of the suburbs of Beacon Hill, Brookvale, Curl Curl, Freshwater and North Curl Curl. The catchment (Figure 1) drains into Curl Curl Lagoon which is an intermittently closed and open lagoon (termed an ICOLL) of approximately 5.7 ha (0.057 km^2) (Reference 2).

The catchment generally flows from west to east, with the upper Greendale Creek catchment flowing in a south-easterly direction through residential and industrial areas. The trunk drainage system exits at Harbord Road into the modified semi-natural channel of Greendale Creek which discharges into Curl Curl Lagoon. Elevations in the upper part of the catchment to the north-west reach approximately 150 mAHD (mapping of the topography from LiDAR aerial survey is shown in Figure 2). The topography within the study area ranges from moderately steep terrain, in the upper catchment where grades of approximately 9% in the suburban areas are common to gently sloping, particularly in the industrial areas upstream of Harbord Road.

The land use within the catchment consists primarily of medium and high density urban residential development with a considerable amount of industrial development in the suburb of Brookvale, together with parks and sporting ovals, adjacent to Curl Curl Lagoon. The steep forested area in around “The Kilns” is a notable feature of the upper catchment. Brookvale Oval is situated adjacent to Pittwater Road and the earthen bund to the north and east of the field is a notable topographic feature, which affects overland flow behaviour in the local area. A large industrial area characterises the lower Greendale Creek catchment which is bounded by Harbord Road to the east, Pittwater Road to the north and west and Wattle Road to the south.

The catchment includes natural creek channels, kerbs and gutters, pits and pipes, and a network of trunk drainage elements including culverts and concrete-lined or otherwise modified open channels. These trunk drainage assets are primarily owned by Northern Beaches Council.

2.2. Curl Curl Lagoon

Curl Curl Lagoon is classified as an ICOLL, as the ocean entrance can be either open or closed. The Curl Curl Lagoon entrance condition has a significant influence on water levels up to the vicinity of Harbord Road (Reference 3), where there is a gross pollutant trap. The berm height at the entrance of Curl Curl Lagoon varies significantly over time.

Management of the Curl Curl Lagoon entrance is the responsibility of Northern Beaches Council, and involves mechanical opening of the entrance berm when the lagoon gauge reaches a specified level (2.2 mAHD at the time of writing), allowing breakout to occur and for the lagoon to discharge into the ocean. This trigger level for mechanical opening is based on consideration of inundation of roadways in the vicinity of low lying properties in Surf Road, with the minimum level of Surf Road at approximately 2.3 mAHD.

The lagoon often breaks out naturally before the lagoon reaches this height, as the lagoon volume is relatively small compared with the catchment size, and the runoff that is generated is often sufficient to cause a breakout at the entrance before the berm reaches the level requiring intervention. If the berm has built up higher than the trigger level and enough rainfall is forecast for possible flooding, Council lowers the berm to allow a natural breakout just below the trigger level. Unauthorised breakouts have also been known to occur.

2.3. Historical Flooding

Flooding in the Greendale Creek catchment can occur when intense local rainfall generates runoff exceeding the capacity of drainage channels and creeks, producing overbank flow or overland flooding. The lower catchment is also subject to mainstream flooding from Curl Curl Lagoon. Flooding in some areas may be exacerbated by the blockage of hydraulic structures and the presence of obstructions to overland flow paths such as buildings.

The April 1998 flood event was selected for model calibration in the 2005 Dee Why Lagoon and Curl Curl Lagoon Flood Study (Reference 2) due to the availability of anecdotal evidence describing flooding in the Greendale Creek catchment. This peak burst rainfall intensity for this event approximates a 10% AEP event (Reference 2).

A more recent notable flood event occurred in November 2018, which caused flooding in the catchment and a breakout of Curl Curl Lagoon.

Some properties located below the berm height of Curl Curl Lagoon, such as those located at the southern end of Surf Road, can be inundated due to elevated lagoon levels.

2.4. Previous Studies

Previous flood investigations have been completed for the Greendale Creek catchment. A brief summary of previous studies relevant to the current investigation are provided below.

2.4.1. Dee Why Lagoon and Curl Curl Lagoon Flood Studies

This study was undertaken by Lyall and Associates (Reference 2) in 2004 to determine flood behaviour for Dee Why Lagoon and Curl Curl Lagoon. The area under investigation included the floodplains of Greendale Creek and Curl Curl Lagoon catchment to Harbord Road and parts of the industrial areas to the intersection of Winbourne Road and Mitchell Road. Flood behaviour upstream of the Winbourne Road culvert was not included in the hydraulic model.

The study used a RORB hydrologic model to estimate runoff hydrographs and a 1D MIKE-11 hydraulic model to define flood behaviour, except for the “Brookvale Industrial Estate Area” between Winbourne Road and Harbord Road, where a series of HEC-RAS models were established to represent flooding along the road network. A limited calibration of the Mike-11 hydraulic model was undertaken using the available historical data for the April 1998 event. The

HEC-RAS modelling of the Brookvale industrial estate was not calibrated or verified – it was a relatively crude modelling approach that used a separate HEC-RAS model for each street with inflows determined from the hydrologic model.

Rainfall data from AR&R 1987 (Reference 6) was applied to produce design flood levels for the 20%, 10%, 2% and 1% AEP design events.

The study did not define the extent of inundation upstream of Harbord Road due to the limitations in the HEC-RAS modelling approach discussed above. It was noted that future flood studies should extend the hydraulic model westward of Harbord Road to provide a more detailed understanding of flood levels in this area.

2.4.2. Dee Why Lagoon and Curl Curl Lagoon Floodplain Risk Management Study & Plan

This 2005 study by Lyall and Associates (References 3 and 4) was prepared to assist the former Warringah Council in the development of a Floodplain Risk Management Study and Plan using the results drawn from the Flood Study (Reference 2). This was undertaken by identifying the nature and extent of the flood hazard and the flood damage costs for commercial/industrial and residential developments.

3. AVAILABLE DATA

3.1. Overview

The first stage in the investigation of flooding matters is to establish the nature, size and frequency of the problem. On larger urban river systems such as the Hawkesbury River there are generally stream height and historical records dating back a considerable period, in some cases over one hundred years. However, in smaller urban catchments stream gauges and/or official historical records are generally not available, and there is more uncertainty about the frequency and magnitude of flood problems. Additionally, overland flooding in urban areas is highly dependent on localised changes to development, intensification of development (i.e. increased building sizes and more paved surfaces), and localised drainage features such as kerbs and guttering in roadways. These features are subject to relatively frequent modification and renewal, making it difficult to compare flood behaviour over time.

There are several pluviometers surrounding the catchment. There is one pluviometer situated within the catchment, at Curl Curl Lagoon, which was installed in 2014 and captured data for the November 2018 event. Two nearby pluviometers were also available to the south-west of the catchment for this event.

An understanding of historical flooding was obtained from an examination of Council's records, previous flood assessment reports, rainfall records and local knowledge obtained through community consultation (see Section 4).

Airborne Light Detection and Ranging (LiDAR) data in urbanised areas and detailed bathymetry survey of lower Greendale Creek and Curl Curl Lagoon (collected as part of previous studies) was available for modelling. A relatively high quality GIS database of surveyed pits and pipes was also available. This data required slight corrections to some invert levels and dimensions before incorporation into the hydraulic model. As part of this study, analysis of the available data along with site visits were undertaken to address the limitations of the data in key areas.

It should be recognised that while the information about the drainage system for this study is not perfect, this is often not a critical issue, since the majority of runoff cannot usually be contained within the formal drainage network for the types of flood events being considered. Sub-surface drainage networks in metropolitan Sydney are typically only designed to cater for the 20% to 10% AEP flow. Therefore, caution must be exercised when applying the broad catchment modelling results at individual properties, particularly for smaller floods or in areas where the pit/pipe drainage network plays a significant role in the flood behaviour.

3.2. Data Sources

Data utilised in the study has been collated from a variety of sources. Table 1 provides a summary of the type of data sourced, the supplier, and its application for the study.

Table 1: Data Sources

Type of Data	Source	Application
Ground levels from LiDAR data (2013)	Digital Elevation Model - DEM (LPI)	Hydrologic and hydraulic models
Curl Curl Lagoon Bathymetric Data	MHL	Hydraulic model
Pits, Pipes and Hydraulic Structures	Northern Beaches Council	Hydraulic model
GIS Information (Cadastre)	Northern Beaches Council	Hydraulic model
Historic Flood Level Data	MHL, Northern Beaches Council, Local Residents	Hydraulic model
Rainfall Gauge (Pluviometer)	MHL	Hydrologic model
Rainfall Gauge (Daily)	BoM	Hydrologic model
ARR Design Rainfalls, Temporal Patterns and Loss Rates	BoM	Hydrologic model

3.3. Topographic Data

Airborne Light Detection and Ranging (LiDAR) survey of the catchment and its immediate surroundings was obtained from Land and Property Information (LPI), which is a division of the Department of Finance, Services and Innovation (NSW Government). The LiDAR survey was collected in 2013. The typical accuracy of this dataset is:

- +/- 0.15 m (for 70% of points) in the vertical direction on clear, hard ground; and
- +/- 0.75 m in the horizontal direction.

The accuracy of the LiDAR data can be influenced by the presence of open water or vegetation (tree or shrub canopy) at the time of the survey. The 1 m by 1 m Digital Elevation Model (DEM) generated from the LiDAR, which formed the basis of the two-dimensional hydraulic modelling for the study, is shown in Figure 2.

Council provided additional information for the following two developments which took place after the LIDAR capture date and the model topography was modified accordingly:

- St Augustine's School (ground levels and building footprints); and
- 40 Chard Road (building footprints only).

Furthermore, modifications to the DEM were made to include features observed during the site inspection at "The Kilns" development (see Section 6.6.9).

3.4. Hydraulic Structures

Structures including bridges and culverts can have a significant impact on flood behaviour. Therefore, appropriate representation of these structures is essential for the accuracy of the hydraulic model. Data for hydraulic structures was primarily obtained from:

- Northern Beaches Council (Works-As-Executed drawings); and

- Measurements obtained during site visits.

During the inspection of the study area WMAwater obtained photographs and additional measurements of key hydraulic structures in the catchment. The locations of these structures are shown on Figure 12.

3.5. Bathymetric Survey

Within Curl Curl Lagoon and lower Greendale Creek, the bathymetry is not accurately captured by LiDAR data, since LiDAR is unable to penetrate the water surface. A bathymetric DEM within the lagoon was provided by Northern Beaches Council (obtained from MHL). The DEM was constructed from detailed survey of Curl Curl Lagoon, sampled at a regular grid cell size of 5 m.

3.6. Pit and Pipe Data

A database of surveyed stormwater pits and pipes within the catchment was provided by Northern Beaches Council (see Figure 3). The pits and pipes data generally contained inverts and dimensions for most pits and pipes. Where data was not available pit inlets and pipe sizes were determined from the following principles:

- Pipes were assumed to have a depth of cover of 0.5 m to the top of the pipe below the recorded ground level at pits and junctions;
- Pit inlets were modelled as having the inlet level at the LIDAR ground level;
- Where inlet pit dimensions were not provided a lintel opening width of 1.2 m was assumed;
- Where unavailable pipe sizes were estimated based on the sizing of connected upstream and downstream pipes.

Following this initial estimation, further corrections to pit inverts were undertaken to correct pipes with negative slope or pipes that were located above ground in the model.

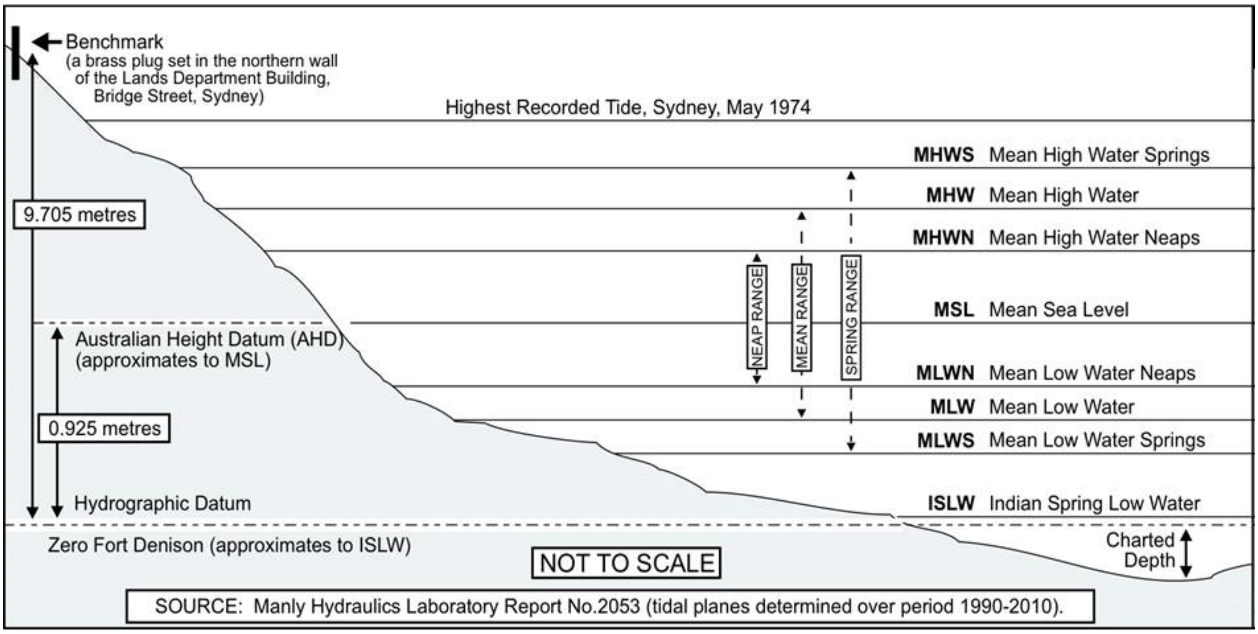
3.7. NSW Tidal Planes Analysis

Manly Hydraulics Laboratory prepared the *NSW Tidal Planes Analysis: 1990-2010 Harmonic Analysis* report on behalf of OEH (Reference 7). It was released in October 2012 and was based on data from 188 tidal monitoring stations from 1st July 1990 to the 30th June 2010. Data from the relevant stations are shown in Table 2 with a tidal plane diagram shown as Diagram 1. Curl Curl Lagoon may be subject to tidal influence in large flood events when lagoon breakout occurs however due to the elevation of the entrance berm, peak flood levels will not generally be affected by the tidal conditions, unless there is a major storm surge and wave action accompanying the rainfall.

Table 2: Tidal Planes Analysis Results (MHL, 2012)

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

Diagram 1: Tidal Planes Diagram



3.8. Stream Gauge Data

Historical stream gauge data is available from the Greendale Creek Brookvale gauge (213499) at Harbord Road and Curl Curl gauge (213426) located on Griffin Road Bridge. Recordings at Curl Curl gauge (213426) were available from August 1991. A subset of these recordings is shown in Figure 13. This gauge provides water level information but not flow information, as a rating curve cannot be developed due to the influence of the berm.

Recordings at Greendale Creek Brookvale (213499) were available from MHL for April 2013 to July 2018 as shown in Figure 14. This gauge provides discharge information but the data period ends prior to the November 2018 calibration event. Hence a water level hydrograph was available for the November 2018 event for use in hydraulic model calibration (at gauge 213426) however no flow measurement data was available.

The water level gauge in Curl Curl Lagoon (213426) provides sufficient information to characterise the regularity of breakouts, and the nature of the lagoon water level response after a breakout. This includes the lagoon response to the November 2018 event, during which a breakout occurred. However there is insufficient data to determine design flows for the catchment.

3.9. Historical Rainfall Data

3.9.1. 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, due to the nature of the ‘tipping bucket’ used for rainfall collection, can fail during the most intense portion of a storm. While this data has

fewer limitations than daily read data and provide a much greater insight into the intensity (depth vs. time) of rainfall events, they are not without limitations.

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.

3.9.2. Rainfall Stations

There are a number of rainfall stations located across the Sydney metropolitan area, including daily read and pluviometer gauges. The continuous pluviometer stations record rainfall in sub-daily increments (with output typically reported approximately every 5 minutes). These records were used to create detailed rainfall hyetographs, which form the model input for historical events against which the model was calibrated. The nearby continuous pluviometers used in the calibration process are shown in Table 3 with locations shown on Figure 4 (daily-read gauge locations are also shown). Only one pluviometer gauge at Curl Curl is located within the catchment. These gauges commenced between 1999 and 2014 and were all operational during the November 2018 rainfall event.

Table 3: Pluviometer Rainfall Stations

Station Number	Station Name	Authority
213426	Curl Curl	MHL
566152	Allambie Heights	MHL
566151	North Manly	MHL

3.9.3. Analysis of November 2018 Rainfall Event

The daily rainfall depths recorded at nearby gauges are shown in Table 4. The November 2018 storm event (the sole event used for model validation in this study) comprised an intense rainfall burst between approximately 4:30 am and 7 am on 28/11/2018, which was then followed by less intense sporadic rain over the next 24 hours. This storm was captured by the three nearby pluviometers. One of these pluviometers, the Curl Curl gauge at Griffin Road Bridge (213426) is located within the catchment. The total depths recorded at nearby rainfall gauges over the 2 day period ranged from 53.6 mm to 168 mm.

Table 4: Daily Rainfall Depths (mm) for the November 2018 Event

Station Number	Station Name	Type	28-Nov 2018 Rainfall (mm)	29-Nov 2018 Rainfall (mm)
66126	Collaroy (Long Reef Golf Club)	Daily	27.2	26.4
66188	Belrose (Evelyn Place)	Daily	42.6	27.6
66080	Castle Cove (Rosebridge Ave)	Daily	75	35
66011	Chatswood Bowling Club	Daily	135	33
66059	Terry Hills AWS	Daily	36.4	44.2
66141	Mona Vale Golf Club NSW	Daily	51.8	39
66209	Dover Heights (Portland St)	Daily	59.8	27.8
66206	St Ives (Richmond Avenue)	Daily	51.6	33.8
66006	Sydney Botanic Gardens NSW	Daily	106.6	31.4
66214	Sydney (Observatory Hill Comparison)	Daily	104.2	29.8
66062	Sydney (Observatory Hill) NSW	Daily	105.6	30.2
566151	North Manly	Pluviometer	61.5	21
566152	Allambie Heights	Pluviometer	62.5	16
213426	Curl Curl	Pluviometer	51	26

Rainfall isohyets which describe the spatial distribution of rainfall for this event are shown on Figure 5. Cumulative rainfall data which describes the temporal pattern of rainfall recorded at the nearby pluviometers is shown on Figure 6.

The total rainfall depths were generally higher further inland, for example at Chatswood and Sydney Observatory Hill, and lower towards the coast. The areas around the Sydney CBD and Manly received the most intense short duration rainfalls (i.e. over the period of 30 minutes to 60 minutes), with intensities approximating a 20% AEP.

A comparison of the peak recorded rainfall bursts with design rainfall curves taken at the Curl Curl (213426) gauge is shown on Figure 7. This indicates that the rainfall approximated a 20% AEP event for the 15 minute duration, and a 50% AEP event for the 30 minute to 6 hour duration. The North Manly and Allambie Heights gauges indicated slightly rarer AEPs.

4. COMMUNITY CONSULTATION

4.1. Overview

A newsletter/questionnaire (Appendix H) was distributed to residents in the study area to inform them about the study, and to obtain information about historical flooding. 113 responses relating to flooding in the Greendale Creek catchment were received, with 34 respondents indicating they had experienced flooding of their home or business due to flood water or stormwater. One respondent indicated that they had experienced above floor flooding of the main building on their property with the remainder of respondents indicating that flooding had affected their garage, yard or other parts of their property.

Respondents identified flooding in streets, parks or other public areas as primarily occurring in the suburbs of Curl Curl, Brookvale and North Curl Curl. Half of the flood affected respondents indicated that they noticed blocked drains or culverts during the flood. 14 respondents provided additional comments which raised obstruction of drains and waterways as a concern.

The results of the community consultation process, indicating the locations of flood affected respondents are shown in Figure 8. The results from the community consultation questionnaire are summarised in Figure 9.

4.2. Community Responses

A selection of photographs provided by the community is shown below (Photo 1 to Photo 4)

Photo 1: The Kilns (March 2013)

Photo 2: Flooding near Unit 6, The Kilns (2014)

Photo 3: Stirgess Reserve (March 2011)

Photo 4: Flooding at The Kilns (September 2013)

Community consultation responses indicate that flows exceeding the capacity of the channels around The Kilns occurred several times between 2013 and 2014 due to a landslide which caused blockage of a section of the channel. The rocks from this landslide were removed from the channel in early 2015, which the respondent suggests has lowered the flood risk in this area.

Photo 3 indicates that ponding of water in Stirgess Reserve is likely to occur relatively frequently.

The following issues were raised by the respondents:

- Several respondents expressed concerns that development in the catchment is exacerbating flooding;
- Some respondents were concerned about debris and blockage of drains due to vegetation and rubbish;
- Most reported flood observations related to yards, streets, parks or Curl Curl Lagoon, rather than overfloor flooding;
- One respondent suggested the berm on Frank Gray Oval was particularly effective in mitigating flooding;
- Few residents raised concerns about above floor level flooding on their property. This observation is reflected in previous studies (Reference 2 to Reference 4) which found that most affected properties in the Greendale Creek catchment are located in industrial areas such as the Brookvale industrial area.

4.3. Public Exhibition

A draft version of this report was placed on public exhibition from 29 March 2023 to 7 May 2023 to invite comment from the community. A copy of the report was available for download from Council's 'Your Say' website. The website also contained a section addressing frequently asked questions. Instructions for making formal written submissions were also provided to those wishing to comment on the study.

Residents affected by the draft Flood Planning Area (see Section 9.5.4) and draft PMF extent were notified via mail. Residents were invited to book a one-on-one 15 minute appointment with Council and WMAwater staff to discuss the study and its implications. Four sessions were held:

- Wednesday 5 April 2023, 4 pm – 7 pm at the Curl Curl Sports Centre
- Thursday 13 April 2023, 11.30 am – 3 pm at the Curl Curl Sports Centre
- Wednesday 26 April 2023, 9.30 am – 4 pm at the Curl Curl Sports Centre
- Saturday 29 April 2023, 9.30 am – 1 pm at the Brookvale Community Centre South Hall

There were a total of 74 on-on-one appointments were attended by residents of the study area. Apart from several very property-specific questions raised, there were recurring questions and concerns about the study. These included:

- Concern about how the identification of a property as flood affected, or about how the study in general may affect house insurance premiums.
- Concern about how the identification of a property as flood affected, or about how the study in general may affect house prices.
- Concern about how the identification of a property as flood affected affects current or future redevelopment plans.
- Queries about how a property could be flood affected when it is much higher than Curl Curl Lagoon. The overland flow approach of the study was explained in this case. Some queries also related to the fact that flooding was due to inadequate drainage. The design of the stormwater network for frequent events was explained and that in large events overland flow is to be expected. The follow up question was typically what is Council going to do about the flooding issues. It was explained that the next stage of the NSW Flood Program was to conduct a Floodplain Risk Management Study to investigate flood risk mitigation options.
- Requests to be removed from the Flood Planning Area. In some cases, it was identified that only a minor portion of the lot was affected. The selection criteria for flood affectation was reviewed in detail and it was considered reasonable that properties with minor affectation of the Flood Planning Area be removed. Additional filtering criteria at the lot level was applied to ensure a consistent approach for all properties across the catchment. In other cases, the identification of the lot as flood affected remained.

Several community members also raised the recent March 2022 storm event, which was a significant event that affected the Northern Beaches. This event took place after the first draft of the Greendale Creek Flood Study Report was produced, and as such it has not been considered in this study. In general, comments were made regarding:

- The fact that a property was not affected by the March 2022 storm event, however, is still

identified as flood-affected. In this case it was explained that the storm event was most likely smaller than a 1% AEP event which is used to determine flood affectation of properties.
- Confirmation of the flood modelling results, with observations from the March 2022 storm event aligning with the flood modelling results that were mapped. Where people noted flood inundation, this was mapped as such in the design flood events.

There were also a small number of residents who brought forward information (such as photographs or topographic survey) to indicate features that may affect overland flows. Where appropriate, the model was updated with these features following the public exhibition period. This resulted in minor and highly localised changes to overland flow behaviour.

A total of 28 submissions were made during the Public Exhibition period. The majority of these were following up on a one-to-one appointment by formalising their concerns or as a way of providing additional information. A further 3 submissions were received via emails and telephone calls following the Public Exhibition period. The themes of the submissions closely followed the one-to-one meetings, as outlined above. Written responses were provided to each respondent.

5. HYDROLOGIC MODEL

5.1. Introduction

A hydrologic model is a tool for estimating the amount of runoff that flows from a catchment for a given amount of rainfall, and the timing of this runoff flow. Stream gauges (which measure water level in a stream) are a way of directly measuring this information, but they are expensive to setup and maintain. They also require a long record (several decades) to be of most use for flood estimation. The majority of small creeks in NSW are not gauged, and there are no long-term stream gauges in the Greendale Creek catchment. In such cases, using a computer-based hydrologic model is the best practice method for determining how much flow may occur 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 are available as described in ARR2019 (Reference 1). 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.

A WBNM hydrologic runoff-routing model was used to determine flows for the entire Greendale Creek catchment to the outlet at the ocean. The WBNM model has a relatively simple but well supported method, where the routing behaviour of the catchment is primarily assumed to be correlated with the catchment area. If flow data is available at a stream gauge, then the WBNM model can be calibrated to this data through adjustment of various model parameters including the stream lag factor, storage lag factor, and/or rainfall losses. When flow data is not available (as is the case here), typical practice is to jointly verify the hydrologic and hydraulic models by comparing the model results to observed water level information.

The hydrological model for the entire Greendale Creek catchment (Figure 10), including the coastal overland flow areas draining directly to the ocean to the north and south, was created and used to calculate the flows for inclusion in the TUFLOW hydraulic model. The hydraulic model is discussed in Section 6.

5.2. Sub-catchment delineation

The total catchment area covered by the WBNM model of the entire Greendale Creek catchment is approximately 4.7 km^2 consisting of 755 sub-catchments (Figure 10) with an average sub-catchment size of 0.6 hectares. This relatively fine-resolution sub-catchment delineation ensures that where significant overland flow paths exist in the catchment, they are accounted for and incorporated into the TUFLOW hydraulic model.

5.3. Impervious Surface Area

Runoff from connected impervious surfaces (such as roads, gutters, roofs or concrete surfaces) occurs significantly faster than from pervious surfaces. This can result in a faster concentration of flow within the downstream area of the catchment as well as increased peak flow in some situations. This is accounted for in the model through an estimate of the proportion of both impervious and pervious surfaces.

The pervious and impervious area of each sub-catchment was estimated by assessing the proportion of the sub-catchment area covered by different surface types (from aerial photography) and then applying the impervious percentage of each surface type as indicated in Table 5.

Table 5: ARR2019 Effective Impervious Area Estimation

Landuse Type	Pervious Area (%)	Indirectly Connected Impervious Area (%)	Effective Impervious Area (%)
Natural Vegetated Area	100	0	0
Grass/ Field	80	0	20
Medium Density Residential	0	30	70
Industrial/ Commercial/ High Density Residential	0	10	90
Lagoon	10	0	90

5.4. Rainfall Losses and WBNM Lag Parameters

Methods for modelling the proportion of rainfall that is "lost" to infiltration are outlined in ARR2019 (Reference 1). The methods are of varying degrees of complexity, with the more complex options only suitable if sufficient data are available. The method most typically used for design flood estimation is to apply an initial and continuing loss to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues. The initial/continuing loss method was adopted for this study.

Rainfall losses from a paved or impervious area are considered to consist of only an initial loss (an amount sufficient to wet the pavement and fill minor surface depressions). Losses from grassed and vegetated areas are comprised of an initial loss and a continuing loss.

WBNM requires a catchment lag parameter and a stream lag factor to be selected which describes the average travel time for runoff from the catchment surface. The lag parameter is applied to pervious surfaces and adjusted to apply to impervious surfaces by multiplication by an impervious lag factor. The WBNM parameters selected are summarised in Table 6.

Table 6: Adopted WBNM Parameters for Calibration

WBNM Parameter	Value
Initial Loss (Pervious surface)	19.6 mm – 28 mm
Continuing Loss (Pervious surface)	1.5 mm/hr – 2.5 mm/hr
Lag Parameter (C)	1.29
Stream Lag Factor	1.0
Impervious Lag Factor	0.1

The parameter values applied are generally consistent with the recommended values in the WBNM manual and are the recommended values for ungauged urban catchments (Reference 8). Initial and continuing loss values for rural pervious areas were obtained from the ARR2019 Datahub and modified to account for the various urban land-use types.

6. HYDRAULIC MODEL

6.1. Introduction

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.

2D hydraulic modelling is currently the best practice standard for urban flood modelling (Reference 9). It requires high resolution information about the topography, which is available for this study from the LiDAR aerial survey. Various 2D software packages are available (SOBEK, TUFLOW, RMA-2). The TUFLOW package was adopted as it meets requirements for best practice, and is currently the most widely used model of this type in Australia for riverine flood modelling.

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 2018-03-AE-iSP (using the finite volume HPC solver), and further details regarding TUFLOW software can be found in the User Manual (Reference 10).

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

6.2. Model Extent and Grid Resolution

The study implemented a TUFLOW model with a cell size of 2 m by 2 m. This resolution provides an appropriate balance between providing sufficient detail for roads and overland flow paths and workable computational run-times.

The TUFLOW hydraulic model encompasses the entire Greendale Creek catchment, including Curl Curl Lagoon, and the overland flow areas to the north and south draining directly to the ocean.

Typically, developed areas require a grid resolution of no more than 2 m to capture the various overland flow mechanisms characteristic of a built-up environment. In 2017, a new TUFLOW version was released with High-Performance Computing (HPC) Graphical Processor Unit (GPU) model support. The new HPC GPU models are significantly faster than the traditional Central Processing Unit (CPU). As such, the HPC Engine with GPU was used for this study, although the HPC models can be run over a longer timeframe using CPU. This enabled a grid size of 2 m to be adopted for the entire model area while producing practical run times.

6.3. Model Topography

The model terrain grid was established from the data discussed in Section 3.2 to Section 3.5. The LiDAR data was generally found to provide an appropriately detailed representation of the catchment topography in most areas however the LiDAR survey is unable to penetrate the water surface. Bathymetric survey of Greendale Creek and Curl Curl Lagoon was used to define the waterways downstream of the Harbord Road gross pollutant trap (GPT). The entrance to Curl Curl Lagoon was defined as a Z shape with variable geometry as discussed in Section 6.6.12 and Section 8.8. Bridges, weirs and the Harbord Road GPT were modelled as 2D elements while culverts were modelled as 1D structures linked to the 2D domain as discussed in Section 6.6.

6.4. Boundary Conditions

6.4.1. Inflow Boundaries

Local runoff hydrographs were extracted from the WBNM model (see Section 5) and applied to the receiving area of the sub-catchments within the 2D domain of the hydraulic model. These inflow locations correspond with gutters, stormwater inlet pits, drainage reserves or open watercourses features which have typically been constructed to receive intra-lot drainage and sheet runoff flows from upstream catchment areas.

6.4.2. Downstream Boundaries

For the November 2018 calibration event a static tailwater level of 0.78 mAHD was adopted as the downstream ocean boundary. Since the berm was known to be substantially elevated above ocean levels (initial water levels greater than 2 m AHD for this event), the adoption of a low static tailwater was considered appropriate as the tidal conditions would not have affected peak flood levels in Curl Curl Lagoon or Greendale Creek.

The sensitivity of peak flood levels to tailwater conditions is discussed in Section 10.

6.5. Surface Roughness

Roughness, represented by the Mannings ‘n’ coefficient, is a key parameter in hydraulic modelling. As part of the calibration process roughness values are adjusted within ranges defined in the literature so that the model better matches observed peak flood levels at a variety of locations. Chow (Reference 11) provides some information with regards to the setting of the of the roughness values for hydraulic calculations. Mannings ‘n’ values are also discussed in

Project 15 of ARR2019 – *Two Dimensional Modelling in Urban and Rural Floodplains* (Reference 12).

The Mannings ‘n’ values adopted for the study area are shown in Table 7. These values have been adopted based on site inspection, past experience in similar floodplain environments, consideration of the above references, and the model calibration process. The spatial variation in Mannings ‘n’ within the model boundary is shown in Figure 11.

Table 7: Mannings 'n' values adopted in TUFLOW

Surface	Mannings ‘n’
Grass	0.04
Light Vegetation	0.06
Medium Vegetation	0.07
Thicker Vegetation	0.09
Creek	0.05
Paved Area	0.02
Lagoon	0.03
Urban Properties	0.065
Industrial	0.20

6.6. Hydraulic Structures

6.6.1. Buildings

Buildings and other significant features likely to obstruct flow were incorporated into the model based on building footprints defined from aerial photography. These types of features were modelled as impermeable obstructions to flow and thus were assumed to have no flood storage capacity. Building delineation was validated in key overland flow areas by site inspection and using aerial and street level photographs.

6.6.2. Fencing and Obstructions

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.

The exception to the above was a concrete wall in the Kilns development, which was clearly part of an overland flow management design (see Photo 12).

6.6.3. Bridges and Culverts

Key hydraulic structures were included in the hydraulic model, at the locations indicated in Figure 12. Griffin Road Bridge, pedestrian crossing and Harbord Road GPT were modelled in the 2D domain to maintain continuity in the model and because the 2 m resolution was generally sufficient to resolve the waterway area accurately. Griffin Road Bridge (Photo 5) is a large relatively clear spanning structure with a large waterway area. Reference 2 stated that the structure does not cause significant afflux during large flood events.

Photo 5: Griffin Road bridge (looking upstream)

Culverts and stormwater pipes that have geometry smaller than the 2D grid were modelled as 1D computational elements.

The modelling parameter values for the hydraulic structures were based on the geometrical properties of the structures obtained from survey data and site inspections, using the guidance provided in the TUFLOW manual (Reference 10).

6.6.4. Surface and Sub-Surface Drainage 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 pit and pipe GIS layers supplied by Northern Beaches Council. Figure 3 shows the location of pits and pipes included as 1D elements in the hydraulic model.

Only pipes with a minimum dimension of 300 mm or greater were included in the model. Smaller pipes than this are unlikely to have a significant influence on flood behaviour during major overland flow events.

6.6.5. Inlet Pits

For the modelling of inlet pits the "R" pit channel type was utilised, which requires a width and height dimension for the inlet in the vertical plane. The width dimension represents the effective inlet length exposed to the flow, and the vertical dimension reflects the depth of flow where the inlet becomes submerged, and the flow regime transitions from the weir equation to the orifice equation. For lintel inlets, the width was based on the length of the opening which was generally available in surveyed pits database provided by Northern Beaches Council. In cases where the lintel length of inlet pits was erroneous or unavailable, a length of opening of 1.2 m was assumed. Details of the 1D solution scheme for the pit and pipe network are provided in the TUFLOW user manual (Reference 10).

6.6.6. Road Kerbs and Gutters

LiDAR typically does not have sufficient resolution to adequately define the kerb and gutter system within roadways. The density of the aerial survey points is in the order of one per square metre, and the kerb/gutter feature is of a smaller scale than this, so the LiDAR does not pick up a continuous line of low points defining the drainage line along the edge of the kerb. Reference 12 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."*

The road gutter network plays a key role for overland flow in the urbanised parts of the study area. In order to model the system effectively, the gutters were stamped into the mesh using the method described above. The method used was to digitise breaklines along the gutter lines, and reduce the ground levels along those model cells by 0.1 m, creating a continuous flow path in the model.

6.6.7. Harbord Road GPT

A GPT is located at the outlet of the two large culverts under Harbord Road. The geometry of this was incorporated into the model based on the cross-section data in the plans provided by Northern Beaches Council. The trash rack was modelled as an obstruction to flow with a high percentage blockage (100%) and a large form loss to account for the substantial amounts of debris which are likely to become lodged in this structure. Photographs of the GPT were obtained during the site visit and these are shown in Photo 6 and Photo 7.

Photo 6: GPT (looking upstream at Harbord Road culverts)

Photo 7: GPT trash rack (looking downstream)

6.6.8. Footbridge and Rock Weir

Two pedestrian bridges cross Greendale Creek between Harbord Road and Griffin Road. The bridges are clear spanning with a large waterway area. A rock weir was constructed downstream of the eastern pedestrian footbridge circa 2000 as part of the Greendale Creek Rehabilitation Project for water quality control purposes.

Dimensions of the eastern footbridge over Greendale Creek were obtained during the site visit. Photographs of the rock weir and western footbridge were obtained during the site visit and these are shown in Photo 8 and Photo 9, respectively.

Photo 8: Rock Weir (looking downstream from western pedestrian footbridge)

Photo 9: Western pedestrian footbridge (looking upstream)

6.6.9. The Kilns

The steep channels downstream of Governor Philip Lookout descend to 'The Kilns' development. A semi-natural channel to the west of the site directs flows from north to south, before meandering eastwards. Flows from the channel then enter into a culvert passing under several low lying properties on Consul Road. A concrete lined channel to the east of the site directs flow to a large grated inlet pit, with the piped stormwater system joining the semi-natural channel to the south-west of the site. The concrete wall for the eastern channel was included in the model as a raised 2D element which presents an impermeable barrier to flow.

Photographs of the hydraulic structures of interest around The Kilns were obtained during the site visit and these are shown in Photo 10, Photo 11, Photo 12 and Photo 13.

Photo 10: Typical semi-natural channel section at the Kilns (looking downstream)

Photo 11: The Kilns channel through property (looking downstream)

Photo 12: Western stormwater channel at The Kilns (looking downstream)

Photo 13: Inlet grate (The Kilns, western channel)

6.6.10. Brookvale Oval

The pit/pipe database provided by Northern Beaches Council contains stormwater drainage infrastructure information within Brookvale Oval. Three 375 mm pipes which connect to the Council's stormwater network service Brookvale Oval and these were included in the hydraulic model.

Photographs of Brookvale Oval were obtained during the site visit and these are shown in Photo 14 and Photo 15.

Photo 14: Brookvale Oval (eastern bund looking north)

Photo 15: Brookvale Oval (eastern bund looking south)

6.6.11. St Augustine's School

Photo 16: St Augustine's College flow path

The recent re-development of St Augustine's School at Brookvale involved modifications to the building footprints and ground levels around the school. The building footprints were incorporated in the model based on the most recent available aerial photography. Updated ground levels were incorporated based on plans provided by Northern Beaches Council. During the site visit it was noted that a planter wall had been constructed in the drainage easement passing between the new primary school and new senior science buildings which is likely to present a significant obstruction to overland flows through the site. This was modelled as a flow constriction with a large blockage factor (70%) applied.

The flow path through St Augustine's School was inspected during the site visit and photographs are shown in Photo 16.

6.6.12. Curl Curl Lagoon Entrance

Lagoon breakout is a complex process which involves constant changes to the geometry of the breakout channel during the lagoon opening. Closure of the channel rapidly occurs as a result of sand movement into the breakout channel via coastal processes including tide and wind action. A detailed description of these lagoon breakout and closure mechanisms is presented in Appendix A from Reference 2.

The entrance to Curl Curl Lagoon was inspected during the site visit and a photograph is shown in Photo 17.

Photo 17: Curl Curl Lagoon entrance

The changing geometry of the sand berm during the calibration event (November 2018) on Curl Curl beach was modelled using a variable Z shape with parameters as detailed in Section 8.8.

7. MODEL CALIBRATION

7.1. Approach

Typically, in urban catchments with short gauge records 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. For example, there is only a single water level gauge in Curl Curl Lagoon and a single flow gauge at Harbord Road, Brookvale. Both of these gauges have a short record and the Harbord Road gauge failed to capture data for the November 2018 calibration event.
- 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; and
- Changes to the catchment over time due to urban development may result in significant changes to land uses and drainage structures.

These limitations are typical of the majority of urban catchments and the calibration exercise undertaken here constitutes recommended practice as outlined in Reference 12.

7.2. Hydraulic Model Calibration

The November 2018 event was a significant flood event in the Greendale Creek catchment which produced overland flooding and caused breakout of the entrance to Curl Curl Lagoon. A recorded water level hydrograph was available from Curl Curl gauge at Griffin Road Bridge (213426). However no additional data were available for this event upstream of the lagoon influence and no quantitative peak flood level marks were able to be obtained from the community consultation responses for this event. Flooding observations collected from the community consultation process were therefore used to validate modelled flow behaviour to ensure that overland flow paths and areas of ponded water were captured in the modelled flood event.

At the time the model calibration was undertaken, the stream gauge data for this event was not yet quality controlled by the gauge operator (MHL), indicating that it represents raw data from the instrument with only preliminary quality checks performed. WMAwater assessed that the data was of sufficient reliability for the purposes of the calibration exercise.

Mannings ‘n’ roughness values in the TUFLOW hydraulic model were set based on past experience and recommended values from the literature. The sensitivity of peak flood levels to Mannings ‘n’ roughness values is discussed in Section 10.4.

As noted in Reference 2 information on the entrance conditions of Curl Curl Lagoon is limited. The berm height varies substantially over time and hence lagoon flood levels for a given event will be highly dependent on entrance conditions. The modelled entrance characteristics were based on the adopted geometry in Reference 2 and the recorded lagoon water level data and the adopted model parameters are shown in Table 8. The adopted design berm height in Reference 2 was 2.2 mAHD and hence the initial berm elevation at the lagoon entrance is considered reasonable.

Table 8: Adopted Parameters of Curl Curl Lagoon Entrance

Parameter	Value
Width	70 m
Final Elevation	0.35 m AHD
Scour Initiation Level	2.5 m AHD
Initial Berm Elevation	~2.3 m AHD (based on LiDAR/ Lagoon survey data)
Period	0.1 hr

Lagoon water level data for the November 2018 event indicates that at the onset of the storm burst the initial lagoon water level was approximately 2.0 mAHD. Several preliminary model runs were completed to determine the appropriate geometry, scour initiation level and scour period to achieve a reasonable match to both the shape of the modelled rising limb and peak flood levels.

Joint calibration of the hydrologic and hydraulic models was undertaken by comparing the modelled flood levels with the stage hydrograph recorded on the downstream side of Griffin Road Bridge. A comparison of the modelled and recorded peak flood levels at Curl Curl gauge is shown in Table 9.

Table 9: Comparison of Modelled and Recorded Flood Levels

ID	Location	Recorded Level (mAHD)	Modelled Level (mAHD)	Difference (Modelled minus Recorded) (m)
213426	Curl Curl Lagoon at Griffin Road bridge	2.54	2.51	-0.03

The model was found to produce a reasonable match to the observed historical peak flood levels within approximately ±0.05 m of the recorded level. This error is considered reasonable due to the uncertainty in observed rainfall and model parameters, and measurement error of the gauge. The model is unable to perfectly match the recession limb due to the complex and event-specific opening characteristics of the lagoon entrance. This portion of the event is not important for the peak flood behaviour which is the focus of this study.

Mapping of peak flood levels and depths for the November 2018 calibration event are shown in Figure C1. Figure C2 shows a reasonable match for both the shape of the rising limb and the recorded peak flood height.

Sensitivity testing was undertaken to determine the effect of the assumptions made to the modelled entrance conditions.

The selection of appropriate entrance conditions for design flood estimation should be based on the joint probability of catchment rainfall, lagoon entrance conditions (particularly berm height) and initial lagoon water levels. The parameters adopted for design modelling are discussed in Section 8.

7.3. Validation

Descriptions of flood affectation provided by community consultation respondents and Council's customer complaints database were of some utility in validating key overland flow paths and the ponding of floodwaters at sag points. Residents did not specifically identify flooding as occurring during the November 2018 event however most respondents were able to identify where the flooding had occurred on the property around that time, or during previous events. Several respondents indicated that frequent flooding occurs in a specific location. The locations identified by this process were taken to represent areas where historic flooding may have occurred.

The results shown in Figure C3 present a comparison of the data collected from Northern Beaches Council's customer complaints database, flooding investigations and community consultation responses against modelled flood behaviour for the November 2018 event, recognising that not all the observed flooding corresponded to this event. A comparison of each description of flooding against the modelled flood behaviour for the November 2018 event is shown in Table 10.

Table 10: Validation of Modelling from Community Responses and Customer Complaints

ID	Description of Flooding	Calibration Match
W_002	Back yard flood affected	Reasonable
W_008	Garage or shed flood affected	Good
W_020	Front yard and back yard flood affected, above 100 yr flood zone	Good
W_024	Back yard and pool area flood affected	Good
W_028	Flood affected	Reasonable
W_044	Main building - below floor level flood affected	Poor(1)
W_054	Main building above floor level flood affected	Poor(2)
O_109	Front yard flood affected	Reasonable
W_009	Back yard flood affected	Good
W_011	Back yard flood affected	Good
W_016	Flood affected	Reasonable
W_033	Back yard and garage flood affected	Poor(3)
W_043	Back yard flood affected	Poor(4)
W_053	Main building below floor level and garage flood affected	Reasonable
O_088	Garage or shed flood affected	Reasonable
O_113	Main building below floor level flood affected	Good
O_105	Back yard flood affected	Good
W_015	Front yard flood affected flooding was experienced in August 1970	No address provided
W_046	Main building - below floor level flood affected, 3 times in last 40 years	Good

ID	Description of Flooding	Calibration Match
W_052	Front and back yard and garage flood affected in one off event 30 years ago	Good
W_068	Back yard flood affected	Good
W_072	Garage or shed flood affected. No address provided.	n/a
W_074	Garage or shed flood affected before kerb and guttering	Reasonable
W_007	Main building - below floor level flood affected	Poor(5)
W_017	Back yard flood affected	Good
W_018	Back yard, garage and main building - below floor level flood affected	Poor(6)
W_021	Front yard and garage flood affected	Good
W_026	Back yard flood affected	Good
W_027	Front and back yard and garage flood affected	Good
W_048	Front yard flood affected	Good
O_082	Business access road flood affected	Good
O_095	Front yard frequently flood affected	Good
O_104	Back yard flood affected	Good
DF2011/0034	Landslip to the south of building, blocking creek and water flowing onto property	N/A
DF2011/0325	Land slide at the back of property. About a metre high and blocking the creek	N/A
DF2011/0336	Tradelink premises flooded	Reasonable
DF2012/0266	Flooding is coming into the back of 16 Chard Rd Brookvale from 77-79 Winbourne Rd Brookvale. Water is pooling up from 77 Winbourne Rd and flooding into the factory building of 16 Chard Rd	Poor(7)
DF2013/0647	Blocked stormwater drain outside property. Capacity problem, could not take the flow of water	N/A
DF2014/0152	Stormwater floods down Beacon Hill Road into Elizabeth Place, and into Early Learning Centre buildings and playground	Reasonable
DF2015/0024	Claims neighbour’s stormwater is directed into property	N/A
DF2016/0575	Fast flowing flood water eroded the creek bank, collapse of bank towards Council footpath. Small cracks appeared on road and side of footpath	Reasonable
DF2018/1482	Pit lid blown off by water frequently. Pushed up and half open, a hazard for pedestrians and reversing cars	N/A
DF2018/0524	Water overflowing out of storm water asset	N/A
DF2018/0608	Major flooding problems, 10cm flooded at garage whilst using sandbags	Poor(8)
DF2019/0358	Path out the front of property floods, cracked and subsided slightly towards the street causing water to pool	N/A
DF2018/0452	Water flow when there is heavy rain	Reasonable
DF2011/0110	Water from Weldon Oval along fence line	Reasonable
DF2012/0098	Large flooding in the street area of Manuela Place	Reasonable
DF2012/0503	Overland flows occur through site	Reasonable
DF2015/0423	Large deep pool of water forms and does not drain away	Reasonable
DF2017/0247	Water eroded soil now exposing concrete	N/A
DF2017/0259	Water runs through like a river under property, might be due to a broken council stormwater pipe	Reasonable
DF2014/0513	Street flooded, cars having to turn back	Reasonable(9)
DF2015/0242	N/A	N/A
DF2015/0352	Flash flooding across garden, footpath and the entire road	Reasonable(9)

ID	Description of Flooding	Calibration Match
DF2016/0031	Water coming into property from the main road	Reasonable
DF2016/0402	Front of property flooded, creating a sinkhole	Reasonable
DF2017/0071	Road flooded, a car had to be towed out	Reasonable(9)
DF2017/0101	Street flooded, issues at school pick up time as children need to walk on road	Reasonable(9)
DF2017/0105	Water flooding from stormwater	Reasonable

(1) Modelling shows property as unaffected and likely is a local drainage issue. Main building of business/ garage is located below street level so drainage from within the property may be limited.
(2) Modelling shows property as unaffected. Shopfront is located approximately 0.6 m above street level and flooding above the main floor level considered unlikely. However the lower ground level is below the street gutter level and it is plausible that runoff may have entered the below ground garage and the comment refers to the garage.
(3) Modelling shows property as unaffected in November 2018. It may be a local drainage issue in the backyard.
(4) Modelling does indicate flow close to the backyard although not quite within this property.
(5) Modelling shows property as unaffected. Topography rises steeply to the rear of the property and there may be local drainage issues with runoff from adjacent lots.
(6) Modelling shows property as unaffected. Property is located in a valley below street level. No Council owned pit/pipe stormwater drainage infrastructure is installed at this location. Due to the small catchment area this is likely to be a local drainage issue rather than overland flooding.
(7) Modelling shows property as unaffected and likely a local drainage issue related to runoff from the neighbouring industrial property.
(8) Modelling shows property as unaffected and likely a local drainage issue. Crest of driveway may be insufficient to prevent inflow from gutter at the front of property.
(9) Each of these comments relates to flooding over road at the sag point in front of Northern Beaches Secondary College, which is reflected in the modelling.

There is generally good agreement between the locations of flooding complaints and modelled flood behaviour for the November 2018 event, with the exception of some properties where local drainage issues occur, due to small catchment areas or inter-lot drainage from neighbouring properties, rather than overland flooding. Most of the flooding complaints, local flood investigations and community reports of flooding in the catchment relate to properties in overland flowpaths or ponded areas at sag points which are adequately captured in the model.

7.4. Summary

Due to the lack of streamflow data and limited availability of peak flood level data, only a limited calibration of the hydrologic and hydraulic models to recorded water levels was possible. Generally the model reproduces flood behaviour as described in Council's customer complaints database and community consultation responses (as discussed in Section 7.3). Recorded peak lagoon levels at Curl Curl Lagoon water level gauge are reasonably matched.

Overland flooding within the catchment is generally the result of short, localised rainfall bursts which may not have been accurately captured by surrounding pluviometers. No peak flood level data was available for calibration outside of the influence of Curl Curl lagoon flooding. The modelled flood behaviour was validated against Council's customer complaints database and community consultation responses. Most customer complaints and community consultation responses relate to overland flowpaths and areas of ponded water at sag points which are generally well reproduced in the model. Given these observations it is considered that the model has been reasonably calibrated to historical flooding in the catchment.

As with all flood studies, the accuracy of the modelling in reproducing recorded flood behaviour could be improved by the inclusion of additional high quality historical flood and rainfall data from future events. In particular historical peak flood level data for the upstream portion of Greendale Creek would allow for a more robust calibration of the model.

It is recommended that following future flood events a program of data collection should be implemented which includes the collection of accurately surveyed peak flood levels as soon as practical following large flood events.

8. DESIGN EVENT MODELLING

8.1. Overview

ARR2019 guidelines for design flood modelling were adopted for this study, including the use of ARR2019 design information for the 50%, 20%, 10%, 5%, 1%, 0.5% and 0.2% AEP events. The PMF event was derived using the Bureau of Meteorology's Generalised Short Duration Method (GSDM) (Reference 13) to estimate the probable maximum precipitation (PMP).

The flows generated by the WBNM model for each design flood event were used as inflows in the calibrated TUFLOW model to define the flood behaviour across the catchment. The ARR2019 temporal patterns, the procedure for the selection of the critical pattern duration and adopted hydrologic model parameters are discussed in the following sections. The resulting flood behaviour simulated in the TUFLOW model is subsequently presented, including an analysis of the results.

8.2. ARR2019 IFD

ARR2019 IFD rainfall information was obtained from the Bureau of Meteorology (BoM). IFD information was sourced for each sub-catchment individually from the BoM's gridded IFD data and applied in the WBNM hydrologic model. A summary of average design rainfall depths across the catchment is provided in Table 11.

Table 11: Average Design Rainfall Depths (mm)

Duration	AEP						
(min)	50%	20%	10%	5%	2%	1%	0.5%	0.2%
30	26	35	41	47	55	61	67	76
45	30	40	47	54	63	70	76	87
60	33	44	51	59	69	77	83	95
90	38	50	58	66	78	87	94	107
120	42	54	63	72	85	95	103	117
180	48	62	72	83	97	109	118	133
270	55	71	83	96	113	127	137	155
360	61	80	93	108	127	143	154	174
540	71	94	111	128	153	173	185	209
720	80	107	126	147	176	198	213	240
1080	94	127	152	178	213	242	261	295

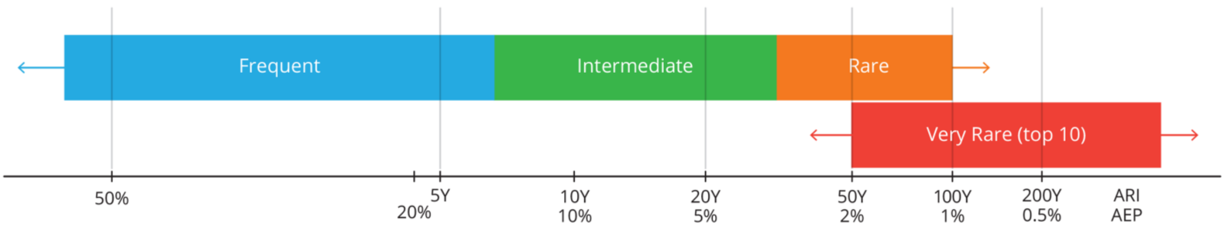
8.3. Temporal Patterns

Temporal patterns are a hydrologic tool that describe how rainfall falls over time and are often used in hydrograph estimation. Previously in ARR1987, a single burst temporal pattern has been adopted for each rainfall event duration. However ARR2019 (Reference 1) discusses the potential inaccuracies with adopting a single temporal pattern, and recommends an approach where an ensemble of different temporal patterns are investigated.

Temporal patterns for this study were obtained from the ARR2019 data hub (Reference 1, http://data.arr-software.org/). A summary of the data hub information at the catchment centroid is presented in Appendix A. There are a wide 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 recommended methodology is to consider an ensemble of design rainfall events and determine the median catchment response from this ensemble.

The ARR2019 method divides Australia into 12 temporal pattern regions, with the Greendale Creek catchment falling within the South East Coast (NSW) region. ARR2019 provides 30 patterns for each duration, which are sub-divided into three temporal pattern bins based on the frequency of the events. Diagram 2 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 ARR2019 that were utilised in this study for the 50% AEP to 0.2% AEP events.

Diagram 2: Temporal Pattern Bins



The method employed to estimate the PMP utilises a single temporal pattern (Reference 13).

8.4. Critical Duration

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

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

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

For this study, there are two primary flood mechanisms of interest:

1. Mainstream / Lagoon: The dominant flood mechanism in the lower catchment is

mainstream flooding from Greendale Creek and Curl Curl Lagoon; and

2. Overland: The small creeks and urban drainage lines within the upper catchment which have smaller contributing catchment areas and fast runoff processes due to urbanisation.

A range of storm durations from 15 minutes to 360 minutes were run through the TUFLOW hydraulic model. For each AEP, a single representative storm was able to be selected that produced peak flood levels closest to (and slightly above) the mean ensemble peak flood levels at every point in the catchment. The adopted critical duration and representative temporal pattern for each event is shown in Table 12.

Table 12: Design Event Critical Durations and Representative Temporal Patterns

Design Event	Critical Duration	Temporal Pattern ID
50% AEP	45 minutes	TP4551
20% AEP	60 minutes	TP4565
10% AEP	60 minutes	TP4565
5% AEP	60 minutes	TP4565
2% AEP	45 minutes	TP4496
1% AEP	45 minutes	TP4496
0.5% AEP	45 minutes	TP4496
0.2% AEP	45 minutes	TP4496
PMF	30/60/120 minutes	GSDM

For the PMP the full range of applicable GSDM durations from 15 minutes to 6 hours was run through the hydraulic model to determine the critical duration for the study area.

For the PMF it was found that the 120 minute storm produced the peak flood levels within Curl Curl Lagoon, the 60 minute was within Greendale Creek and the Brookvale industrial area, and the 30 minute duration was critical in urban overland flow areas. These durations were run and the maximum taken at each location ("enveloped") to produce the flood mapping.

8.5. Design Rainfall Losses and Pre-Burst Rainfall

NSW State Government guidance for ARR2019 implementation (Reference 14) was followed to select appropriate losses for use in design flood modelling. Design rainfall losses were obtained from the ARR2019 Datahub (http://data.arr-software.org/).

Probability neutral burst initial losses were applied directly to the design storm bursts modelled. The continuing loss value from the Datahub was factored by 0.4 and applied to the design storms. Losses are generally in the order of 3 to 14 mm for burst initial loss, and 0.6 to 1 mm/hour for continuing loss. Probability neutral burst initial loss values are dependent on the AEP and duration of the design event. An initial loss of 1.5 mm was applied to impervious surfaces. For the PMF event an initial loss of 0 mm and a continuing loss of 0 mm/hr were applied.

8.6. Areal Reduction Factors

Areal Reduction Factors (ARF) were applied in the WBNM model for the design storm events based on ARR2019 (Reference 1). The design rainfall estimates are based on point rainfalls and in reality, the catchment-average rainfall depth will be less. It allows for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms simultaneously over the whole catchment area. The ARF varies with AEP and duration and the resulting matrix of ARFs for the design storms is shown in Table 13. The equations used to derive these reduction factors can be found in Appendix A.

Table 13: Areal Reduction Factors for the Design Storm Events

Duration	AEP							
(min)	50%	20%	10%	5%	2%	1%	0.5%	0.2%
30	0.94	0.94	0.94	0.93	0.93	0.93	0.92	0.92
45	0.95	0.95	0.95	0.94	0.94	0.94	0.93	0.93
60	0.96	0.96	0.95	0.95	0.94	0.94	0.94	0.93
90	0.97	0.96	0.96	0.96	0.95	0.95	0.94	0.94
120	0.97	0.97	0.96	0.96	0.95	0.95	0.94	0.94
180	0.98	0.97	0.97	0.96	0.96	0.95	0.94	0.94
270	0.98	0.98	0.97	0.97	0.96	0.96	0.96	0.95
360	0.99	0.98	0.98	0.98	0.97	0.97	0.97	0.97
540	0.99	0.99	0.99	0.98	0.98	0.98	0.98	0.98
720	0.99	0.99	0.99	0.99	0.98	0.98	0.98	0.98
1080	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
1440	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
1800	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99

8.7. Blockage

There are multiple factors to be considered in assessing the potential for blockage of culverts and bridges. These considerations include:

- the type and mobility of debris that can be washed into the waterway to block the structure or inlet;
- the dimensions of the debris in comparison to the structure;
- dimensions of the structure in relation to the upstream and downstream channels;
- the presence of piers, service crossings, or other obstructions to flow on which debris can accumulate; and
- catchment land-use.

Design blockage factors were adopted based on the ARR2019 guidance for blockage (Reference 15) with consideration of the control inlet dimensions and debris potential. The catchment upstream of The Kilns consists primarily of steep, forested areas. Landslides may occur in this area resulting in the transportation of debris downstream. Between Pittwater Road and Harbord Road the catchment consists of industrial development. The remainder of the catchment consists primarily of medium density urban residential development, with several parks and sporting fields located adjacent to Curl Curl Lagoon. Based on the catchment land-uses an assessment of debris availability, debris mobility and debris transportability was undertaken. The likelihood of debris blockages was deemed to be in the medium category for the Greendale Creek catchment.

The design blockage factors applied to hydraulic structures are detailed in Table 14.

Table 14: Design blockage factors applied at mainstream hydraulic structures

Structure	Type	Design Blockage (%)
Harbord Road GPT (trash rack)	GPT	100
Western Footbridge	Bridge	0
Eastern Footbridge	Bridge	0
Griffin Road Bridge	Bridge	5
Culverts/ Pipes (headwall entrances)	Culvert	25
Inlet Pits	Stormwater Pit	25

Culverts and pipes with headwall entrances were modelled as 25% blocked due to the potential for debris to bridge the structure, blocking the entrance, or become lodged in the barrel. The trash rack on the Harbord Road GPT was modelled as 100% blocked due to the high likelihood for debris to block the structure. Low blockage factors were applied for clear spanning footbridges and bridges in the lower catchment. Sensitivity analysis was undertaken for these blockage assumptions (see Section 10.5).

8.8. Ocean Level Boundary Conditions

Tailwater level assumptions at the downstream ocean boundary do not have a significant influence on peak flood levels in the area of interest due to the perched nature of Curl Curl Lagoon. The major conditions factors controlling flooding in the lower catchment are the lagoon water levels and the level of the sand berm, which is common for a “Group 4” ICOLL. The primary driving factor for flood levels in the lagoon is the berm height rather than the ocean water level or wave conditions, and the design flood modelling approach reflects this, in accordance with the relevant guideline (Reference 16).

Curl Curl Lagoon has a relatively small lake volume compared to the annual catchment runoff volume, and this characteristic results in frequent overtopping of the berm. When water overtops the berm it erodes, opening the lagoon to the ocean. Once the water has flowed out of the lagoon, tidal forces and wave action begin to push sand back up into the lagoon entrance. There is abundant supply of sand from littoral drift and the wave climate at the entrance builds the berm height back up. This process occurs rapidly at Curl Curl lagoon, and accounts from Council staff indicate that the entrance is often closed again within 24-48 hours. Wave and wind action increases the height of the berm to well above ocean high tide levels. Therefore, the lagoon is closed for a significant majority of the time, and it is the height of the berm that provides the controlling influence over water levels within the lagoon, rather than ocean levels.

Council has an active entrance management strategy for Curl Curl Lagoon which involves mechanical opening of the entrance when the lagoon gauge reaches a specified level (2.2 mAHD at the time of writing). However, it is not reasonable to assume that Council will always be able to open the lagoon and limit the water level to this height. Analysis of the water level gauge since 1991 indicates that the water level has frequently exceeded 2.2 mAHD. During the period of record, the berm height for the events with the 10 highest breakout levels averaged about 2.6 mAHD. The highest recorded level in the lagoon in the 28 year record is 2.71 mAHD (in May 2010). Given that these levels have actually occurred, it is appropriate to assume that the 1% AEP lake level is at least this high.

Peak water levels in Curl Curl Lagoon in a storm event are the result of a complex interaction of the initial water level, berm height, storm runoff, breakout characteristics and ocean conditions. An approach was adopted where the historic gauge levels in the lagoon were analysed to determine various design flood levels in the lagoon. A Flood Frequency Analysis (FFA) was undertaken for recorded water levels in Curl Curl Lagoon at Griffin Road Bridge (213426). This analysis provides a reasonable indicator for the variability of the berm height, and the likelihood of a given height being exceeded in a year. The maximum lagoon level in any given year is very close to the maximum berm height, because typically openings occur with only a shallow overtopping depth across the berm. The gauge had 28 years of recorded heights in the lagoon at the time the FFA was undertaken. A reasonable fit was obtained using either log-normal or Log Pearson Type 3 (LP3) distributions, and the LP3 distribution was adopted. The flood frequency analysis results are graphed in Diagram 3 and summarised in Table 15 for various AEP events. For the PMF event a berm height based on extrapolation past the 1 in 100,000 AEP berm height was adopted. The water levels obtained in the FFA were assumed to directly correspond with berm heights.

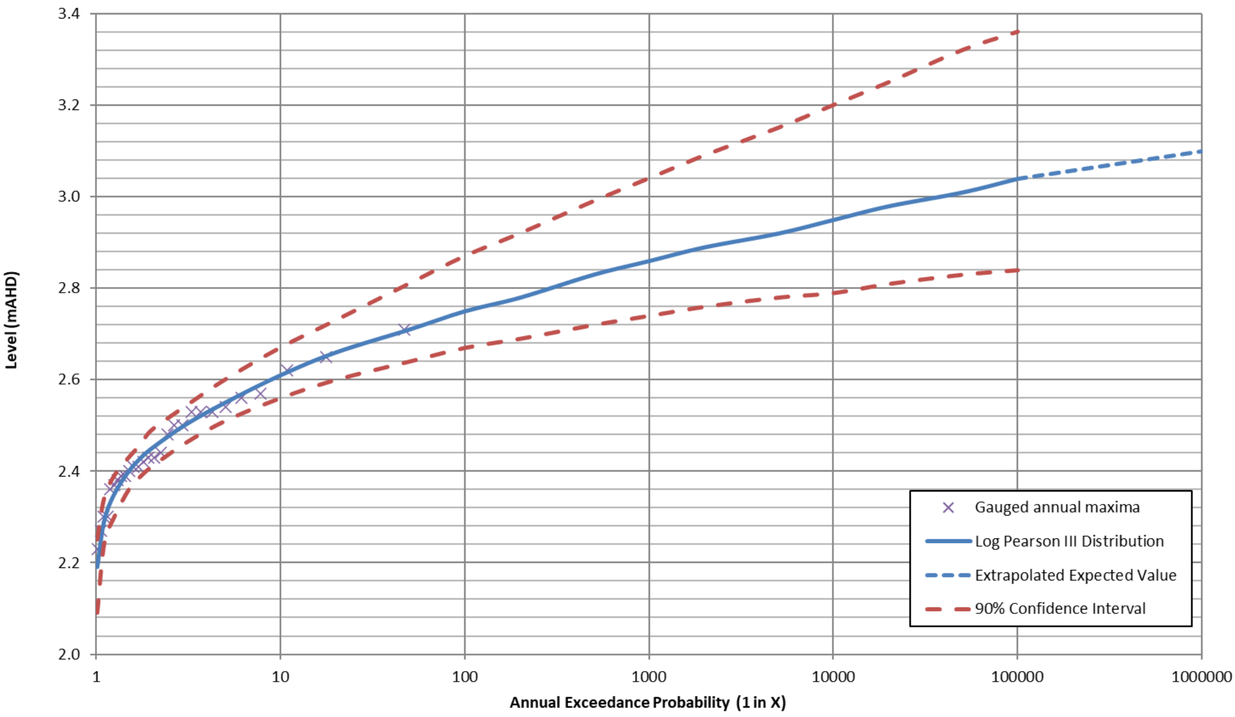


Diagram 3: Flood Frequency Analysis Results for Curl Curl Lagoon Heights

Table 15: Catchment Rainfall Event and Corresponding Design Berm Height

Design Event	Berm Height[1] (mAHD)
50% AEP	2.45
20% AEP	2.55
10% AEP	2.61
5% AEP	2.66
2% AEP	2.71
1% AEP	2.75
0.5% AEP	2.78
0.2% AEP	2.83
PMF	3.10

1. Derived from the FFA results up to the 0.2% AEP event and extrapolated to the PMF event

Design berm heights were set by adopting the corresponding levels determined from the water level FFA. The berm height directly controls water levels in the lagoon. This essentially forces the hydraulic model to simulate the design water level in the lagoon according to the FFA. The adopted berm heights typically resulted in design flood levels approximately equal to, or slightly greater than (within 0.1 m up to the 0.2% AEP event), those determined from the FFA on the downstream side of Griffin Road Bridge. For rare events flood levels were typically higher since these events have faster rates of rise in the lagoon, and produce a greater overtopping depth across the berm. Once the berm is overtopped, the berm is assumed to erode, leaving a 70 m wide channel to the ocean. This erosion was assumed to occur over a period of 6 minutes, based on iterative modelling of the November 2018 historical event. This event appears to provide a reasonable indication of how breakouts are likely to unfold during an intense local storm event. This rapid breakout behaviour ensures that modelled peak water levels in the lagoon remain close to the adopted berm height, and hence the design water levels from the FFA.

The downstream ocean boundary was set to mean sea level (0 mAHD) for design flood analysis. Sensitivity to the downstream water level is contained in Section 10.8. In accordance with Reference 16, to determine the peak 1% AEP velocity, a low tailwater corresponding to the Indian Spring Low Water of -0.95 mAHD should be adopted. While this tailwater level does not change the peak flood levels or velocities within the lagoon with the berm in place (see Section 10.8), an open entrance scenario was run with this tailwater condition and the velocity results were enveloped with the standard 1% AEP runs to obtain the maximum velocity through Curl Curl Lagoon. It is the enveloped velocity that has been mapped for the 1% AEP design flood event in this study. The open entrance scenario with the low tailwater level resulted in lower velocities downstream of Griffin Road bridge, but higher velocities upstream of the bridge by up to 0.6 m/s.

8.9. Initial Water Level Assumptions

The initial water level in Curl Curl Lagoon was set to 2 mAHD. This initial water level is equal to the level adopted for the 2004 Flood Study (Reference 2) and the initial water level recorded for the November 2018 calibration event. This initial water level is considered to be reasonably typical of lagoon water levels prior to a flood event based on recorded water levels at the gauge and slightly below the level of 2.2 mAHD at which Council would generally consider opening the lagoon via active management of the entrance. The sensitivity of design flood levels to this assumption was assessed in Section 10.7. Any open entrance conditions simulated (such as that for the 1% AEP peak velocity) adopted an initial water level equal to the corresponding ocean level.

9. DESIGN FLOOD MODELLING RESULTS

Design flood behaviour simulated by the TUFLOW model is presented in the following maps:

- Peak flood depths and levels in Figure D1 to Figure D9;
- Peak flood velocities in Figure D10 to Figure D18;
- Hydraulic hazard based on the FDM (Reference 5) in Appendix E
- Figure E1 to Figure E3;
- Hydraulic hazard based on the Australian Disaster Resilience (ADR) Handbook (Reference 17) in Figure E4 to Figure E6;
- Hydraulic categories (flood function) in Figure E7 to Figure E9.

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 maps 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 charts:

- Water level hydrographs at road crossings in Figure F2 to Figure F10 (see Figure F1 for locations); and
- Tables of peak flood levels and flows at key locations in Table 17 to Table 19 below (see Figure 15 for locations).

Discussion of these results is provided in the following sections.

9.1. Flood Behaviour

Much of the upper Greendale Creek catchment is affected by shallow (<0.15 m) overland flow in extreme storm events. This is common for urbanised areas, although in this catchment there several locations where overland flow occurs through property rather than along the road reserves. This is a result of roads often not being aligned with the natural gullies of the upper catchment. The risk to life from this shallow flow is low, and damage to property can generally be minimised provided floor levels are raised relative to surrounding ground levels, and some provision is made to allow overland flow through the properties, rather than blocking it completely.

In the upper catchment there is a relatively flat plateau to the north of Warringah Road and water ponds at a sag point in McKillop Road. To the south of Warringah Road, the upper reach of Greendale Creek forms through the joining of several small creek lines. This creek discharges into a 2.5 m (W) x 1.2 m (H) trunk drainage line just upstream of Consul Road. The capacity of this trunk line at the upstream end is exceeded in a 10% AEP event, causing overland flow that follows the drainage line to Pittwater Road. Water flows through properties as well as along Gulliver Street, Alfred Road and eventually ponding on Pittwater Road. Pittwater Road also collects shallow overland flows from the catchment to the north.

Downstream of Pittwater Road, there is more significant flooding through the Brookvale industrial area. The trunk drainage line (now a 1.5 m diameter pipe) discharges into a small open channel downstream of Winbourne Road, before being carried by pipes (2 x 1.8 m diameter) to just downstream of Ethel Avenue. From Ethel Avenue, water is discharged through a series of pipes and small open channels into the Greendale Creek channel immediately downstream of Harbord Road. Through the industrial area, significant flood depths can occur at Mitchell Road (and at properties just downstream), at the Winbourne Estate (at the end of Chard Road) and through properties and roads just upstream of the Greendale Creek channel (along Ada Avenue, Ethel Avenue and Harbord Road). This ponding occurs in events as small as the 50% AEP event.

The open channel portion of Greendale Creek downstream of Harbord Road is approximately 30 m wide and conveys flows towards Curl Curl Lagoon. Flows up to and including the 0.2% AEP are contained within the channel. The main body of the lagoon is between the rock weir and Griffin Road Bridge. Downstream of the bridge the lagoon discharges through North Curl Curl Beach into the ocean. This is dependent on whether the lagoon is open or closed, dictated by a sand bar that forms at the entrance. Within Curl Curl Lagoon, the water level is primarily influenced by this berm height. Different berm heights have been adopted for different design flood events, which dictates the peak flood levels within the lagoon. A summary of these design levels, as well as a comparison with the 2004 Flood Study (Reference 2) is provided in Table 16.

Table 16: Design Flood Levels for Curl Curl Lagoon

Design Event	Berm Height[1] (mAHD)	Design Flood Level[2] (mAHD)	2004 Flood Study Design Level (mAHD) at ocean entrance	2004 Flood Study Design Level (mAHD) at Griffin Road
50% AEP	2.45	2.49	-	-
20% AEP	2.55	2.60	2.59	3.15
10% AEP	2.61	2.67	-	-
5% AEP	2.66	2.73	-	-
2% AEP	2.71	2.78	-	-
1% AEP	2.75	2.82	2.81	5.69
0.5% AEP	2.78	2.86	-	-
0.2% AEP	2.83	2.92	-	-
PMF	3.10	4.60	3.87	5.68

1. Derived from the FFA results and applied in the TUFLOW model
2. Design flood level in the TUFLOW model at upstream Griffin Road Bridge

Downstream of Harbord Road, there are numerous stormwater lines and overland flow paths that discharge into Greendale Creek and Curl Curl Lagoon on both the northern and southern sides. On the southern side, ponding occurs in the vicinity of the Harbord Bowling Club and around Weldon Oval in events as small as the 50% AEP. Flooding also occurs at the rear of properties along Stirgress Avenue and Stewart Avenue. Flooding from the lagoon also affects the Adam Street Reserve. On the northern side, ponding occurs in three locations along Abbott Road between Harbord Road and Pitt Road. To the east of this, there are four main flow paths that traverse residential properties and discharge into Greendale Creek. These are located just to the west of Playfair Road, between Ross Street and Grainger Avenue, between Spring Street and Blackwood Road, and through Surf Reserve. In several cases for these local drainage lines into the lagoon, overland flow is obstructed from reaching the lagoon due to the filled playing fields being higher than the upstream ground levels.

9.2. Tables of Peak Flood Levels, Depths and Flows

A tabulated summary of peak flood levels, depths and flows at selected locations as shown in Figure 15 are detailed in Table 17, Table 18 and Table 19, respectively. These key locations coincide with the key locations used for the sensitivity analysis discussed in Section 10.

Table 17: Peak Flood Levels (mAHD) at Key Locations

ID	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H01	McKillop Road	128.83	128.98	129.06	129.14	129.21	129.28	129.32	129.38	129.73
H02	Upstream 44 Consul Road	30.31	30.51	30.62	30.69	30.88	31.00	31.04	31.03	31.82
H03	Consul Road	29.95	29.97	30.01	30.36	30.59	30.68	30.72	30.78	31.32
H04	Gulliver Street	26.11	26.27	26.34	26.39	26.42	26.47	26.50	26.54	27.07
H05	West of Brookvale Oval (Pittwater Road)	22.51	22.62	22.68	22.73	22.75	22.79	22.82	22.87	23.48
H06	Pittwater Road	18.98	19.07	19.12	19.16	19.22	19.26	19.28	19.32	19.83
H07	Winbourne Road	15.58	15.66	15.69	15.72	15.74	15.76	15.77	15.80	16.18
H08	Upstream Chard Road	10.53	10.97	11.41	11.92	12.24	12.40	12.49	12.62	13.81
H09	Ethel Avenue	5.92	6.13	6.32	6.46	6.58	6.69	6.78	6.91	8.38
H10	Upstream Harbord Road	4.97	5.33	5.80	6.05	6.20	6.33	6.41	6.54	7.79
H11	Harbord Road	4.97	5.01	5.04	5.06	5.12	5.20	5.26	5.34	6.80
H12	Downstream Harbord Road	3.63	3.87	4.01	4.13	4.24	4.38	4.49	4.67	6.71
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	3.53	3.77	3.91	4.03	4.13	4.27	4.38	4.56	6.47
H14	Downstream Rock Weir	2.49	2.60	2.68	2.76	2.80	2.86	2.92	2.99	4.66
H15	Upstream Griffin Road	2.49	2.60	2.67	2.71	2.77	2.82	2.85	2.91	4.60
H16	Downstream Griffin Road	2.48	2.60	2.65	2.67	2.77	2.78	2.79	2.86	4.46
H17	Upstream Berm	2.41	2.49	2.56	2.64	2.63	2.71	2.77	2.78	3.04
H18	Bennett Street	9.39	9.41	9.42	9.43	9.46	9.47	9.48	9.49	9.63
H19	Mitchell Road	9.98	10.00	10.13	10.23	10.30	10.36	10.41	10.47	11.10
H20	Pitt Road	13.35	13.36	13.39	13.41	13.45	13.46	13.47	13.50	13.76
H21	Abbott Road	4.01	4.06	4.09	4.11	4.13	4.15	4.16	4.19	4.64
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.71	3.71	3.72	3.74	3.74	3.75	3.76	3.76	4.66
H23	Upstream Reub Hudson Oval (Abbott Road)	10.06	10.23	10.36	10.45	10.52	10.59	10.63	10.68	10.98
H24	Downstream Northern Beaches Secondary College	9.96	10.21	10.34	10.45	10.54	10.60	10.64	10.69	11.01
H25	Manuela Place	5.08	5.24	5.29	5.33	5.33	5.35	5.37	5.39	5.66
H26	Upstream Western Footbridge	3.16	3.39	3.50	3.61	3.70	3.83	3.93	4.10	5.56
H27	Downstream Western Footbridge	3.15	3.38	3.49	3.60	3.68	3.80	3.89	4.05	5.52
H28	Upstream Eastern Footbridge	2.51	2.65	2.71	2.80	2.88	2.93	2.98	3.09	4.73
H29	Downstream Eastern Footbridge	2.49	2.62	2.68	2.77	2.82	2.86	2.92	3.01	4.68

Table 18: Peak Flood Depths (m) at Key Locations

ID	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
D01	McKillop Road	0.31	0.46	0.55	0.62	0.69	0.76	0.80	0.86	1.21
D02	Upstream 44 Consul Road	1.51	1.70	1.82	1.89	2.08	2.20	2.24	2.23	3.02
D03	Consul Road	0.00	0.02	0.07	0.41	0.65	0.74	0.78	0.83	1.38
D04	Gulliver Street	0.00	0.16	0.23	0.28	0.31	0.36	0.39	0.43	0.96
D05	West of Brookvale Oval (Pittwater Road)	0.25	0.37	0.43	0.48	0.50	0.54	0.57	0.62	1.23
D06	Pittwater Road	0.51	0.59	0.64	0.68	0.75	0.78	0.81	0.85	1.36
D07	Winbourne Road	0.12	0.20	0.23	0.26	0.27	0.29	0.31	0.33	0.71
D08	Upstream Chard Road	1.59	2.02	2.47	2.98	3.30	3.46	3.55	3.68	4.87
D09	Ethel Avenue	0.40	0.60	0.79	0.93	1.05	1.17	1.25	1.39	2.85
D10	Upstream Harbord Road	1.90	2.26	2.73	2.97	3.13	3.25	3.34	3.47	4.72
D11	Harbord Road	0.07	0.12	0.14	0.16	0.23	0.31	0.37	0.45	1.91
D12	Downstream Harbord Road	2.80	3.04	3.18	3.30	3.41	3.55	3.66	3.84	5.88
D13	Downstream Harbord Road GPT (Gross Pollutant Trap)	2.43	2.67	2.81	2.93	3.03	3.17	3.27	3.46	5.37
D14	Downstream Rock Weir	1.82	1.93	2.01	2.09	2.13	2.19	2.25	2.32	3.99
D15	Upstream Griffin Road	1.92	2.03	2.10	2.15	2.21	2.25	2.28	2.35	4.03
D16	Downstream Griffin Road	2.38	2.49	2.55	2.57	2.67	2.68	2.69	2.76	4.36
D17	Upstream Berm	1.69	1.76	1.84	1.92	1.91	1.99	2.05	2.06	2.32
D18	Bennett Street	0.33	0.34	0.36	0.37	0.39	0.40	0.41	0.42	0.57
D19	Mitchell Road	0.02	0.05	0.18	0.27	0.35	0.41	0.45	0.51	1.14
D20	Pit Road	0.16	0.17	0.20	0.22	0.26	0.27	0.29	0.31	0.58
D21	Abbott Road	0.06	0.11	0.14	0.16	0.18	0.20	0.21	0.23	0.69
D22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	0.24	0.25	0.25	0.27	0.28	0.29	0.29	0.30	1.19
D23	Upstream Reub Hudson Oval (Abbott Road)	0.13	0.29	0.43	0.52	0.59	0.65	0.69	0.74	1.05
D24	Downstream Northern Beaches Secondary College	0.17	0.42	0.55	0.67	0.76	0.82	0.86	0.91	1.23
D25	Manuela Place	0.26	0.42	0.48	0.51	0.51	0.53	0.55	0.57	0.84
D26	Upstream Western Footbridge	1.99	2.22	2.33	2.44	2.53	2.66	2.76	2.93	4.39
D27	Downstream Western Footbridge	2.32	2.54	2.66	2.76	2.85	2.96	3.05	3.21	4.68
D28	Upstream Eastern Footbridge	2.00	2.14	2.20	2.29	2.37	2.42	2.48	2.59	4.22
D29	Downstream Eastern Footbridge	2.02	2.15	2.21	2.30	2.35	2.39	2.45	2.54	4.21

Table 19: Peak Flood Flows (m^3/s) at Key Locations

ID	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Q01	Upstream of The Kilns (West)	2.6	3.2	3.5	4.0	5.1	5.7	6.2	6.7	18.9
Q02	Upstream of The Kilns (East)	0.8	1.0	1.2	1.4	1.7	2.0	2.5	3.1	16.3
Q03	Upstream Consul Road	4.0	5.2	6.1	7.0	8.5	9.9	10.8	12.4	38.7
Q04	Consul Road	0.0	0.0	0.3	0.6	1.8	3.2	4.1	5.6	35.4
Q05	Gulliver Street	0.9	3.6	5.3	6.9	7.5	9.7	10.9	13.7	65.9
Q06	Downstream Winbourne Road	6.4	7.4	8.2	8.9	9.5	10.2	10.7	11.4	29.7
Q07	Upstream Harbord Road	13.6	15.8	17.7	19.7	21.9	24.2	26.2	29.6	97.0
Q08	Downstream Harbord Road	15.7	18.8	21.5	24.3	26.8	30.3	33.3	38.3	166.1
Q09	Upstream Western Footbridge	17.3	21.9	25.0	28.0	30.3	33.8	36.7	41.6	141.4
Q10	Upstream Eastern Footbridge	21.4	28.6	31.9	35.0	37.8	41.9	45.4	51.9	208.3
Q11	Downstream Rock Weir	27.8	35.5	38.4	41.2	46.8	50.8	54.8	62.3	245.9
Q12	Downstream Griffin Road	49.0	55.9	61.3	65.6	67.5	73.4	76.6	81.7	295.6
Q13	Curl Curl Lagoon Berm	55.3	64.3	67.8	88.0	78.1	90.1	104.0	97.9	316.9
Q14	Adams Street	0.5	0.6	0.8	0.9	1.4	1.6	1.7	2.0	9.9
Q15	Bennett Street	0.6	0.9	1.1	1.4	2.1	2.4	2.7	3.1	11.3
Q16	Manuela Place	0.0	0.0	0.1	0.2	0.2	0.3	0.4	0.6	4.8
Q17	Pit Road	0.4	0.5	0.6	0.8	1.2	1.4	1.6	1.9	8.7
Q18	Abbott Road	1.0	2.0	2.9	3.8	4.7	5.9	6.9	8.5	42.2
Q19	Upstream Community Centre (Abbott Rd)	0.6	0.7	1.0	1.4	1.5	1.7	1.9	2.3	12.3
Q20	Upstream Reub Hudson Oval (Abbott Rd)	0.2	0.4	0.6	0.8	1.1	1.2	1.4	1.7	11.1
Q21	Downstream Northern Beaches Secondary College	0.0	0.0	0.1	0.1	0.2	0.3	0.4	0.5	8.3

9.3. Hydraulic Hazard Categorisation

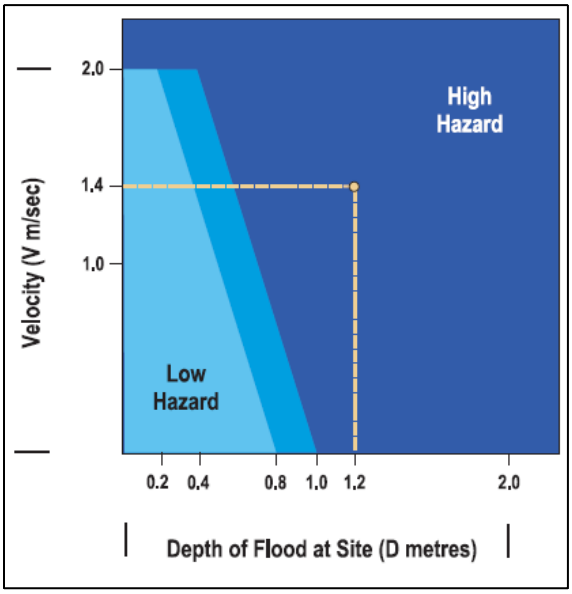
Hydraulic hazard is a measure of potential risk to life and property damage from flooding and is typically determined by considering the depth and velocity of floodwaters. In recent years, there have been a number of developments in the classification of hazards. Research has been undertaken to assess the hazard to people, vehicles and buildings based on flood depth, velocity and velocity depth product.

Provisional hazard categories have been determined for the Greendale Creek catchment by two methods - one in accordance with the NSW FDM (Reference 5), and the other in accordance with the Australian Disaster Resilience Handbook Collection (Reference 17). Each method of provisional flood hazard categorisation is discussed below.

9.3.1. Floodplain Development Manual

Appendix L of the NSW FDM (Reference 5) provides one method for hydraulic hazard, which is shown in Diagram 4. In this study, the transition zone was considered to be high hazard.

Diagram 4: Provisional "L2" Hydraulic Hazard Categories (Reference 5)



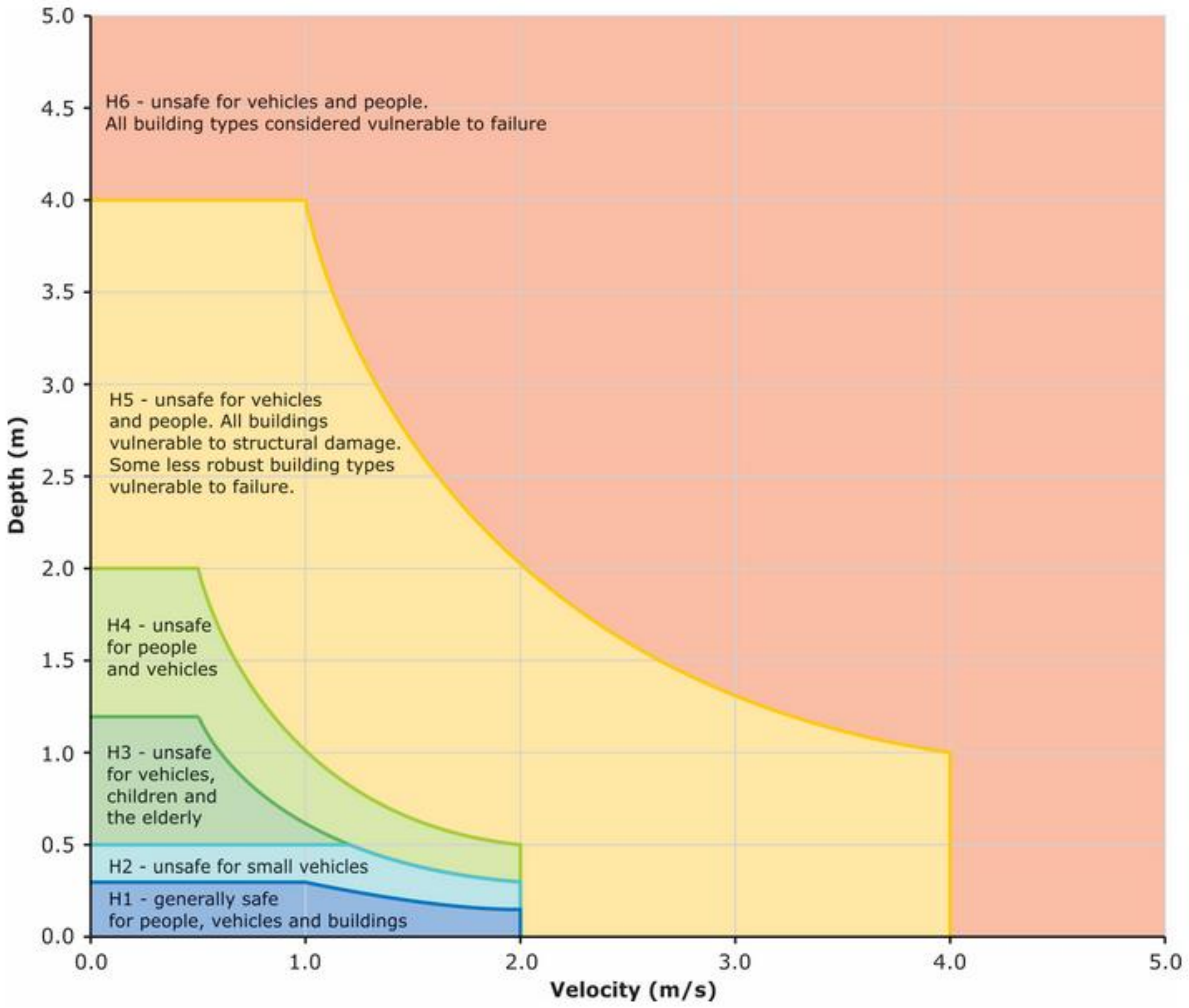
The hydraulic hazard utilising the FDM categorisation is mapped on Appendix E
Figure E1 to Figure E3 for the 5% AEP, 1% AEP and PMF events respectively. The FDM hazard categorisation has been included for applicability to existing council policy documents that may refer to this hazard classification.

In the 5% and 1% AEP events, high hazard areas are generally restricted to the creek channels and Curl Curl Lagoon, with some areas of deeper ponding on roads also classified as high hazard. In the PMF event, high hazard covers a much larger area, including areas adjacent to Curl Curl Lagoon, most of the roads within the Brookvale industrial area and other roads that have a high conveyance of flows throughout the catchment.

9.3.2. Australian Disaster Resilience Handbook Collection

The Australian Disaster Resilience (ADR) Handbook Collection deals with floods in Handbook 7 (Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia). The supporting guideline 7-3 (Reference 17) contains information relating to the categorisation of flood hazard. A summary of this categorisation is provided in Diagram 5.

Diagram 5: General Flood Hazard Vulnerability Curves (ADR)



This classification provides a more detailed distinction and practical application of hazard categories, identifying the following 6 classes of hazard:

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

The hazard maps using the ADR classification are shown in Figure E4 to Figure E6 for the 5% AEP, 1% AEP and PMF events respectively. In the 5% AEP and 1% AEP events, H4 and H5 hazard are typically contained to the creek channel and Curl Curl Lagoon. In areas of deeper ponding, the hazard is H2 and H3, with the remaining areas affected by overland flooding being H1. In the PMF event the hazard reaches H6 in the Greendale Creek channel and H5 in the Brookvale industrial area.

9.4. Flood Function

Identification of flood function 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. Typically, development within floodway or flood storage areas would be likely to push water into other areas and redistribute the flood risk, unless the development is carefully designed to avoid these impacts.

The 2005 NSW Government's FDM (Reference 5) 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 depending on knowledge of an area and flood behaviour, hydraulic modelling and previous experience in categorising flood function. The method defined by Howells *et al* (Reference 18), relies on combinations of velocity and depth criteria to define the floodway.

For this study, hydraulic categories were defined by the following criteria, which correspond in part with the criteria proposed by Howells *et al*, 2003 (Reference 18):

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

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

- <u>Flood Storage</u> comprises areas outside the floodway where peak depth > 0.2 m; and
- <u>Flood Fringe</u> comprises areas outside the Floodway where peak depth < 0.2 m.

Figure E7 to Figure E9 show the provisional hydraulic categorisations for the Greendale Creek catchment for the 5% AEP, 1% AEP and PMF events, respectively. In the 5% AEP and 1% AEP events the floodway is generally restricted to the Greendale Creek channel and some of the roads that have a high conveyance of flows. Flood storage areas include the lagoon and areas of deep ponding adjacent to the creek and through the Brookvale industrial area. In the PMF event, the floodway and flood storage areas are extensive, covering much of the Brookvale industrial area and areas surrounding Greendale Creek and Curl Curl Lagoon.

9.5. Information to Support Decisions on Activities in the Floodplain and Managing Flood Risk

It is considered good practice to permit land use and development that is compatible with the nature of flooding in a particular area. The following sections provide information that is relevant to support decisions on activities in the floodplain and managing flood risk.

9.5.1. Road Inundation

An analysis of road inundation was undertaken at key locations (Figure F1) in the study area. Stage hydrographs showing the depths at selected roads and crossings of Greendale Creek and Curl Curl Lagoon are shown in Figure F2 to Figure F10.

9.5.2. Pipe Capacity Assessment

The design flood results were used to determine how frequently the stormwater pipe system capacity is likely to be exceeded throughout the catchment. Defining the capacity of a pipe is not straightforward, as it depends on multiple factors including shape, the flow regime (e.g. upstream or downstream controlled), inlet and outlet connection, pipe grade, and other factors.

TUFLOW provides output indicating the proportion of the cross-section area of a pipe that has flow in it. For this assessment, pipes were assumed to be "full" when the flow area was equal to or in excess of 85% of the pipe's cross-sectional area. This is the point at which circular pipes tend to be close to their most efficient, since at 100% of cross-sectional area the additional friction from the top of the pipe reduces pipe conveyance. Similarly, box culverts designed for a supercritical flow regime will typically be designed for free surface flow approximately 80% of the depth of the culvert, as when flow touches the pipe soffit it will typically "trip" the flow regime to become pressurised, resulting in lower capacity, depending on the pipe grade. Additionally, due to energy losses associated with adjoining pits, inlets, bends etc., some culverts may never reach "100% full" capacity by waterway area, although they may be 90% full for a range of design events (e.g. from the 5% AEP through to the PMF). In such circumstances, it is informative to know the design storm for which the pipe is almost at its maximum capacity.

Figure 16 shows the results of the pipe capacity assessment for the modelled range of design events. A large proportion (approximately 70%) of the pipes are full in the 50% AEP event.

9.5.3. Flood Planning Constraint Categories

Guideline 7-5 of the Australian Disaster Resilience Handbook Collection (Reference 19) recommends using Flood Planning Constraint Categories (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.

Council's existing planning policies and framework do not reference FPCCs, but they can still provide value for Council's internal strategic planning activities to understand flood constraints. Specific developments should be assessed on a merits basis taking into consideration the full range of flood behaviour possible at that location and the type of development proposed.

The following range of flood map outputs were considered and combined to develop a Flood Planning Constraint Category Map for the study area:

- Flood Extents,
- Hydraulic Hazard,
- Flood Function,
- Flood Emergency Response Classifications for Communities, and
- Flood Planning Area.

The methodology adopted was to delineate the floodplain into four planning categories, consistent with the approach from (Reference 19), adopting the 1% AEP as the defined flood event, and the 0.2% (1 in 500) AEP as the larger event. The definition for each FPCC category is provided below:

- FPCC1: Flow conveyance (floodway) and storage areas in the 1% AEP and H6 hazard areas in the 1% AEP. The majority of developments and uses have adverse impacts on flood behaviour. Consider limiting uses and development to those compatible with the flood behaviour. Development involving structures or fill in these areas is likely to produce adverse flood impacts in other areas.
- FPCC2: Flow conveyance (floodway) areas in the 0.2% AEP, H5 hazard category in the 1% AEP, H6 in the 0.2% AEP. Consider compatibility of developments and users with rare flood flows in the area. Many uses and developments will be vulnerable to flood hazard. Consider limiting new uses to those compatible with the flood hazard. Consider treatments to reduce the flood hazard which will not adversely affect flood behaviour. Consider evacuation difficulties.
- FPCC3: Outside FPCC2, but within the Flood Planning Area. Hazardous conditions may exist creating issues for vehicles, people and buildings. Standard land-use and development controls aimed at reducing damage and exposure of the development to flooding in the 1% AEP are likely to be suitable. Consider the need for additional conditions for emergency response facilities, key community infrastructure and vulnerable users within these areas due to potential access difficulties.
- FPCC4: Outside FPCC3, but within the PMF extent. Consider the need for special development conditions for emergency response facilities, key community infrastructure and land uses with vulnerable users.

Any changes in land use or new developments should be compatible with the nature of flooding in the area. The information contained in the flood study regarding the flood hazard, flood function and evacuation potential should be used in land use planning activities to ensure that proposed land uses do not increase the flood risk to people or property. The results obtained using the above methodology are mapped on Figure E10

9.5.4. Flood Planning Area

9.5.4.1. 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 over-floor flooding of buildings.

The Floodplain Development Manual (Reference 5) 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.

9.5.4.2. 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:

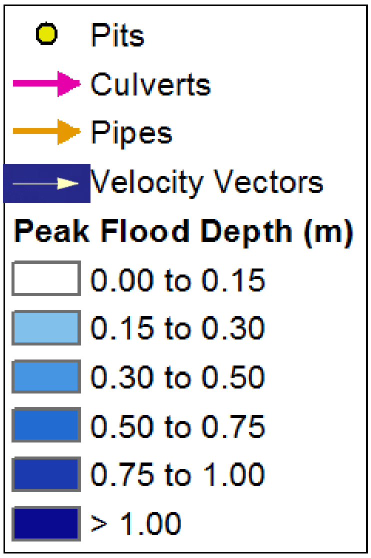
- Mainstream flooding: Areas along the main creeks or trunk drainage alignment, where flow is sufficiently deep and there is sufficient relief that freeboard can be added to the flood surface and the extent then "stretched" to include adjacent land. The mainstream part of the study was defined as Greendale Creek downstream of Harbord Road, including Curl Curl Lagoon. The FPA along this reach was defined as the 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, because the stretching extends across land in a way that would not actually occur even with significant additional flow from a much larger storm, and may even extend beyond the modelled PMF extent. It is therefore appropriate to use a modelled design flood event larger than the 1% AEP event to account for the uncertainty in the results, instead of adding freeboard and stretching. The advantage of this approach is that it includes consideration of flow momentum from actual model results. In overland flow areas, it was considered appropriate to use the filtered 0.2% AEP extent as the preliminary definition of the Flood Planning Area (FPA). The following filters have been applied to the 0.2% AEP event:
    - **Depth Filter** – Exclude results below 150 mm depth;
    - **Velocity-Depth Filter** – Include results if the Velocity x Depth product > 0.3 m^2/s (even if previously excluded by the Depth Filter); and
    - **Small Pond Filter** – Remove isolated ‘puddles’ or ‘orphans’ smaller than 100 m^2.

The resultant extent was then intersected with the cadastre to find lots with only minimal affectation. Lots with a total FPA extent within the lot of less than 15 m^2 were removed from the extent. These were typically lots adjacent to flooding within the road reserve, however, the modelled extent just touched the boundary of the lot, and it was not considered necessary to identify those lots as flood affected. This was subject to a manual review process with consideration of the flood behaviour, flow path formation and the extents within adjacent properties. The modelling results show flow around buildings and for the purpose of the FPA, the ‘holes’ left by buildings were filled in to create a continuous extent. Figure 17 identifies the extent of the preliminary FPA (combined mainstream and overland) developed using the methodology above.

9.6. Descriptions of Hot Spots

Diagram 6: Legend for Hot Spot Diagrams

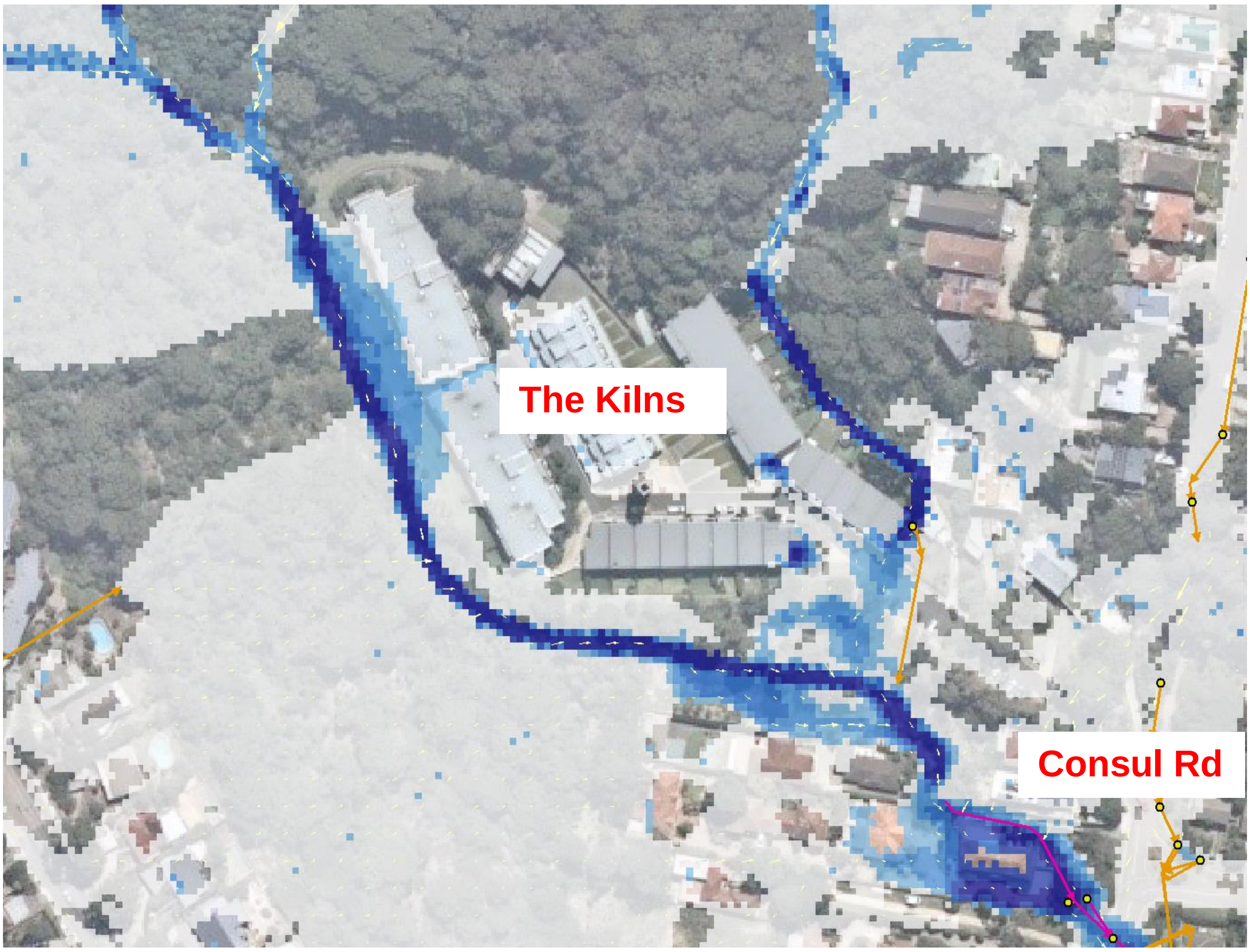


A description of the flow behaviour at locations with the most significant flood risk, or flooding “hot spots” is provided below.

The information shown on the diagram for each hot-spot is as per the legend shown in Diagram 6 to the left. The flood depths indicated on the diagrams in this section are for the 1% AEP event.

9.6.1. The Kilns to St Augustine's College

Diagram 7: "The Kilns" at 48A Consul Road



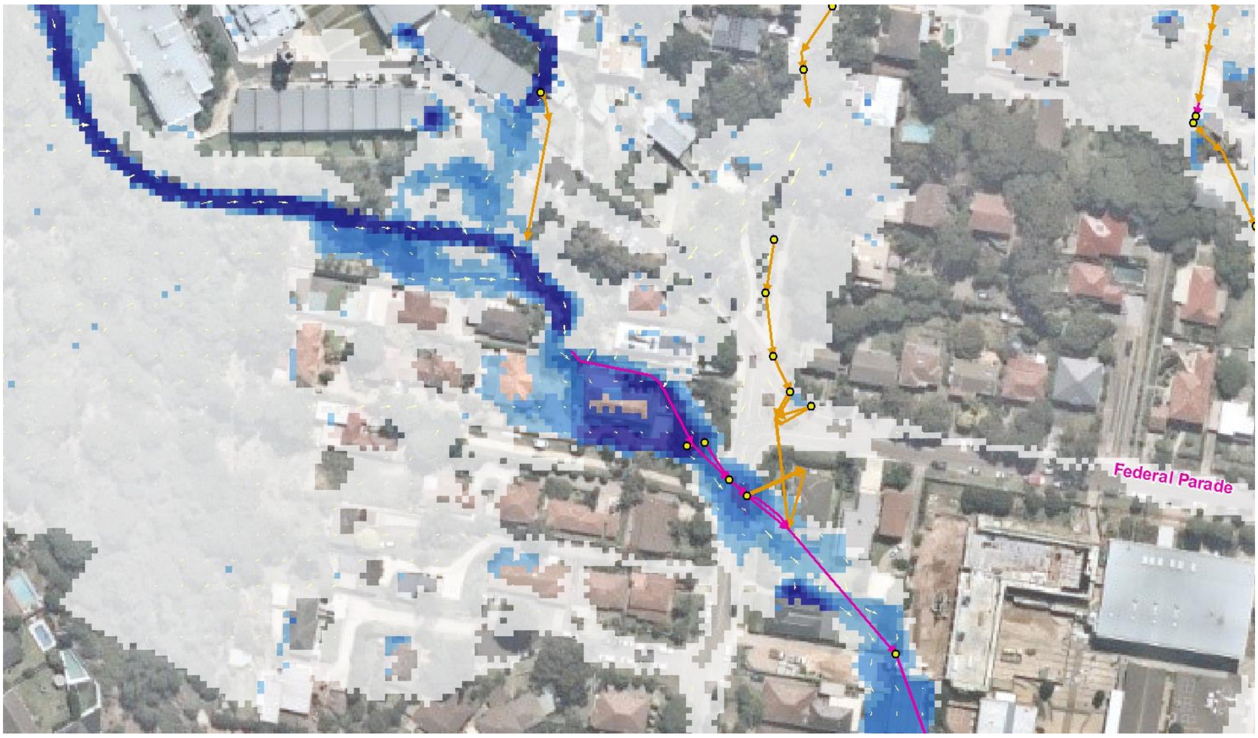
"The Kilns" development lies between two streams that originate at the top of the escarpment at Warringah Road. Each of the streams are intercepted by man-made channels and diverted around the development, before joining on the downstream side and flowing through other properties towards Consul Road (Diagram 7). It is likely that prior to the construction of the brick-making facilities, the creeks went through the site, and the area was filled for the industrial activity.

Photographs of the western flow path are shown in Appendix B, Photo B1 to Photo B5. Photographs of the eastern flow path and cutoff wall are shown in Photo B6 to Photo B10. Photo B11 to Photo B16 show the Greendale Creek channel at the downstream outlet from the Kilns as it flows through private property to Consul Road.

This flow path has the potential to inundate properties on the western side of The Kilns that front onto the channel. Flooding reportedly occurred in the past when there was a landslide of material from the escarpment into the channel. Residents report that there have been fewer issues since the landslide material was removed, but modelling indicates the channel does not have 1% AEP capacity, and in this event some overland flow into the yards and possibly above floor will occur at this location.

Downsteam of the Kilns the channel flows through low lying properties, particularly 44, 46 and 48 Consul Road. There is a local sag point west of Consul Road, and floodwaters will pond to significant depths in 44 Consul Road when the pipe capacity is exceeded (between 20% AEP and 10% AEP capacity). When flows exceed the Consul Road pipe capacity, they overtop Consul Road and flow south-easterly through 35-47 Consul Road, and then through St Augustine's College (see Diagram 8).

Diagram 8: Flow path from the Kilns to St Augustine's College



9.6.2. St Augustine's College to Pittwater Road

Diagram 9: Flow path from St Augustine's College to Pittwater Road

The overland flow path through St Augustine's college follows the same path as the below-ground trunk drainage network (which has 50% AEP to 20% AEP capacity). The flow path exits St Augustine's College at Gulliver Street, where there is a split in both the sub-surface drainage network and the overland flow paths (Diagram 9). The original flow path continues south-eastwards through private property, with some flow being diverted along Gulliver Street and then Alfred Street. These flow paths recombine at the sag point on the bend in Pittwater Road south of Brookvale Oval.

Pictures of the properties and flow paths along Gulliver Street are shown in Appendix B (Photo B17 to Photo B20).

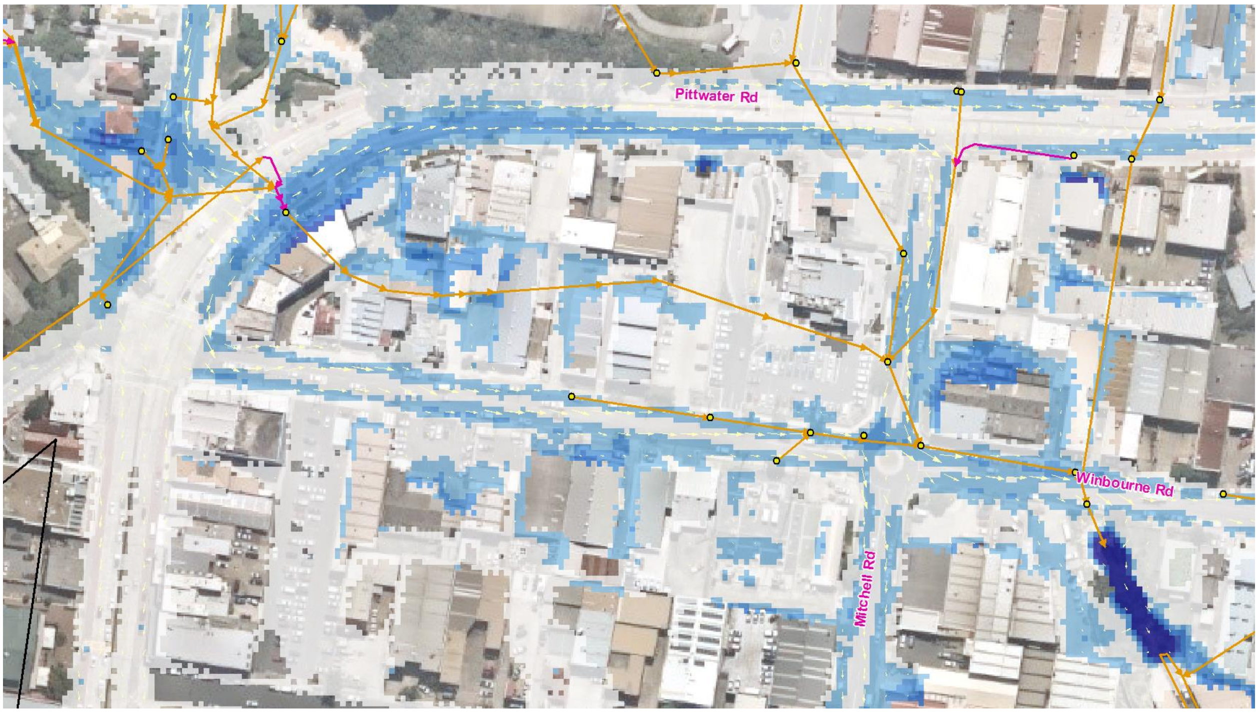
9.6.3. Pittwater Road to Winbourne Road

In the vicinity of Brookvale Oval, there is a bend in Pittwater Road. The road levels are lower on the inside of the bend, towards the south-east, and at this location there is an obstruction to flow from a continuous row of commercial properties (712 to 718 Pittwater Road). The sub-surface trunk drain passes under 712 Pittwater Road, and then through the rear of properties on the northern side of Winbourne Road (Diagram 10). Modelling indicates this trunk drain reaches capacity in a 50% AEP event or smaller. Events exeeding this capacity will cause ponding of water on Pittwater Road, until it reaches a depth sufficient to flow around the commercial properties, with a three-way split:

- down Winbourne Road;
- through the gap between 718 and 724 Pittwater Road; and
- east along Pittwater Road to Mitchell Road.

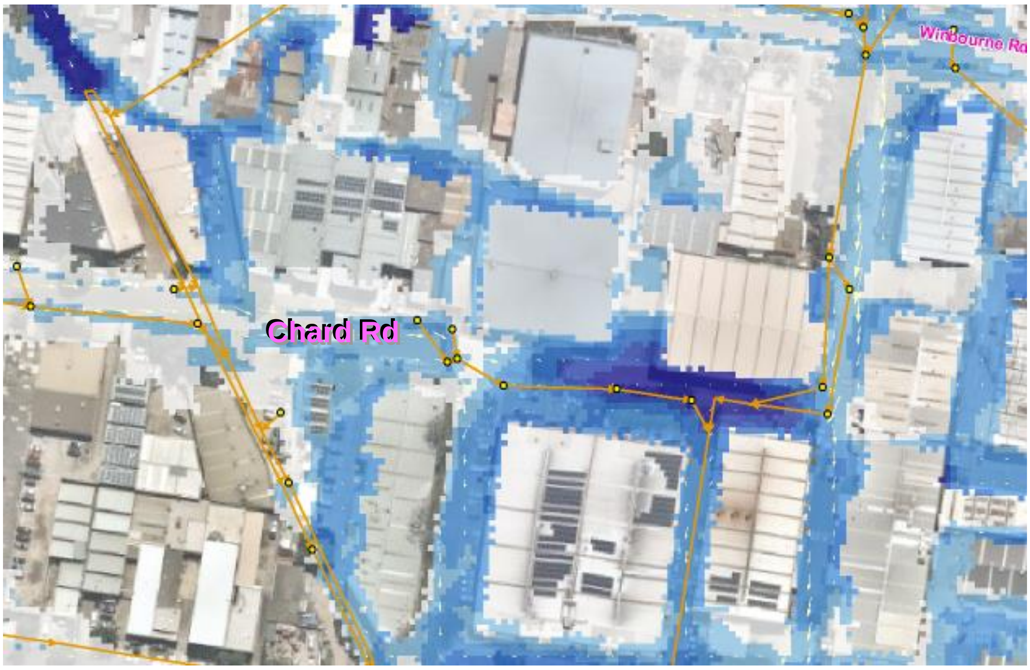
These flow paths recombine near the intersection of Mitchell Road and Winbourne Road, where there is a short section of open channel running south-east from Winbourne Road (see Photo B25 to Photo B27).

Diagram 10: Flow path from Pittwater Road to Winbourne Road



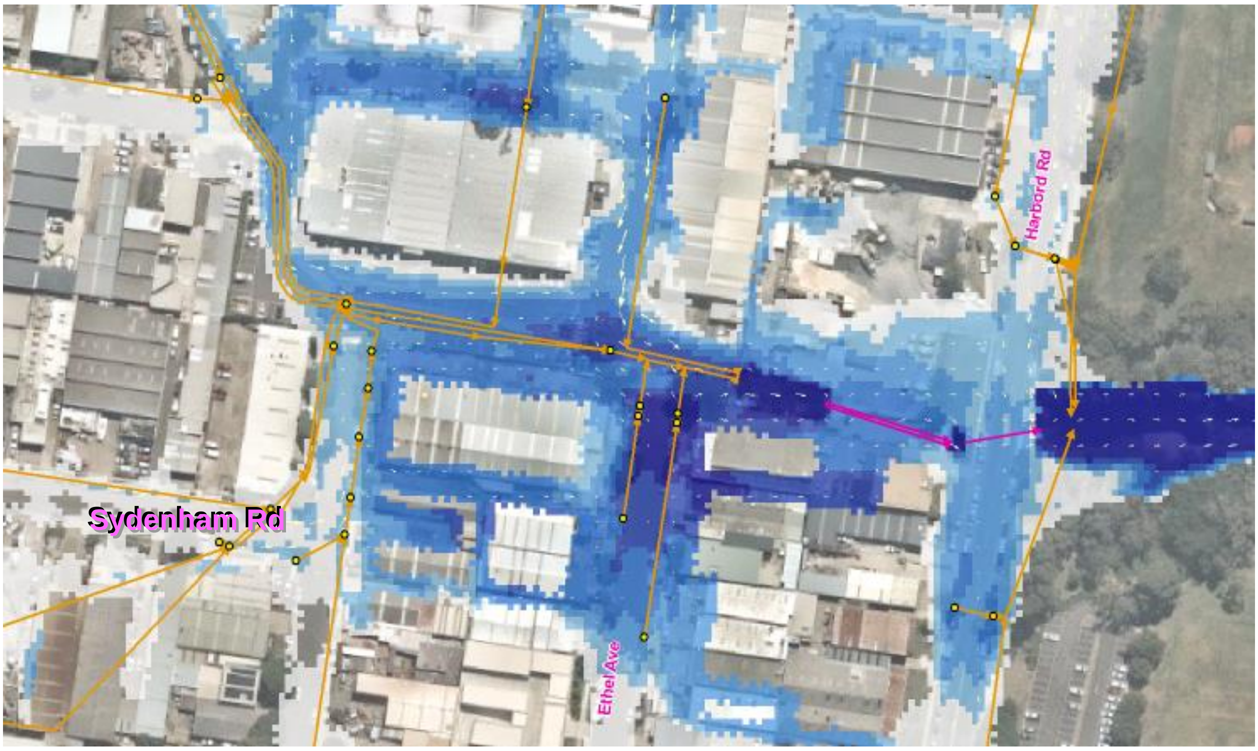
9.6.4. Winbourne Road to Harbord Road

Diagram 11: Sag Point at eastern end of Chard Road



In a 1% AEP event, modelling indicates there will be widespread overland flooding throughout the industrial poperties from downstream of the open channel near Winbourne Road to the open channel at Harbord Road. The overland flow would occur through or around most buildings in this area, not just those in the vicinity of the stormwater network. The most notable areas of significant flooding depth are the low lying area at the eastern end of Chard Road (Diagram 11), and the area between the eastern end of Sydenham Road and the Harbord Road culverts (Diagram 12, also Photo B28 to Photo B30). The pipe drainage capacity throughout this area is generally exceeded in a 50% AEP or 20% AEP event.

Diagram 12: Overland flow paths in Industrial Area near Harbord Road



9.6.5. Pittwater Road at West Street

There is a local sag point in Pittwater Road at West Street. When the stormwater pipe capacity is exceeded (less than 50% AEP capacity), floodwaters will pond in Pittwater Road to depths of up to 0.8 m in the 1% AEP event, with overland flow exiting the sag point via West Street and continuing across the main trunk drainage line towards Carter Road (Diagram 13). Shallow overland flow is likely occur through properties on West Street, Carter Road and Winbourne Road along this flow path.

Diagram 13: Overland flow paths at Pittwater Road near West Street

9.6.6. Pitt Road to Abbott Road

Diagram 14: Overland flow paths from Pitt Road to Abbott Road

There are several stormwater drainage lines running downhill from Pitt Road to Abbott Road, which flow through private property rather than along the road network. Three of these drainage lines are depicted in Diagram 14. The capacity of these lines varies, with some less than 50% AEP capacity and some large enough to convey the 1% AEP flow. When flow exceeds the pipe capacity along these lines, overland flow is expected to occur through the properties.

9.6.7. Playing Fields South of Greendale Creek

Diagram 15: Low-lying terrain between Stirgess Avenue and Weldon Oval / Stirgess Reserve

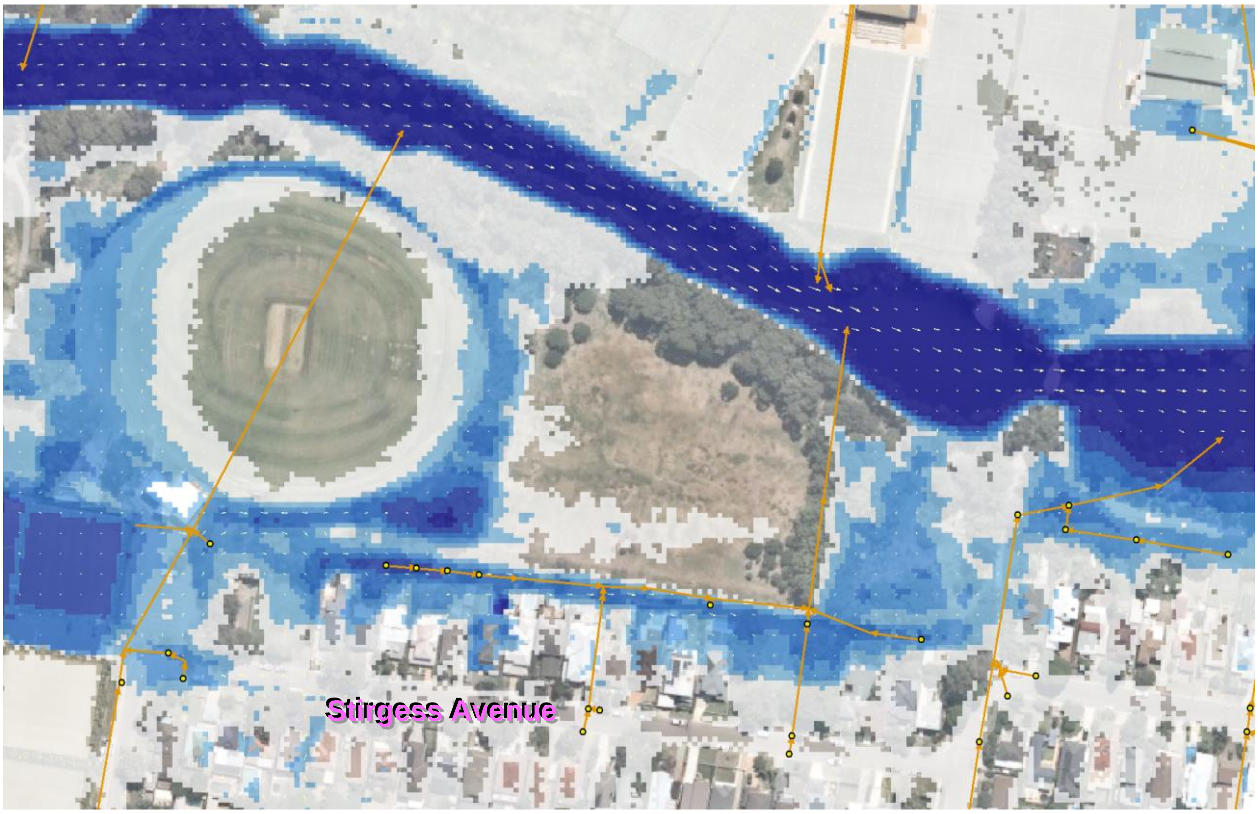
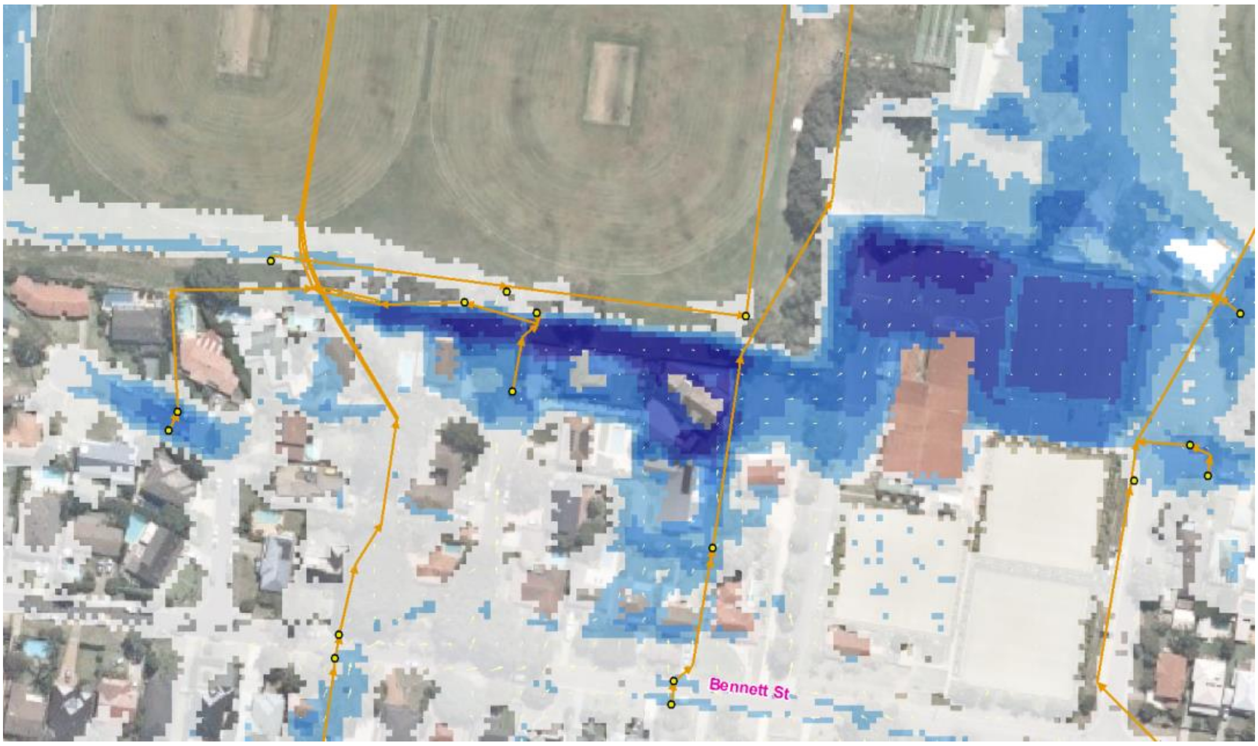


Diagram 16: Low-lying terrain between Bennett Street and Cricket Ovals



There are several playing fields located on reclaimed land south of Greendale Creek. Due to the filling of these areas the playing field surfaces are significantly higher than some of the land to the south, resulting in localised low points where water can accumulate. Modelling indicates the pipes draining these areas are generally full in a 50% AEP event, and flooding will occur in more severe events along the rear of properties on Stirgess Avenue, through the Harbord Bowling and Recreation Club, and other properties that back onto the playing fields (Diagram 15 and Diagram 16).

9.6.8. Harbord Park to Bennett Street

Diagram 17: Overland flow path from Harbord Park to Bennett Street

There is a stormwater drainage line from Harbord Park to Bennett Street, across Brighton Street. The drainage line runs through private property and has capacity ranging from 50% AEP to 10% AEP. In larger storm events exceeding the pipe capacity, overland flow will occur through several properties in this area as indicated on Diagram 17.

9.6.9. Mitchell Road Sag Point near Powells Road

Diagram 18: Mitchell Road Sag Point near Powells Road

There is a confluence of two drainage lines at a sag point on Mitchell Road near Powells Road (Diagram 18). The stormwater pipes downstream of this sag point have a capacity between 50% AEP and 20% AEP, and overland flow will occur in larger events, with depths exceeding 0.5 m in Mitchell Road and properties to the east towards Orchard Road. Overland flow is blocked from exiting the area by buildings and fences between Mitchell Road and Orchard Road / Ada Avenue.

10. SENSITIVITY ANALYSIS

10.1. 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 20.

Table 20: Overview of Sensitivity Analyses

Scenario	Description
Catchment Lag Factor, "C"	The catchment lag factor value was increased and decreased by 20%
Rainfall Losses	The rainfall initial and continuing losses were increased and decreased by 10 mm and 1 mm/h, respectively.
Mannings "n"	The hydraulic roughness values were increased and decreased by 20%
Pit Blockages	Sensitivity to blockage of pits was assessed for 0% and 50% blockage.
Culvert and Bridge Blockage	Sensitivity to blockage of culverts and bridges on open channel sections was assessed for: • 0% blockage for all bridges and culverts; • 10% increased blockage for all bridges and 50% blockage for culverts; and • 20% increased blockage for all bridges and 75% blockage for culverts.
Energy Losses	The energy loss (K parameter) at bridges was increased by 0.2
Initial Water Level	The initial water level in the lagoon was increased and decreased by 0.2 m.

The sensitivity scenario results were analysed for the 1% AEP event and for the 5% AEP for some scenarios. A summary of peak flood level and peak flow differences at various locations are provided in:

- Table 21 for variations in the catchment lag factor (C);
- Table 22 for variations in rainfall losses;
- Table 23 for variations in roughness;
- Table 25 for variations in structure blockage;
- Table 27 for variations in energy losses; and
- Table 28 for variations in initial water level; and

10.2. Catchment Lag Parameter

The catchment lag parameter was increased and decreased by 20%. The increase in the lag factor results in a slight decrease and delay in peak flows, while a decrease in the lag factor generally slightly increases and speeds up the peak catchment flows. This does not necessarily occur immediately in the vicinity of the Curl Curl Lagoon berm since peak flows at this location are dependent on lagoon opening conditions. Changing the lag parameter changes the timing of the overtopping and flow through the breakout compared to the timing of the catchment runoff.

The results of the catchment lag parameter sensitivity analysis are provided in Table 21. The results indicate peak flows are relatively insensitive to the lag parameter assumption in comparison to other key inputs such as rainfall intensity.

Table 21: Sensitivity of 1% AEP Peak Flow (m^3/s) to the Lag Factor

ID	Location	1% AEP Peak Flow (m^3/s)	Change in 1% AEP Peak Flow (m^3/s)	
			-20% C	+20% C
Q01	Upstream of The Kilns (West)	5.7	0.1	-0.2
Q02	Upstream of The Kilns (East)	2.0	0.1	-0.1
Q03	Upstream Consul Road	9.9	0.4	-0.4
Q04	Consul Road	3.2	0.2	-0.3
Q05	Gulliver Street	9.7	0.2	-0.4
Q06	Downstream Winbourne Road	10.2	0.1	-0.1
Q07	Upstream Harbord Road	24.2	0.2	-0.2
Q08	Downstream Harbord Road	30.3	0.3	-0.2
Q09	Upstream Western Footbridge	33.8	0.1	-0.1
Q10	Upstream Eastern Footbridge	41.9	0.1	-0.1
Q11	Downstream Rock Weir	50.8	0.1	-0.1
Q12	Downstream Griffin Road	73.4	0.0	-0.3
Q13	Curl Curl Lagoon Berm	90.1	6.3	-9.7
Q14	Adams Street	1.6	0.0	0.0
Q15	Bennett Street	2.4	0.0	0.0
Q16	Manuela Place	0.3	0.0	0.0
Q17	Pitt Road	1.4	0.0	0.0
Q18	Abbott Road	5.9	0.1	-0.2
Q19	Upstream Community Centre (Abbott Road)	1.7	0.0	-0.1
Q20	Upstream Reub Hudson Oval (Abbott Road)	1.2	0.1	0.1
Q21	Downstream Northern Beaches Secondary College	0.3	0.0	0.0

10.3. Rainfall Losses

The sensitivity of peak flows to initial loss was investigated by increasing and decreasing the initial loss by 10 mm and continuing loss by 1 mm/hr. It was found that flows were relatively insensitive to these rainfall loss variations. Decreasing the initial loss or continuing loss typically resulted in a slight increase in peak flows, while increasing the initial loss typically resulted in a slight decrease in peak flows. This does not necessarily occur immediately upstream of the berm where outflows are dominated by the timing of the entrance breakout.

The results of the rainfall losses sensitivity analysis are shown in Table 22.

Table 22: Sensitivity of 1% AEP peak flows (m^3/s) to rainfall losses

ID	Location	1% AEP Peak Flow (m^3/s)	Change in 1% AEP Peak Flow (m^3/s)			
			-10 mm IL	+10 mm IL	-1 mm/hr CL	+1 mm/hr CL
Q01	Upstream of The Kilns (West)	5.7	0.1	-0.5	0.0	0.0
Q02	Upstream of The Kilns (East)	2.0	0.1	-0.3	0.0	0.0
Q03	Upstream Consul Road	9.9	0.4	-1.1	0.0	0.0
Q04	Consul Road	3.2	0.3	-1.2	0.0	-0.1
Q05	Gulliver Street	9.7	0.4	-1.7	0.0	-0.1
Q06	Downstream Winbourne Road	10.2	0.2	-0.4	0.0	0.0
Q07	Upstream Harbord Road	24.2	0.6	-1.1	0.0	-0.1
Q08	Downstream Harbord Road	30.3	0.9	-1.6	0.1	-0.1
Q09	Upstream Western Footbridge	33.8	0.8	-1.6	0.1	-0.1
Q10	Upstream Eastern Footbridge	41.9	0.9	-1.8	0.1	-0.1
Q11	Downstream Rock Weir	50.8	0.8	-1.6	0.1	-0.1
Q12	Downstream Griffin Road	73.4	0.0	-2.5	0.1	-0.1
Q13	Curl Curl Lagoon Berm	90.1	8.8	-7.9	1.3	-1.9
Q14	Adams Street	1.6	0.0	-0.1	0.0	0.0
Q15	Bennett Street	2.4	0.0	-0.2	0.0	0.0
Q16	Manuela Place	0.3	0.0	0.0	0.0	0.0
Q17	Pitt Road	1.4	0.0	-0.1	0.0	0.0
Q18	Abbott Road	5.9	0.3	-0.8	0.0	0.0
Q19	Upstream Community Centre (Abbott Road)	1.7	0.0	-0.1	0.0	0.0
Q20	Upstream Reub Hudson Oval (Abbott Road)	1.2	0.1	0.0	0.0	0.0
Q21	Downstream Northern Beaches Secondary College	0.3	0.0	-0.1	0.0	0.0

10.4. Roughness Variations

Overall peak flood level results were shown to be relatively insensitive to +/-20% variations in the roughness parameter. Varying the Mannings 'n' by 20% typically resulted in a peak flood height difference within ± 0.1 m. The greatest changes occurred in the Greendale Creek channel between Harbord Road and the western footbridge. Shallow overland flow areas are relatively insensitive to roughness variations as shown in Table 23.

Table 23: Roughness Sensitivity Analysis – Change in Peak Flood Level (m)

ID	Location	5% AEP Peak Flood Level (mAHD)	Change in 5% AEP Peak Flood Level (m)		1% AEP Peak Flood Level (mAHD)	Change in 1% AEP Peak Flood Level (m)	
			-20% Roughness	+20% Roughness		-20% Roughness	+20% Roughness
H01	McKillop Road	129.14	0.00	0.00	129.28	0.00	0.00
H02	Upstream 44 Consul Road	30.69	0.00	0.00	31.00	-0.02	0.01
H03	Consul Road	30.36	0.06	-0.06	30.68	-0.01	0.00
H04	Gulliver Street	26.39	-0.01	0.01	26.47	-0.02	0.01
H05	West of Brookvale Oval (Pittwater Road)	22.73	-0.01	0.00	22.79	-0.01	0.01
H06	Pittwater Road	19.16	0.00	0.00	19.26	-0.01	0.01
H07	Winbourne Road	15.72	-0.01	0.00	15.76	0.00	0.01
H08	Upstream Chard Road	11.92	0.06	-0.07	12.40	-0.02	-0.01
H09	Ethel Avenue	6.46	-0.04	0.03	6.69	-0.04	0.03
H10	Upstream Harbord Road	6.05	-0.01	0.00	6.33	-0.01	0.01
H11	Harbord Road	5.06	0.01	0.00	5.20	0.00	-0.01
H12	Downstream Harbord Road	4.13	-0.15	0.13	4.38	-0.14	0.12
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	4.03	-0.16	0.13	4.27	-0.14	0.13
H14	Downstream Rock Weir	2.76	0.01	-0.03	2.86	0.03	0.00
H15	Upstream Griffin Road	2.71	-0.01	0.01	2.82	-0.01	0.00
H16	Downstream Griffin Road	2.67	0.01	0.04	2.78	-0.03	0.03
H17	Upstream Berm	2.64	-0.01	-0.04	2.71	0.03	-0.04
H18	Bennett Street	9.43	0.00	0.00	9.47	0.00	0.00
H19	Mitchell Road	10.23	0.00	0.00	10.36	-0.01	0.01
H20	Pitt Road	13.41	-0.01	0.00	13.46	-0.01	0.01
H21	Abbott Road	4.11	-0.01	0.00	4.15	0.00	0.01
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.74	0.00	0.00	3.75	-0.01	0.01
H23	Upstream Reub Hudson Oval (Abbott Road)	10.45	0.00	0.00	10.59	0.00	0.00
H24	Downstream Northern Beaches Secondary College	10.45	0.00	0.00	10.60	0.00	0.00
H25	Manuela Place	5.33	-0.01	0.01	5.35	0.00	0.01
H26	Upstream Western Footbridge	3.61	-0.13	0.10	3.83	-0.11	0.10
H27	Downstream Western Footbridge	3.60	-0.12	0.10	3.80	-0.10	0.10
H28	Upstream Eastern Footbridge	2.80	0.03	0.00	2.93	0.03	0.02
H29	Downstream Eastern Footbridge	2.77	0.02	-0.01	2.86	0.04	0.02

10.5. Blockage Variations

The adopted pit blockage was 25%. A sensitivity analysis was undertaken to see the effect of 0% blockage and 50% blockage. The change in peak flood levels is shown in Table 24. The results indicate that peak flood levels are relatively insensitive to pit blockage assumptions.

Table 24: Pit Blockage Sensitivity Analysis – Change in Peak Flood Level (m)

ID	Location	5% AEP Peak Flood Level (mAHD)	Change in 5% AEP Peak Flood Level (m)		1% AEP Peak Flood Level (mAHD)	Change in 1% AEP Peak Flood Level (m)	
			0% Blockage	50% Blockage		0% Blockage	50% Blockage
H01	McKillop Road	129.14	0.00	0.00	129.28	0.00	0.00
H02	Upstream 44 Consul Road	30.69	0.00	0.00	31.00	0.00	0.01
H03	Consul Road	30.36	-0.18	0.15	30.68	-0.03	0.03
H04	Gulliver Street	26.39	0.00	0.00	26.47	-0.01	0.01
H05	West of Brookvale Oval (Pittwater Road)	22.73	0.00	-0.01	22.79	0.00	0.00
H06	Pittwater Road	19.16	0.00	0.01	19.26	0.00	0.00
H07	Winbourne Road	15.72	0.00	0.00	15.76	0.00	0.00
H08	Upstream Chard Road	11.92	0.02	-0.03	12.40	-0.02	0.00
H09	Ethel Avenue	6.46	0.02	-0.03	6.69	0.02	-0.03
H10	Upstream Harbord Road	6.05	0.02	-0.04	6.33	0.02	-0.03
H11	Harbord Road	5.06	0.00	0.00	5.20	0.01	-0.02
H12	Downstream Harbord Road	4.13	0.01	-0.03	4.38	0.02	-0.03
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	4.03	0.01	-0.03	4.27	0.02	-0.03
H14	Downstream Rock Weir	2.76	0.01	-0.02	2.86	0.01	-0.02
H15	Upstream Griffin Road	2.71	0.00	0.01	2.82	0.00	0.00
H16	Downstream Griffin Road	2.67	-0.01	0.02	2.78	-0.01	0.02
H17	Upstream Berm	2.64	0.01	-0.02	2.71	0.01	-0.02
H18	Bennett Street	9.43	0.00	0.00	9.47	0.00	0.00
H19	Mitchell Road	10.23	-0.02	0.04	10.36	-0.01	0.02
H20	Pitt Road	13.41	-0.01	0.00	13.46	-0.01	0.00
H21	Abbott Road	4.11	0.00	0.01	4.15	0.00	0.01
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.74	0.00	0.00	3.75	0.00	0.00
H23	Upstream Reub Hudson Oval (Abbott Road)	10.45	-0.01	0.02	10.59	-0.01	0.01
H24	Downstream Northern Beaches Secondary College	10.45	-0.01	0.02	10.60	0.00	0.01
H25	Manuela Place	5.33	0.00	0.00	5.35	0.00	0.00
H26	Upstream Western Footbridge	3.61	0.01	-0.03	3.83	0.02	-0.03
H27	Downstream Western Footbridge	3.60	0.01	-0.02	3.80	0.01	-0.03
H28	Upstream Eastern Footbridge	2.80	0.01	-0.02	2.93	0.00	0.00
H29	Downstream Eastern Footbridge	2.77	0.01	-0.02	2.86	0.01	0.00

Culvert and bridge blockages typically have a significant impact on peak flood levels upstream and downstream of hydraulic structures. The greatest impact on peak flood levels occurs around Consul Road, Chard Road and Harbord Road, with peak flood level increases of up to 0.7 m. These impacts are relatively localised to the immediate vicinity of the structures that are potentially subject to blockage. The culvert and bridge blockage sensitivity results are shown in Table 25 (5% AEP) and Table 26 (1% AEP).

Table 25: Culvert / Bridge Blockage Sensitivity – 5% AEP Change in Peak Flood Level (m)

ID	Location	5% AEP Peak Flood Level (mAHD)	Change in 5% AEP Peak Flood Level (m)		
			0% Blockage	+10% Bridge 50% Culvert Blockage	+20% Bridge 75% Culvert Blockage
H01	McKillop Road	129.14	0.00	0.00	0.00
H02	Upstream 44 Consul Road	30.69	-0.12	0.24	0.37
H03	Consul Road	30.36	-0.18	0.24	0.35
H04	Gulliver Street	26.39	0.00	-0.01	-0.01
H05	West of Brookvale Oval (Pittwater Road)	22.73	0.00	-0.02	-0.01
H06	Pittwater Road	19.16	0.00	0.00	0.00
H07	Winbourne Road	15.72	0.00	-0.01	-0.01
H08	Upstream Chard Road	11.92	-0.16	0.30	0.71
H09	Ethel Avenue	6.46	-0.05	0.10	0.24
H10	Upstream Harbord Road	6.05	-0.42	0.30	0.52
H11	Harbord Road	5.06	0.00	0.12	0.24
H12	Downstream Harbord Road	4.13	0.02	-0.02	-0.03
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	4.03	0.02	-0.02	-0.03
H14	Downstream Rock Weir	2.76	0.00	-0.01	-0.03
H15	Upstream Griffin Road	2.71	0.00	0.00	0.01
H16	Downstream Griffin Road	2.67	0.00	0.00	0.03
H17	Upstream Berm	2.64	0.00	-0.01	-0.03
H18	Bennett Street	9.43	0.00	0.00	0.00
H19	Mitchell Road	10.23	0.00	0.01	0.01
H20	Pitt Road	13.41	0.00	0.00	0.00
H21	Abbott Road	4.11	0.00	0.00	0.00
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.74	0.00	0.00	0.00
H23	Upstream Reub Hudson Oval (Abbott Road)	10.45	0.00	0.00	0.00
H24	Downstream Northern Beaches Secondary College	10.45	0.00	0.00	0.00
H25	Manuela Place	5.33	0.00	0.00	0.00
H26	Upstream Western Footbridge	3.61	0.02	-0.02	-0.03
H27	Downstream Western Footbridge	3.60	0.02	-0.02	-0.03
H28	Upstream Eastern Footbridge	2.80	0.00	-0.01	-0.01
H29	Downstream Eastern Footbridge	2.77	0.00	-0.01	-0.03

Table 26: Culvert / Bridge Blockage Sensitivity – 1% AEP Change in Peak Flood Level (m)

ID	Location	1% AEP Peak Flood Level (mAHD)	Change in 1% AEP Peak Flood Level (m)		
			0% Blockage	+10% Bridge 50% Culvert Blockage	+20% Bridge 75% Culvert Blockage
H01	McKillop Road	129.28	0.00	0.00	0.00
H02	Upstream 44 Consul Road	31.00	-0.04	0.03	0.16
H03	Consul Road	30.68	-0.02	0.06	0.12
H04	Gulliver Street	26.47	0.00	0.00	0.00
H05	West of Brookvale Oval (Pittwater Road)	22.79	0.00	0.00	0.00
H06	Pittwater Road	19.26	0.00	0.00	0.00
H07	Winbourne Road	15.76	0.00	0.00	0.00
H08	Upstream Chard Road	12.40	-0.07	0.13	0.42
H09	Ethel Avenue	6.69	-0.06	0.08	0.18
H10	Upstream Harbord Road	6.33	-0.23	0.20	0.37
H11	Harbord Road	5.20	-0.08	0.09	0.18
H12	Downstream Harbord Road	4.38	0.01	-0.01	-0.01
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	4.27	0.01	-0.01	-0.01
H14	Downstream Rock Weir	2.86	0.00	-0.01	-0.01
H15	Upstream Griffin Road	2.82	0.00	0.00	0.00
H16	Downstream Griffin Road	2.78	0.00	0.01	0.02
H17	Upstream Berm	2.71	0.00	-0.01	-0.03
H18	Bennett Street	9.47	0.00	0.00	0.00
H19	Mitchell Road	10.36	0.00	0.00	0.00
H20	Pitt Road	13.46	0.00	0.00	0.00
H21	Abbott Road	4.15	0.00	0.00	0.00
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.75	0.00	0.00	0.00
H23	Upstream Reub Hudson Oval (Abbott Road)	10.59	0.00	0.00	0.00
H24	Downstream Northern Beaches Secondary College	10.60	0.00	0.00	0.00
H25	Manuela Place	5.35	0.00	0.00	0.00
H26	Upstream Western Footbridge	3.83	0.02	-0.01	0.00
H27	Downstream Western Footbridge	3.80	0.02	-0.01	-0.01
H28	Upstream Eastern Footbridge	2.93	0.01	0.01	0.02
H29	Downstream Eastern Footbridge	2.86	0.00	0.00	0.00

10.6. Energy Loss Variations

The results of the energy losses sensitivity analysis indicate that increasing the energy losses due to piers at hydraulic structures typically results in peak flood level increases of no more than 0.01 m upstream and downstream of the structure. Results are insensitive to these assumptions as shown in Table 27.

Table 27: Bridge Energy Loss Sensitivity Analysis – Change in Peak Flood Level (m)

ID	Location	5% AEP Peak Level (mAHD)	Change in 5% AEP Peak Flood Level (m) Increased Energy Losses	1% AEP Peak Level (mAHD)	Change in 1% AEP Peak Flood Level (m) Increased Energy Losses
H01	McKillop Road	129.14	0.00	129.28	0.00
H02	Upstream 44 Consul Road	30.69	0.00	31.00	0.00
H03	Consul Road	30.36	0.00	30.68	0.00
H04	Gulliver Street	26.39	0.00	26.47	0.00
H05	West of Brookvale Oval (Pittwater Road)	22.73	0.00	22.79	0.00
H06	Pittwater Road	19.16	0.00	19.26	0.00
H07	Winbourne Road	15.72	0.00	15.76	0.00
H08	Upstream Chard Road	11.92	0.00	12.40	0.00
H09	Ethel Avenue	6.46	0.00	6.69	0.00
H10	Upstream Harbord Road	6.05	0.00	6.33	0.00
H11	Harbord Road	5.06	0.00	5.20	0.00
H12	Downstream Harbord Road	4.13	0.00	4.38	0.00
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	4.03	0.00	4.27	0.00
H14	Downstream Rock Weir	2.76	0.00	2.86	0.00
H15	Upstream Griffin Road	2.71	0.00	2.82	0.00
H16	Downstream Griffin Road	2.67	0.00	2.78	0.00
H17	Upstream Berm	2.64	0.00	2.71	0.00
H18	Bennett Street	9.43	0.00	9.47	0.00
H19	Mitchell Road	10.23	0.00	10.36	0.00
H20	Pitt Road	13.41	0.00	13.46	0.00
H21	Abbott Road	4.11	0.00	4.15	0.00
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.74	0.00	3.75	0.00
H23	Upstream Reub Hudson Oval (Abbott Road)	10.45	0.00	10.59	0.00
H24	Downstream Northern Beaches Secondary College	10.45	0.00	10.60	0.00
H25	Manuela Place	5.33	0.00	5.35	0.00
H26	Upstream Western Footbridge	3.61	0.00	3.83	0.00
H27	Downstream Western Footbridge	3.60	0.00	3.80	0.00
H28	Upstream Eastern Footbridge	2.80	0.01	2.93	0.01
H29	Downstream Eastern Footbridge	2.77	0.00	2.86	0.00
H01	McKillop Road	129.14	0.00	129.28	0.00
H02	Upstream 44 Consul Road	30.69	0.00	31.00	0.00
H03	Consul Road	30.36	0.00	30.68	0.00
H04	Gulliver Street	26.39	0.00	26.47	0.00
H05	West of Brookvale Oval (Pittwater Road)	22.73	0.00	22.79	0.00
H06	Pittwater Road	19.16	0.00	19.26	0.00
H07	Winbourne Road	15.72	0.00	15.76	0.00

10.7. Initial Water Level Variations

The results of the sensitivity analysis indicate that reducing the initial water level by 0.2 m typically results in peak water level decreases of less than 0.1 m, while increasing the initial water level typically results in peak water level increases of less than 0.1 m. This is because peak flood levels in the lower catchment are dominated by entrance conditions and inflows rather than initial water level as shown in Table 28.

Table 28: Initial Water Level Sensitivity Analysis – Change in Peak Flood Level (m)

ID	Location	5% AEP Peak Level (mAHD)	Change in 5% AEP Peak Level (m)		1% AEP Peak Level (mAHD)	Change in 1% AEP Peak Level (m)	
			-0.2 m IWL	+0.2 m IWL		-0.2 m IWL	+0.2 m IWL
H01	McKillop Road	129.14	0.00	0.00	129.28	0.00	0.00
H02	Upstream 44 Consul Road	30.69	0.00	0.00	31.00	0.00	0.00
H03	Consul Road	30.36	0.00	0.00	30.68	0.00	0.00
H04	Gulliver Street	26.39	0.00	0.00	26.47	0.00	0.00
H05	West of Brookvale Oval (Pittwater Road)	22.73	0.00	0.00	22.79	0.00	0.00
H06	Pittwater Road	19.16	0.00	0.00	19.26	0.00	0.00
H07	Winbourne Road	15.72	0.00	0.00	15.76	0.00	0.00
H08	Upstream Chard Road	11.92	0.00	0.00	12.40	0.00	0.00
H09	Ethel Avenue	6.46	0.00	0.00	6.69	0.00	0.00
H10	Upstream Harbord Road	6.05	0.00	0.00	6.33	0.00	0.00
H11	Harbord Road	5.06	0.00	0.00	5.20	0.00	0.00
H12	Downstream Harbord Road	4.13	-0.01	0.00	4.38	-0.01	0.01
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	4.03	-0.01	0.00	4.27	-0.01	0.01
H14	Downstream Rock Weir	2.76	-0.04	0.01	2.86	-0.02	0.02
H15	Upstream Griffin Road	2.71	0.00	0.00	2.82	0.00	-0.01
H16	Downstream Griffin Road	2.67	0.04	0.02	2.78	0.01	-0.03
H17	Upstream Berm	2.64	-0.04	-0.05	2.71	-0.01	0.03
H18	Bennett Street	9.43	0.00	0.00	9.47	0.00	0.00
H19	Mitchell Road	10.23	0.00	0.00	10.36	0.00	0.00
H20	Pitt Road	13.41	0.01	0.00	13.46	0.00	0.00
H21	Abbott Road	4.11	0.00	0.00	4.15	0.00	0.00
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.74	0.00	0.00	3.75	0.00	0.00
H23	Upstream Reub Hudson Oval (Abbott Road)	10.45	0.00	0.00	10.59	0.00	0.00
H24	Downstream Northern Beaches Secondary College	10.45	0.00	0.00	10.60	0.00	0.00
H25	Manuela Place	5.33	0.00	0.00	5.35	0.00	0.00
H26	Upstream Western Footbridge	3.61	-0.01	0.00	3.83	-0.01	0.01
H27	Downstream Western Footbridge	3.60	-0.01	0.00	3.80	-0.01	0.00
H28	Upstream Eastern Footbridge	2.80	0.00	0.03	2.93	0.00	-0.01
H29	Downstream Eastern Footbridge	2.77	-0.02	0.02	2.86	-0.01	0.02

10.8. Downstream Boundary Conditions

The downstream ocean boundary was set to mean sea level (0 mAHD). A sensitivity analysis was conducted on this level by raising it to 2.55 mAHD. This level is recommended in Reference 16 for a “Group 4” ICOLL and corresponds to a 1% AEP ocean water level. This tailwater level made no change to the peak water levels (or velocities) within the lagoon, even when coupled with the 1% AEP rainfall event. This is because water levels are controlled by the berm height (set to 2.75 mAHD for the 1% AEP event), which is set above the tailwater level. All the recommended coincident tailwater levels are lower than the corresponding design berm heights adopted and due to the rapid scouring and breakout of the lagoon, the ocean conditions do not influence the water levels in the lagoon.

While the berm height, rather than ocean levels, was found to control peak water levels in the lagoon, an open entrance condition was also simulated to investigate the influence of the ocean. As per the guidelines (Reference 16), an envelope with the joint probability of 1% AEP storm with 5% AEP ocean conditions (tailwater level of 2.35 mAHD), and 5% AEP storm with 1% AEP ocean conditions (tailwater level of 2.55 mAHD) was taken to represent the 1% AEP event considering both catchment flooding and ocean inundation. The envelope of these scenarios resulted in lower peak water levels through the lagoon (by approximately 0.1 m to 0.2 m) and lower velocities (up to 0.6 m/s, when compared to the 1% design envelope velocities). Again, this demonstrates that the berm is the primary control for water levels in Curl Curl Lagoon.

It is recognised that the adopted breakout and scour behaviour of the berm for the design flood events is rapid. The adopted scour formation time of 6 minutes was based on the calibration to the November 2018 event and is quick enough to release water from the lagoon when the berm height is reached such that the water level does not rise significantly above the berm level (determined by the FFA). A sensitivity was conducted on the scour formation time for the 1% AEP event by increasing it from 6 minutes to 60 minutes. This resulted in an increase in peak water level of up to 0.18 m in the lagoon. It is recognised that the scour formation (timing as well as the physical geometry) may affect water levels in the lagoon. The approach adopted in this study, however, is to simulate design flood levels that are consistent with the FFA undertaken and the adopted scour characteristics maintain this.

11. CLIMATE CHANGE IMPACTS

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. In 2009 the NSW government published guideline for incorporating sea level rise benchmarks in coastal flood risk assessments (Reference 20), which provides a consistent set of sea level rise scenarios for undertaking land use planning for 2050 (0.4 m increase over 1990 levels) and 2100 (0.9 m increase over 1990 levels).

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 Bureau of Meteorology is based on historical climate data and does not currently include any allowance for likely increases to convective storm rainfall intensity. Australian Rainfall and Runoff 2019 (Reference 1, 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. For this study, Council requested that a scenario be modelled with a 10% rainfall intensity increase, corresponding to approximately 2 degrees warming.

It has also been suggested that the cyclone belt may move further southwards. The possible impacts of this outcome on design rainfalls cannot be ascertained at this time as there is insufficient information about the mechanisms that determine the movement of cyclones under future climate scenarios.

Projected increases to evaporation are also an important consideration because increased evaporation would lead to generally drier catchment conditions, resulting in lower runoff from rainfall. Mean annual rainfall is projected to decrease, which will also result in generally dryer catchment conditions. This is a consideration for the Greendale Creek catchment where the initial water level in Curl Curl Lagoon is an important determinant on whether flooding will occur in the lower catchment. Under drier conditions, Curl Curl Lagoon will be less full on average when rain occurs, and a larger proportion of the initial rain will be collected in the lagoon. However the sensitivity analysis has indicated that flooding in the lower catchment is relatively insensitive to initial water levels in Curl Curl Lagoon.

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 14). Specifically, it is suggested that rainfall intensity and sea level rise increase scenarios should be considered. The following climate change scenarios were assessed for this study:

- Comparison of the current 0.5% AEP and 0.2% AEP rainfall intensity with the 1% AEP rainfall intensity (per the relevant guideline, Reference 14). These events provide an indication of how 1% AEP flood levels would change if the rainfall intensity increases to the point that it matches either the current 0.5% AEP (a 7.8% increase in intensity) or 0.2% AEP (a 23% increase in intensity).

- Comparison of the 0.4 m and 0.9 m sea level rise benchmarks against current conditions (per the guidance in Reference 20). These scenarios assumed that the typical sand berm height at the lagoon entrance would rise by an amount equivalent to the sea level rise.
- Combined scenarios with both rainfall increase and sea level rise, assuming a 10% increase in rainfall intensity (corresponding to approximately 2 degrees of average surface temperature increases per guidance in Reference 1) and the 0.4 m and 0.9 m sea level rise benchmarks.

The climate change impact results are shown in Table 29, with maps provided in Appendix G.

The results indicate that there would be widespread impact from changes to design rainfalls relative to other design storm assumptions from Section 10. Increases in rainfall would result in an increase in peak flood levels at most of the locations analysed. The largest variations in flood levels are in ponded areas and in the lower catchment with modelled peak flood levels up to 0.2 m higher in the 0.2% AEP event (23% intensity increase relative to the 1% AEP event).

Peak flood levels in the lower catchment are substantially higher in the sea level rise scenarios while flood levels in overland flow areas in the upper catchment are typically unaffected. Peak flood levels just downstream of Griffin Road Bridge are up to 0.4 m and 0.8 m higher for the sea level rise scenarios of 0.4 m and 0.9 m, respectively.

Table 29: Results of Climate Change Impact Analysis – Change in Peak Flood Level (m)

ID	Location	1% AEP Peak Flood Level (mAHD)	0.5% AEP Event Difference (m)	0.2% AEP Event Difference (m)	+0.4 m Sea Level Rise Difference (m)	+0.9 m Sea Level Rise Difference (m)
H01	McKillop Road	129.28	0.04	0.11	0.00	0.00
H02	Upstream 44 Consul Road	31.00	0.03	0.02	0.00	0.00
H03	Consul Road	30.68	0.04	0.09	0.00	0.00
H04	Gulliver Street	26.47	0.03	0.07	0.00	0.00
H05	West of Brookvale Oval (Pittwater Road)	22.79	0.03	0.08	0.00	0.00
H06	Pittwater Road	19.26	0.03	0.07	0.00	0.00
H07	Winbourne Road	15.76	0.01	0.04	0.00	0.00
H08	Upstream Chard Road	12.40	0.09	0.22	0.00	0.00
H09	Ethel Avenue	6.69	0.09	0.22	0.00	0.00
H10	Upstream Harbord Road	6.33	0.09	0.21	0.00	0.00
H11	Harbord Road	5.20	0.06	0.14	0.00	0.00
H12	Downstream Harbord Road	4.38	0.11	0.30	0.00	0.00
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	4.27	0.11	0.29	0.01	0.01
H14	Downstream Rock Weir	2.86	0.06	0.13	0.35	0.65
H15	Upstream Griffin Road	2.82	0.03	0.10	0.36	0.69
H16	Downstream Griffin Road	2.78	0.01	0.08	0.39	0.72
H17	Upstream Berm	2.71	0.06	0.07	0.37	0.79
H18	Bennett Street	9.47	0.01	0.02	0.00	0.00
H19	Mitchell Road	10.36	0.04	0.10	0.00	0.00
H20	Pitt Road	13.46	0.01	0.04	0.00	0.00
H21	Abbott Road	4.15	0.02	0.04	0.00	0.00
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.75	0.01	0.01	0.00	0.00
H23	Upstream Reub Hudson Oval (Abbott Road)	10.59	0.04	0.09	0.00	0.00
H24	Downstream Northern Beaches Secondary College	10.60	0.04	0.09	0.00	0.00
H25	Manuela Place	5.35	0.02	0.04	0.00	0.00
H26	Upstream Western Footbridge	3.83	0.10	0.27	0.05	0.05
H27	Downstream Western Footbridge	3.80	0.09	0.25	0.05	0.05
H28	Upstream Eastern Footbridge	2.93	0.06	0.17	0.36	0.58
H29	Downstream Eastern Footbridge	2.86	0.06	0.15	0.36	0.64

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14. GLOSSARY

14.1. List of Acronyms

ADR	Australian Disaster Resilience Handbook Collection
AEP	Annual Exceedance Probability
ARF	Areal Reduction Factor
ARI	Average Recurrence Interval
ARR	Australian Rainfall and Runoff
BoM	Bureau of Meteorology
DEM	Digital Elevation Model
DPE	Department of Planning and Environment
ELVIS	Elevation Information System
ERP	Emergency Response Planning
FDM	NSW Floodplain Development Manual
FFA	Flood Frequency Analysis
GIS	Geographic Information System
GPT	Gross Pollutant Trap
GSDM	Generalised Short Duration Method (for PMF estimation)
ICOLL	Intermittently Closed and Open Lakes or Lagoons
IFD	Intensity, Frequency and Duration (rainfall information)
LGA	Local Government Area
LiDAR	Light Detection and Ranging (airborne survey method)
LPI	Land and Property Information
m	metres
m^3/s	cubic metres per second
mAHD	metres above Australian Height Datum
MHL	Manly Hydraulics Laboratory
OEH	Office of Environment and Heritage
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SES	State Emergency Services
TUFLOW	Hydraulic Modelling software
WBNM	Watershed Bounded Network Model (Hydrologic modelling software)
1D/2D	1 Dimensional and 2-Dimensional hydraulic modelling

14.2. Terminology of Flood Risk

Australian Rainfall and Runoff (ARR, editors 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.

ARR2019 recommends the use of Annual Exceedance Probability (AEP). Annual Exceedance Probability (AEP) is the probability of an event being equalled or exceeded within a year. AEP may be expressed as either a percentage (%) or 1 in X. Floodplain management typically uses the percentage form of terminology. Therefore a 1% or 1 in 100 AEP event (sometimes referred to as a 100 year ARI), has a 1% chance of being equalled or exceeded in any year. ARI and AEP are often mistaken as being interchangeable for events equal to or more frequent than 10% AEP. The table below describes how they are subtly different.

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

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

14.3. Glossary of Terms

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 diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.

flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the "flood liable land" concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL's are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the "standard flood event" in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.

hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
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 “water level”. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.

Figures

FIGURE 1
STUDY AREA

J:\Jobs\118094\ArcGIS\ArcMaps\Figure01_Study_Area.mxd

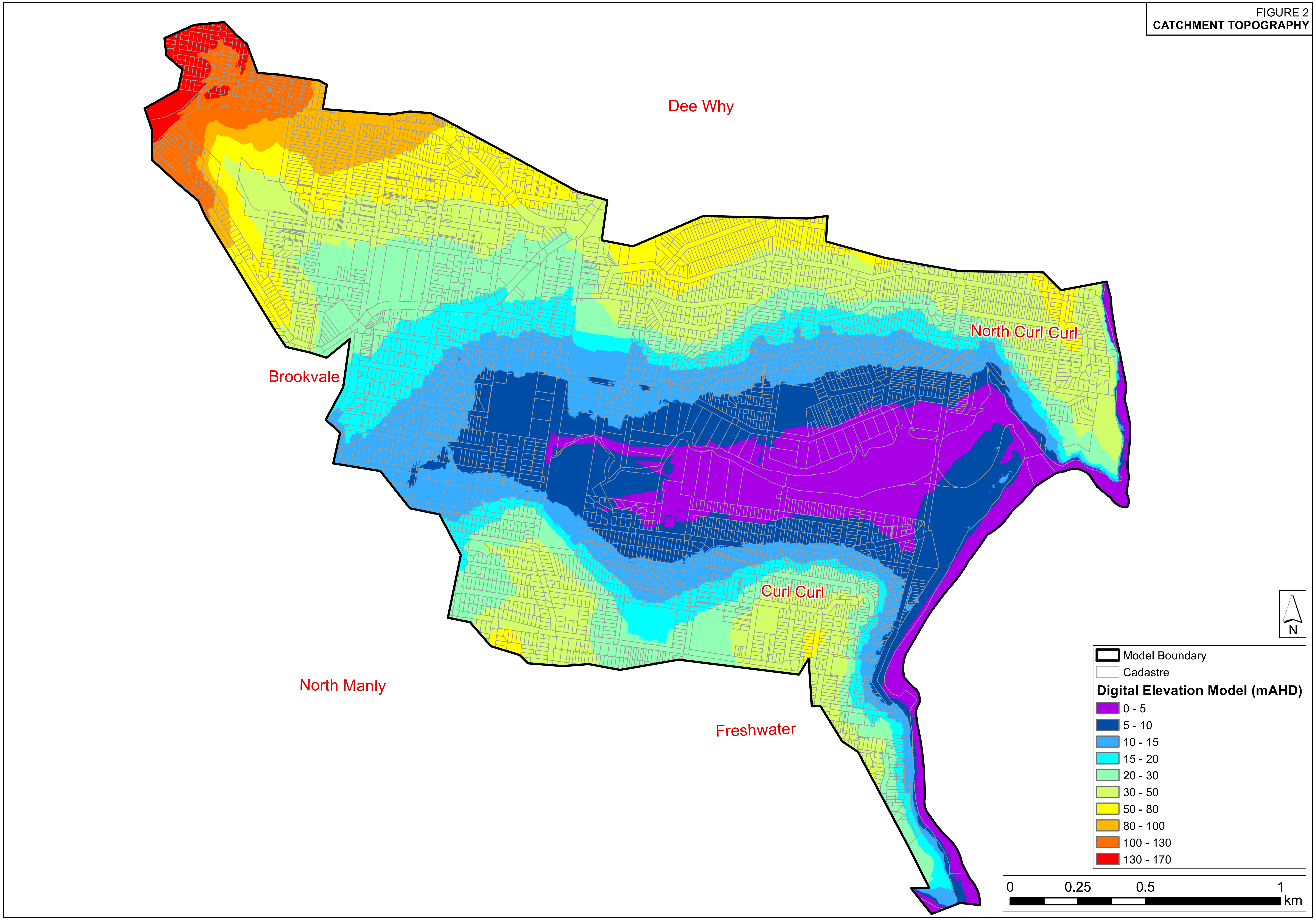
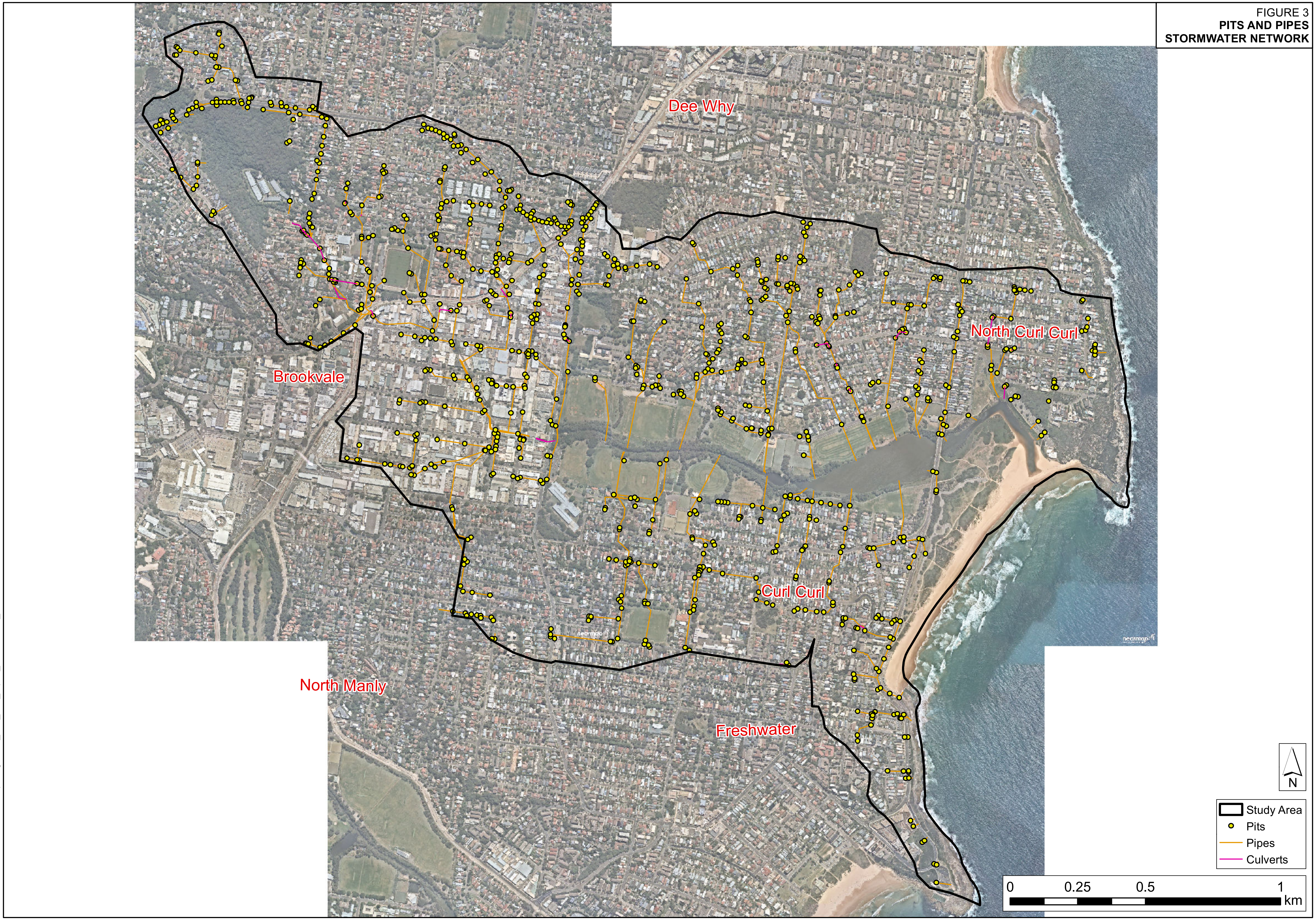


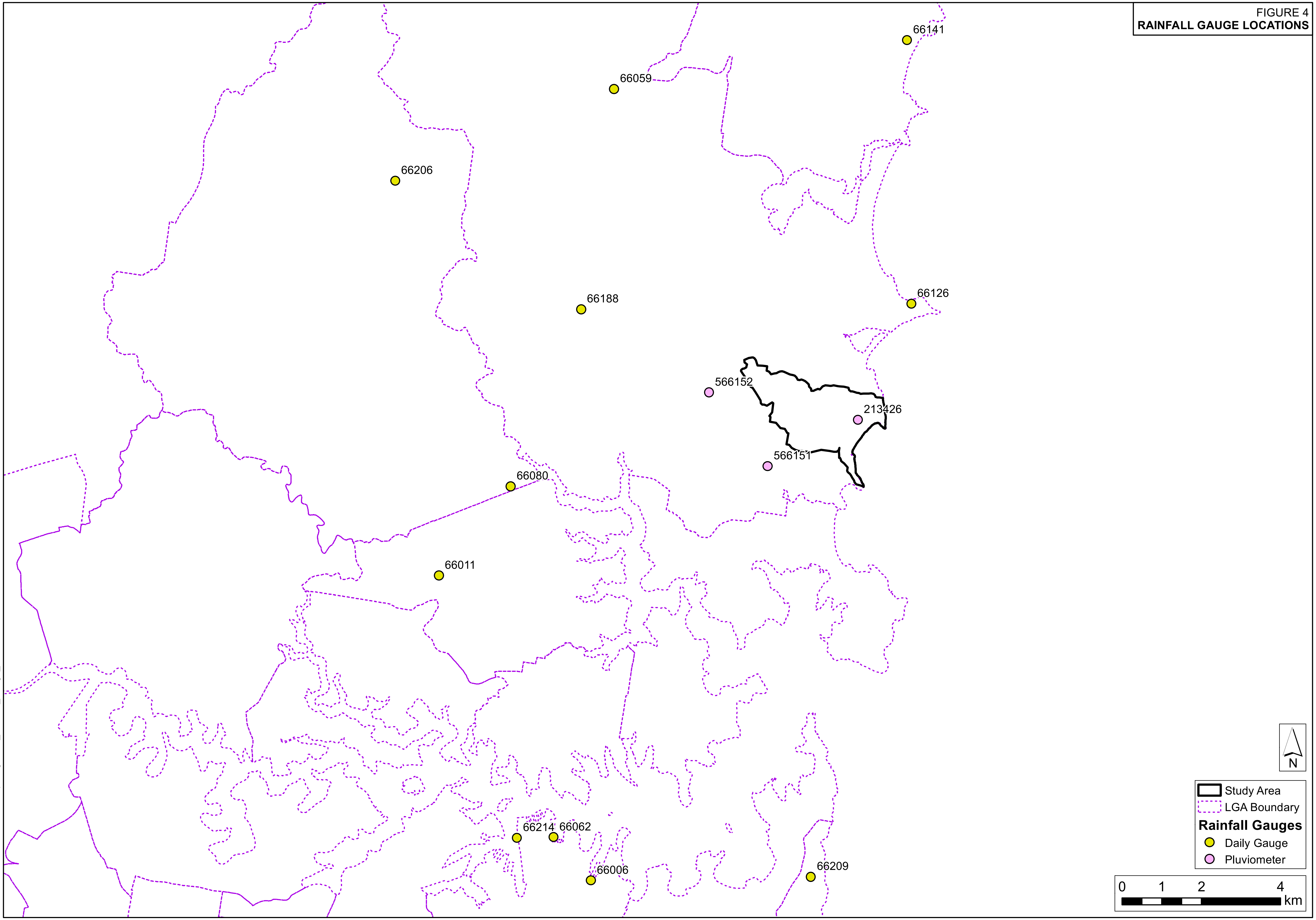
FIGURE 2
CATCHMENT TOPOGRAPHY
Dee Why
North Curl Curl
Brookvale
Curl Curl
North Manly
Freshwater
N
Model Boundary
Cadastre
Digital Elevation Model (mAHD)
0 - 5
5 - 10
10 - 15
15 - 20
20 - 30
30 - 50
50 - 80
80 - 100
100 - 130
130 - 170
0
0.25
0.5
1
km
J:\Jobs\118094\ArcGIS\ArcMaps\Figure02_Catchment_Topography.mxd

FIGURE 3
PITS AND PIPES STORMWATER NETWORK



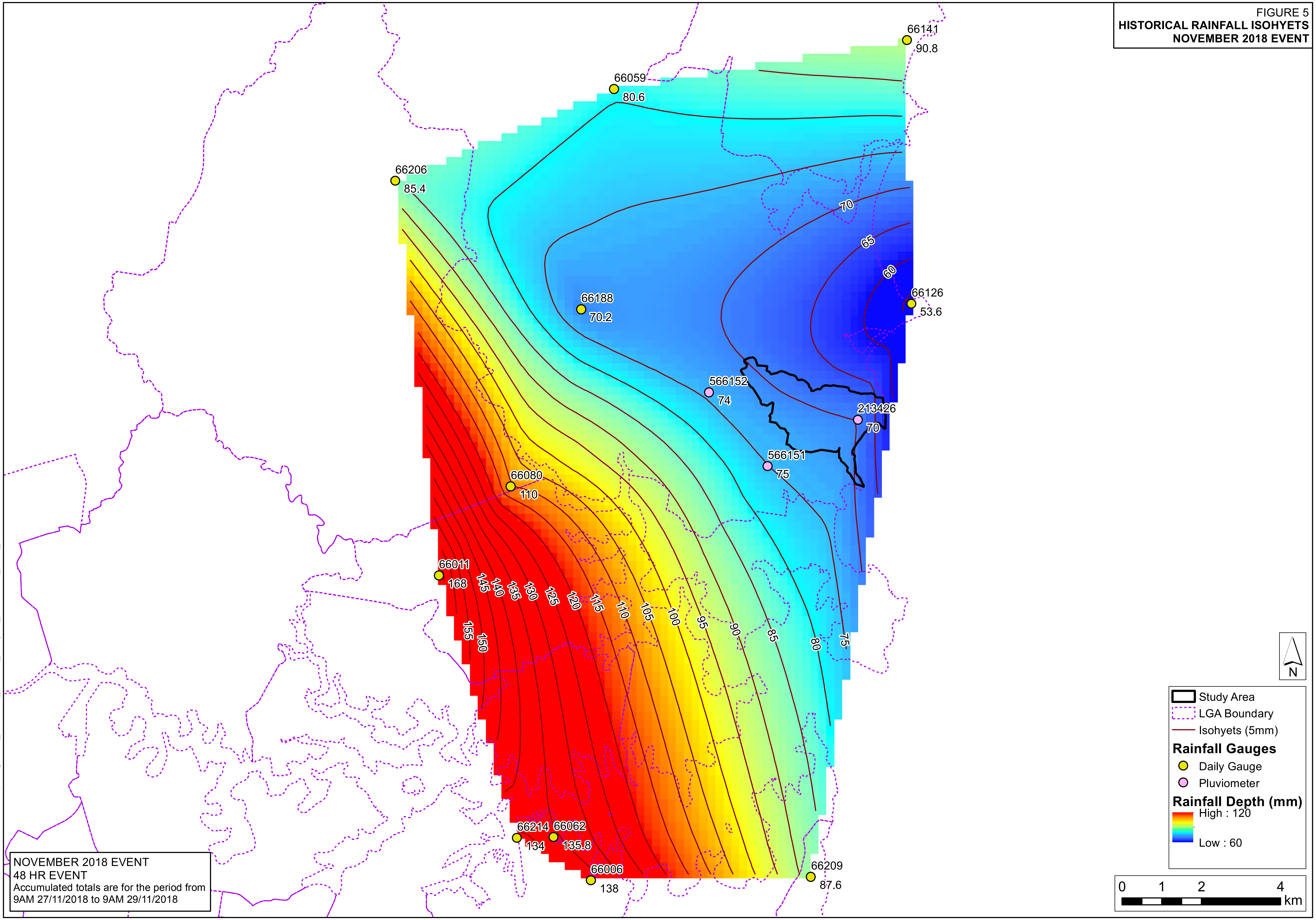
J:\Jobs\118094\ArcGIS\ArcMaps\Figure03_Pits_and_Pipes_Stormwater_Network.mxd

FIGURE 4
RAINFALL GAUGE LOCATIONS



J:\Jobs\118094\ArcGIS\ArcMaps\Figure04_Rainfall_Gauge_Locations.mxd

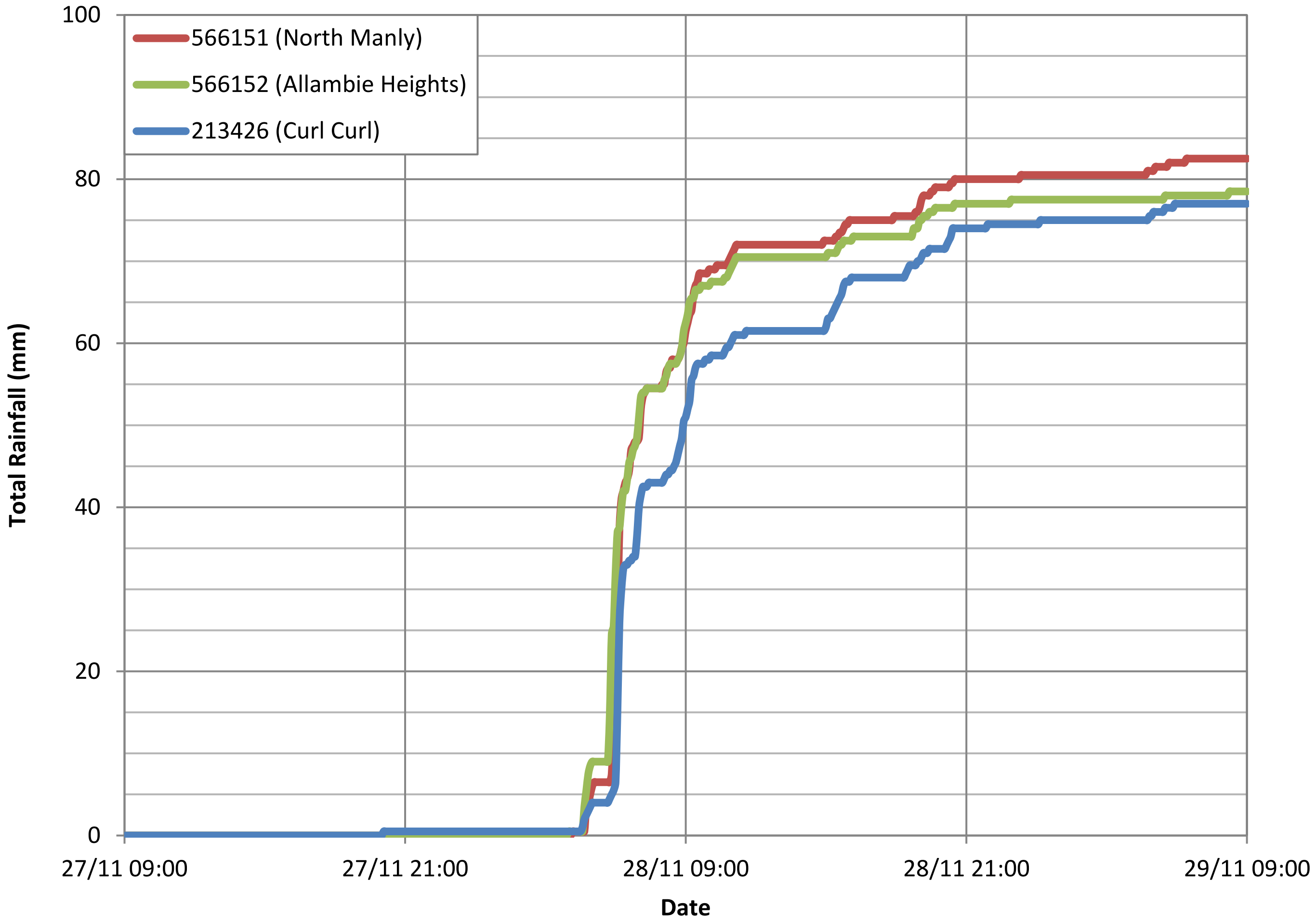
FIGURE 5
HISTORICAL RAINFALL ISOHYETS
NOVEMBER 2018 EVENT

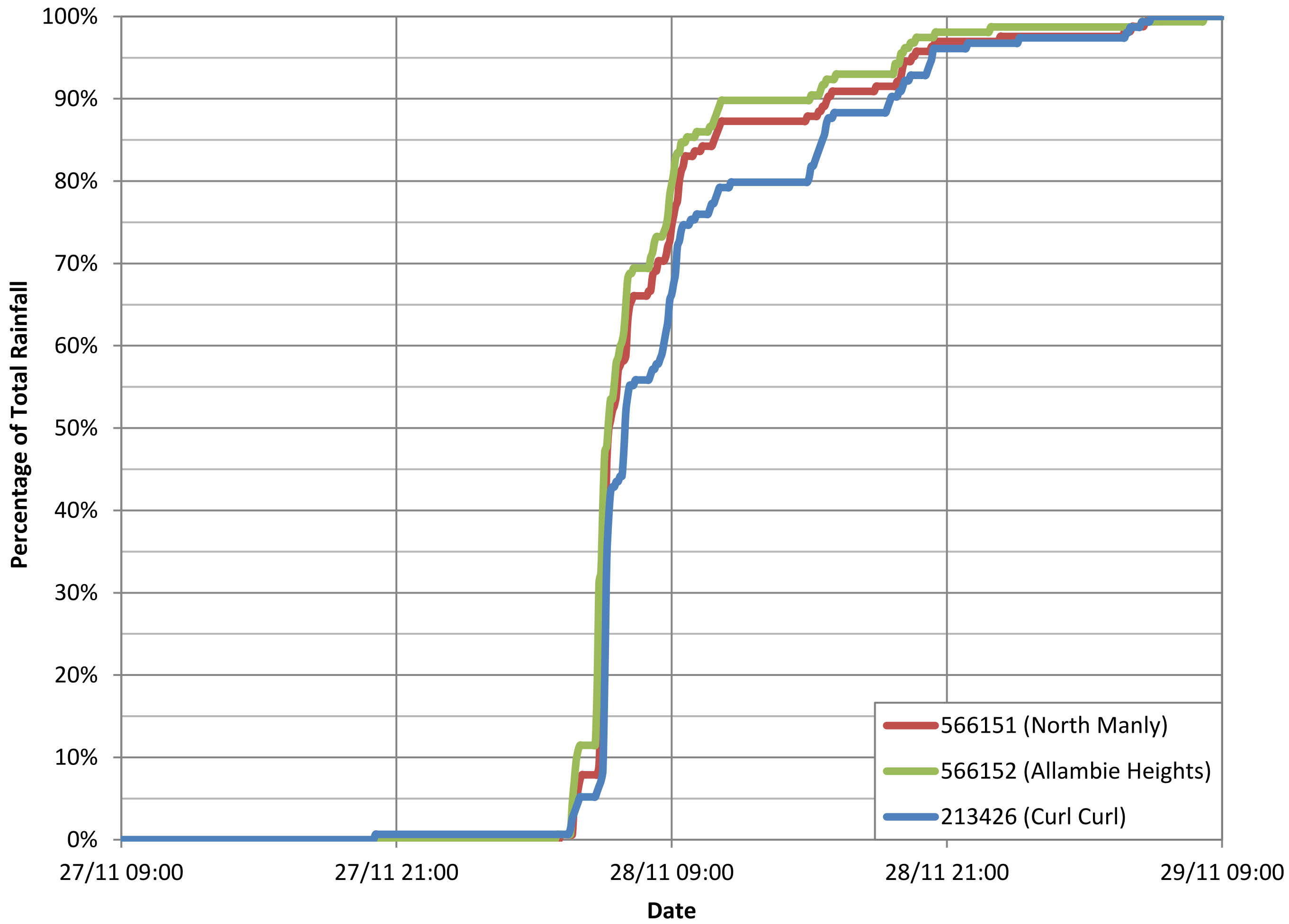


NOVEMBER 2018 EVENT
48 HR EVENT
Accumulated totals are for the period from
9AM 27/11/2018 to 9AM 29/11/2018

J:\Jobs\118094\ArcGIS\ArcMaps\Figure05_Historical_Rainfall_Isohyets_November_2018_Event.mxd

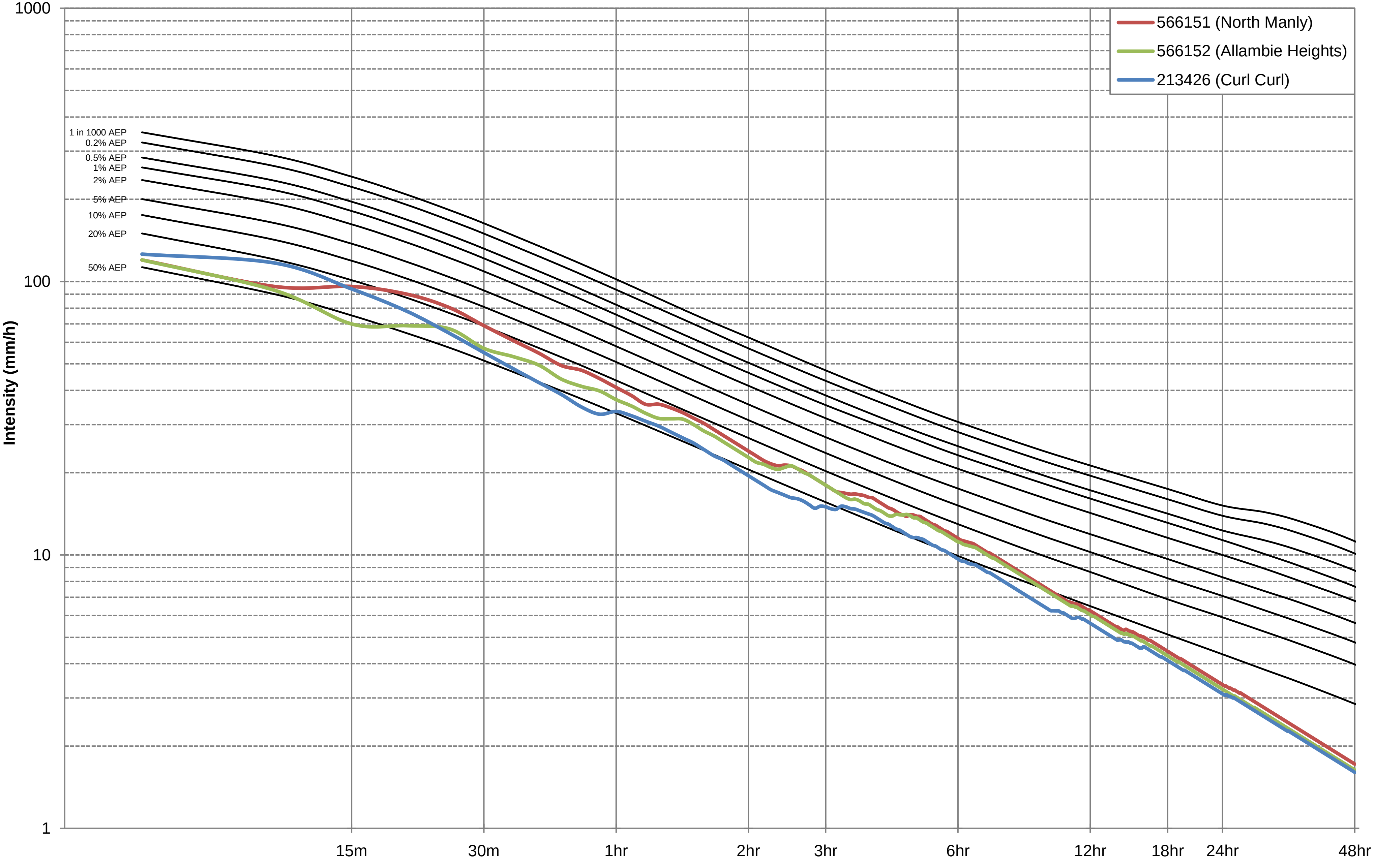
FIGURE 6
CUMULATIVE RAINFALL DATA
NOVEMBER 2018 EVENT





J:\Jobs\118094\Hydrology\Rainfall\Cumulative_Rainfall_Plots\Historical_Burst_IFD_Comparison_2018_2022_v1.4.xlsm

FIGURE 7
BURST INTENSITIES AND FREQUENCIES
NOVEMBER 2018 FLOOD EVENT

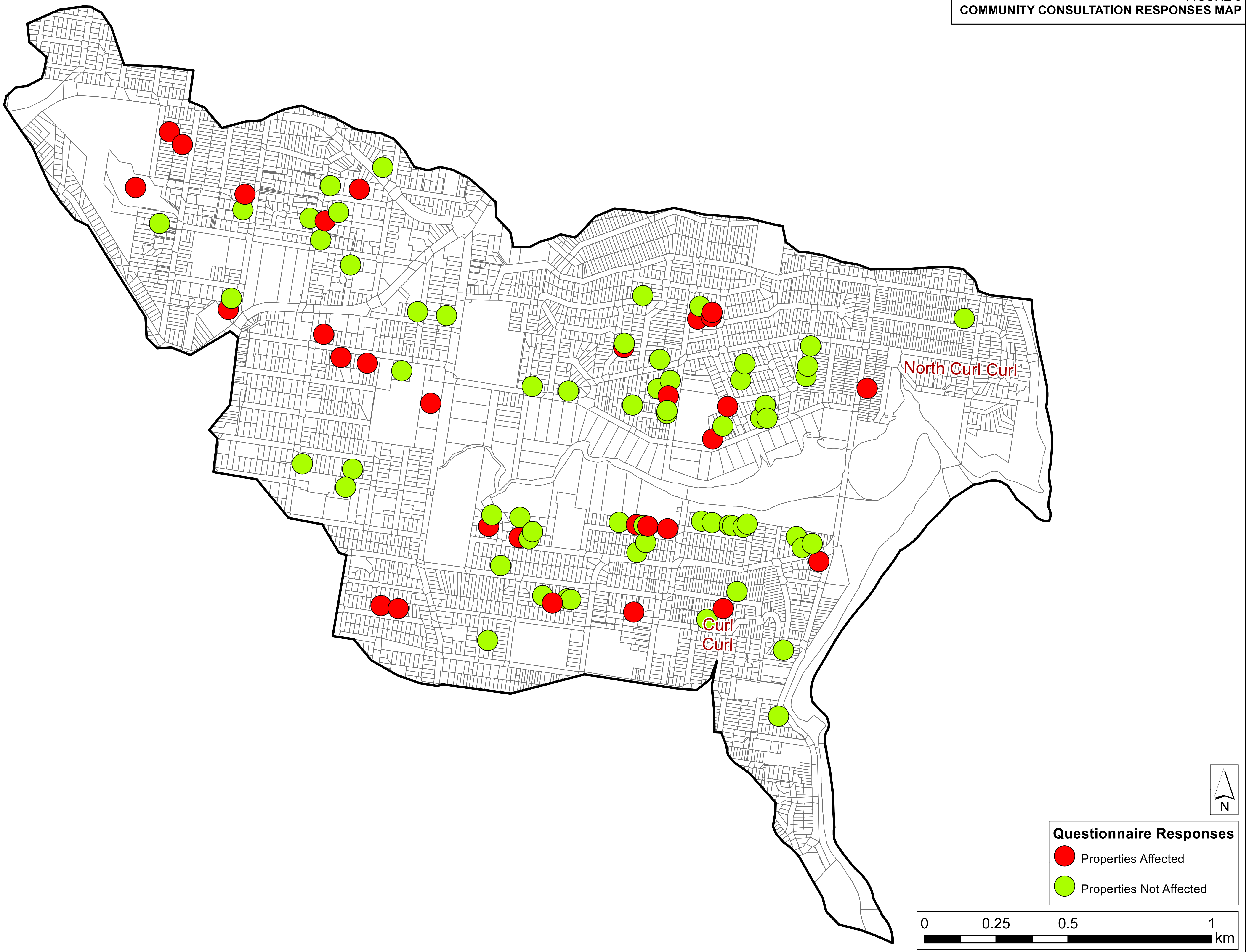


Intensity (mm/h)

Burst Duration

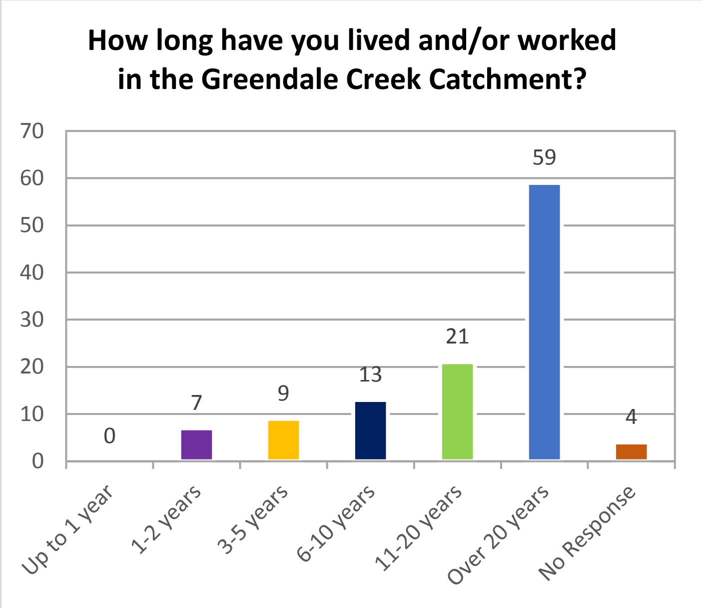
J:\Jobs\118094\Hydrology\Rainfall\Cumulative_Rainfall_Plots\Historical_Burst_IFD_Comparison_2018_2022_v1.4.xlsm

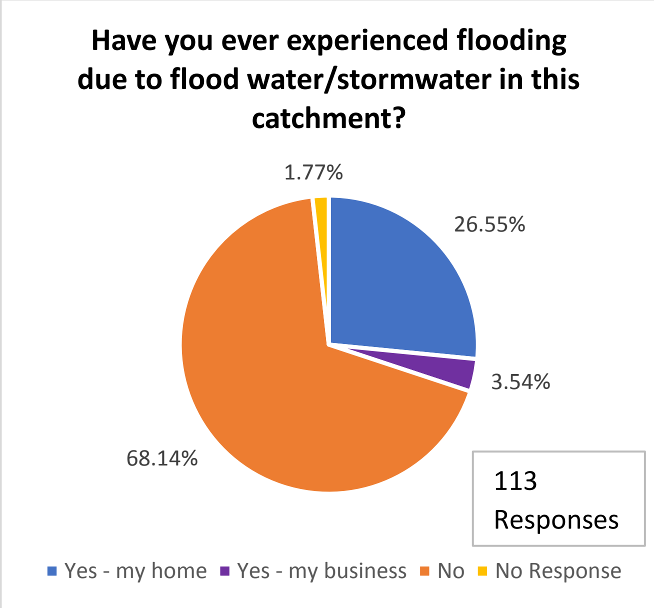
FIGURE 8
COMMUNITY CONSULTATION RESPONSES MAP

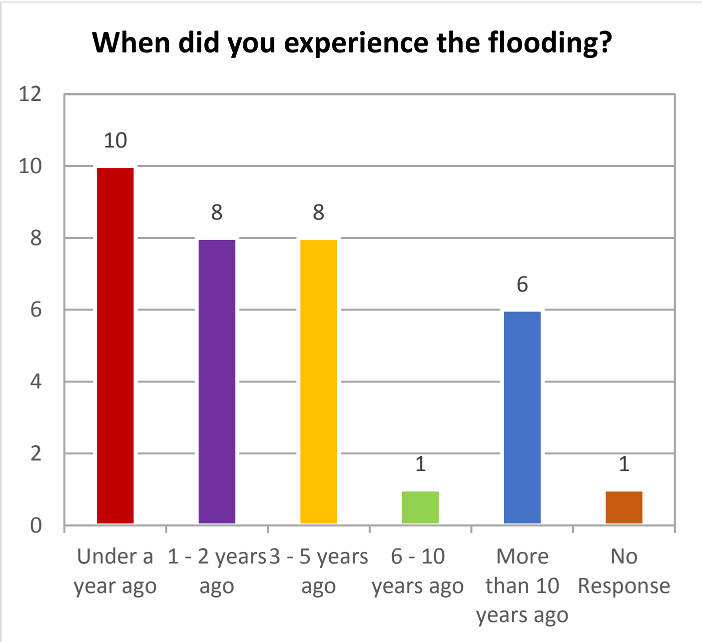


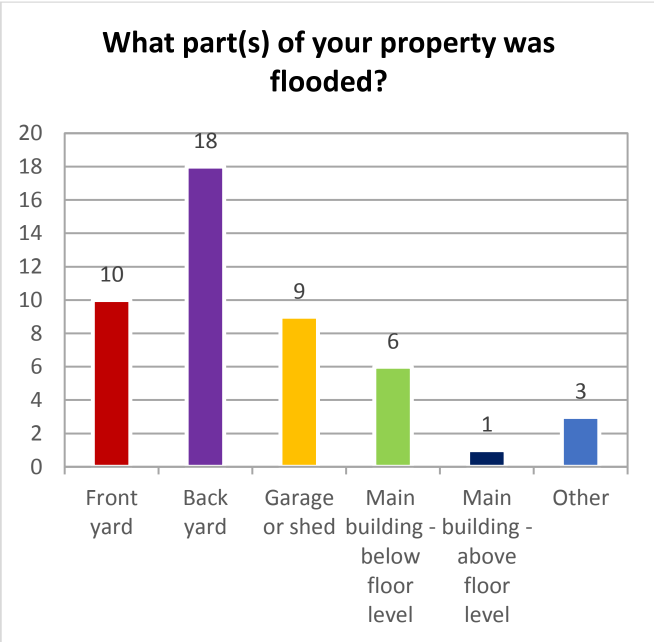
J:\Jobs\118094\ArcGIS\ArcMaps\Figure08_Community_Consultation_Responses_Map.mxd

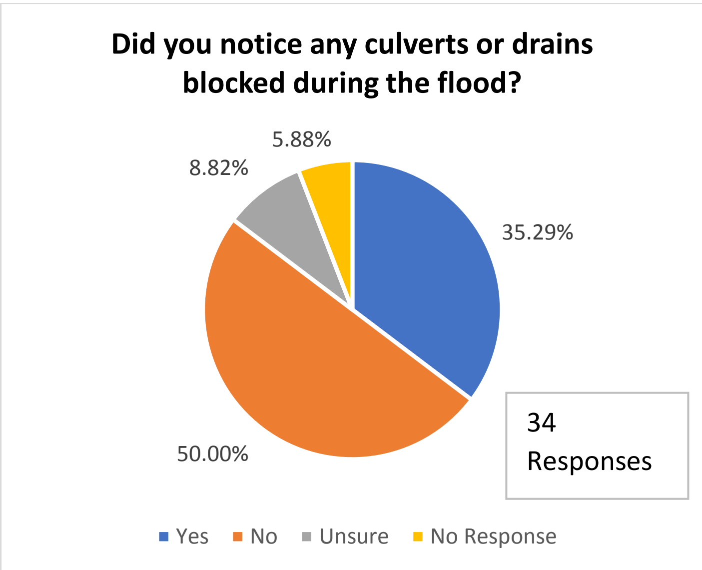
FIGURE 9a
COMMUNITY CONSULTATION RESULTS











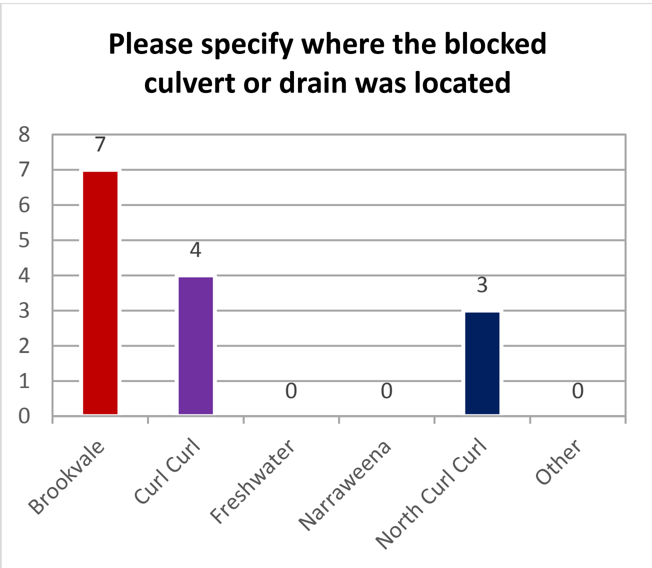
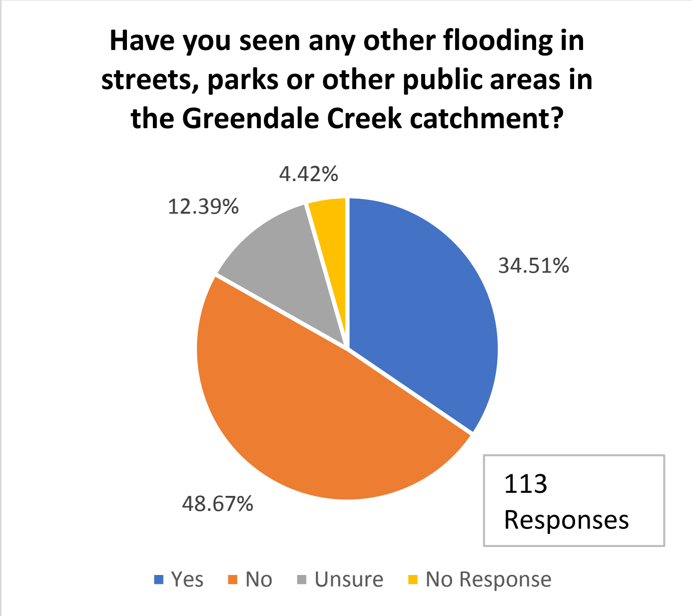
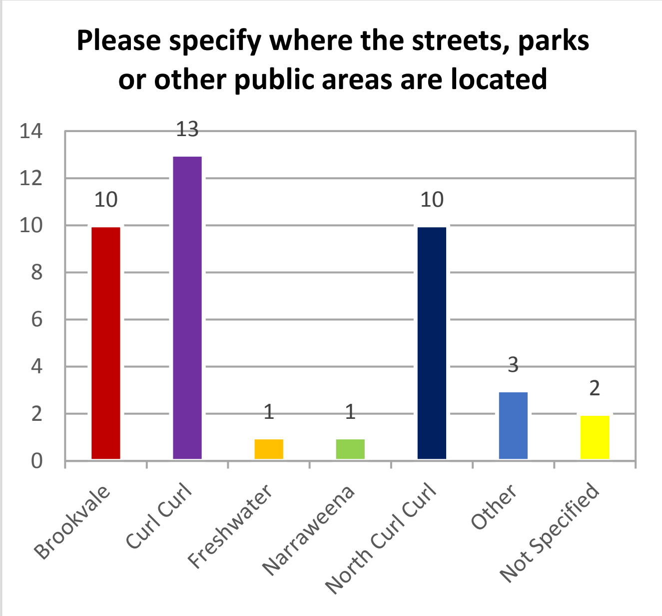
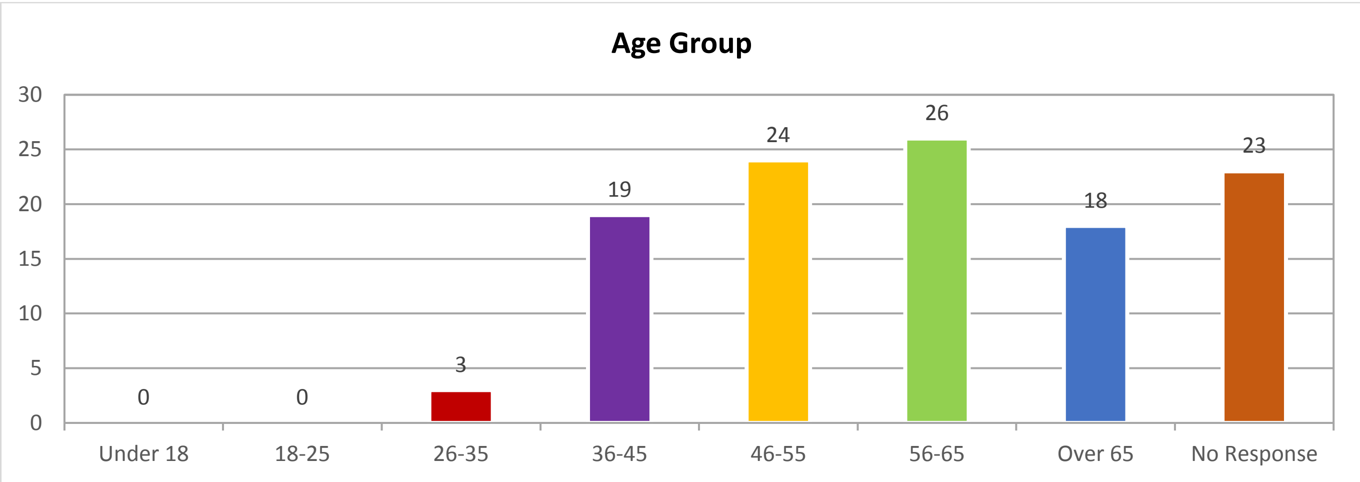


FIGURE 9b
COMMUNITY CONSULTATION RESULTS







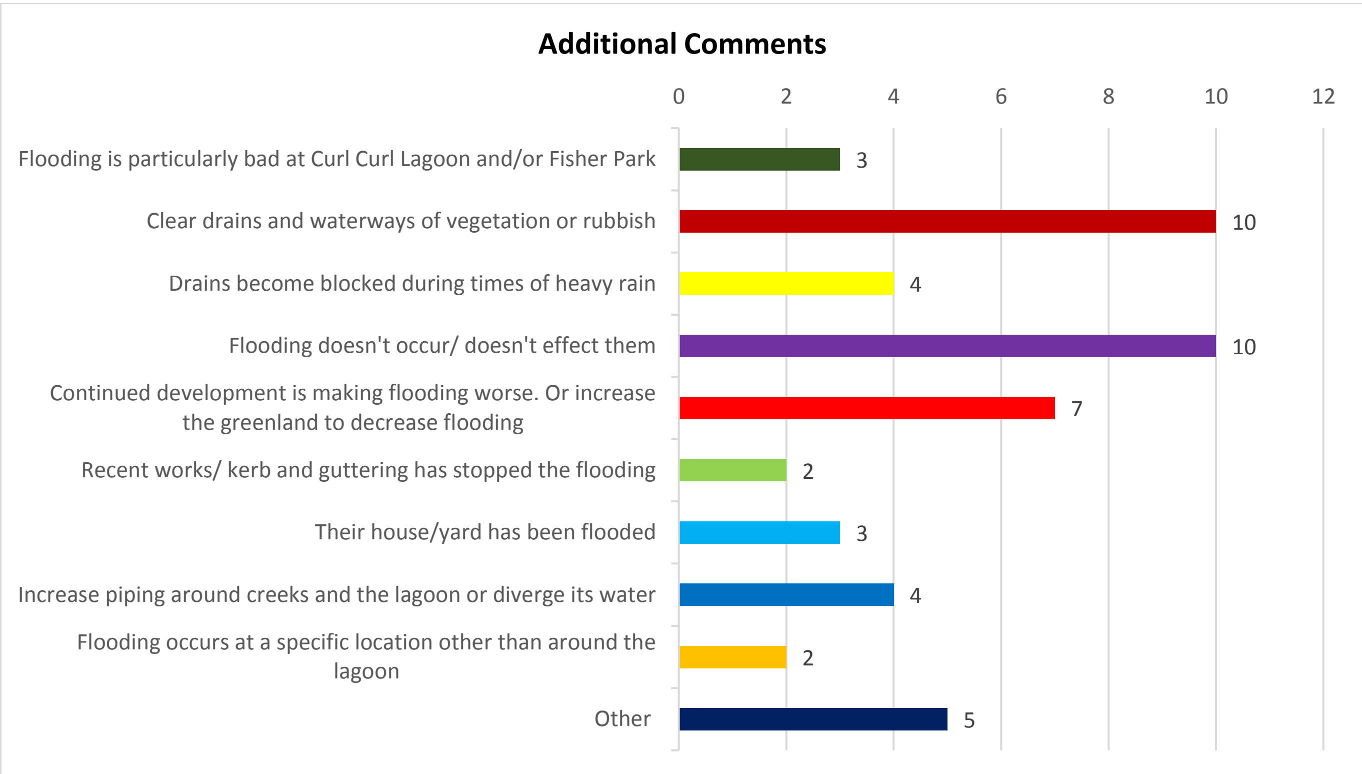


FIGURE 10
WBNM HYDROLOGIC MODEL LAYOUT

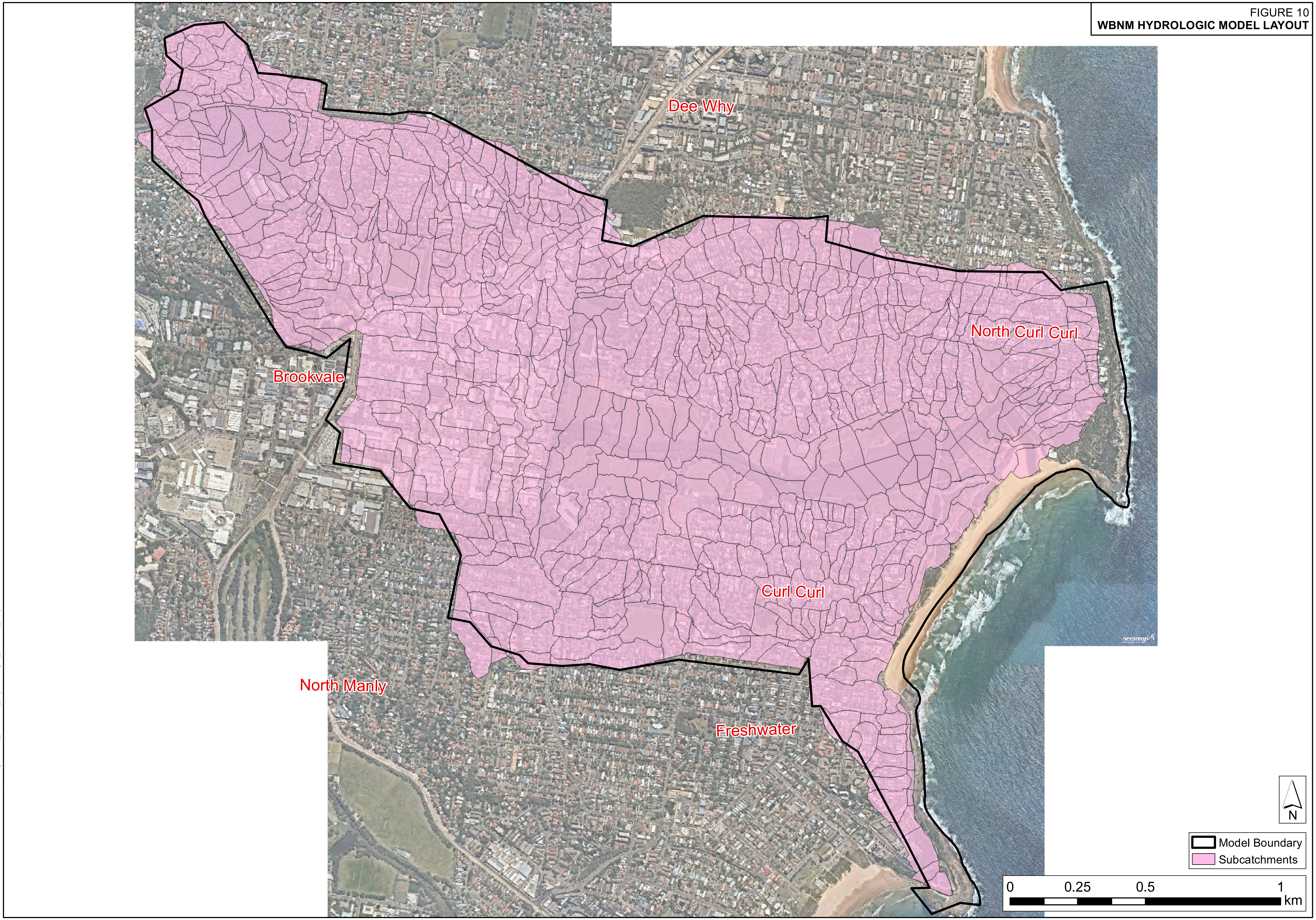


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FIGURE 11
TUFLOW MODEL MANNINGS ROUGHNESS

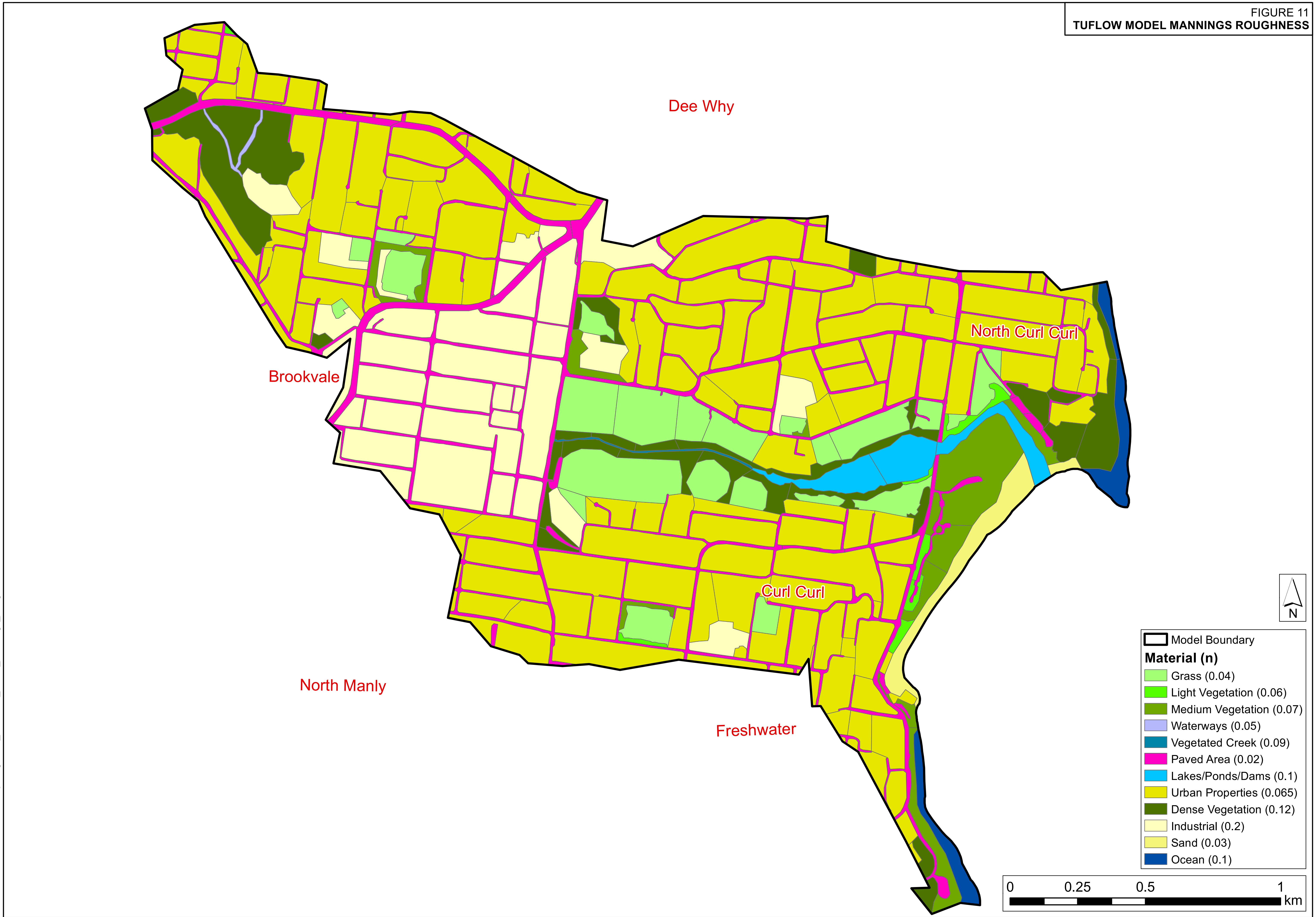

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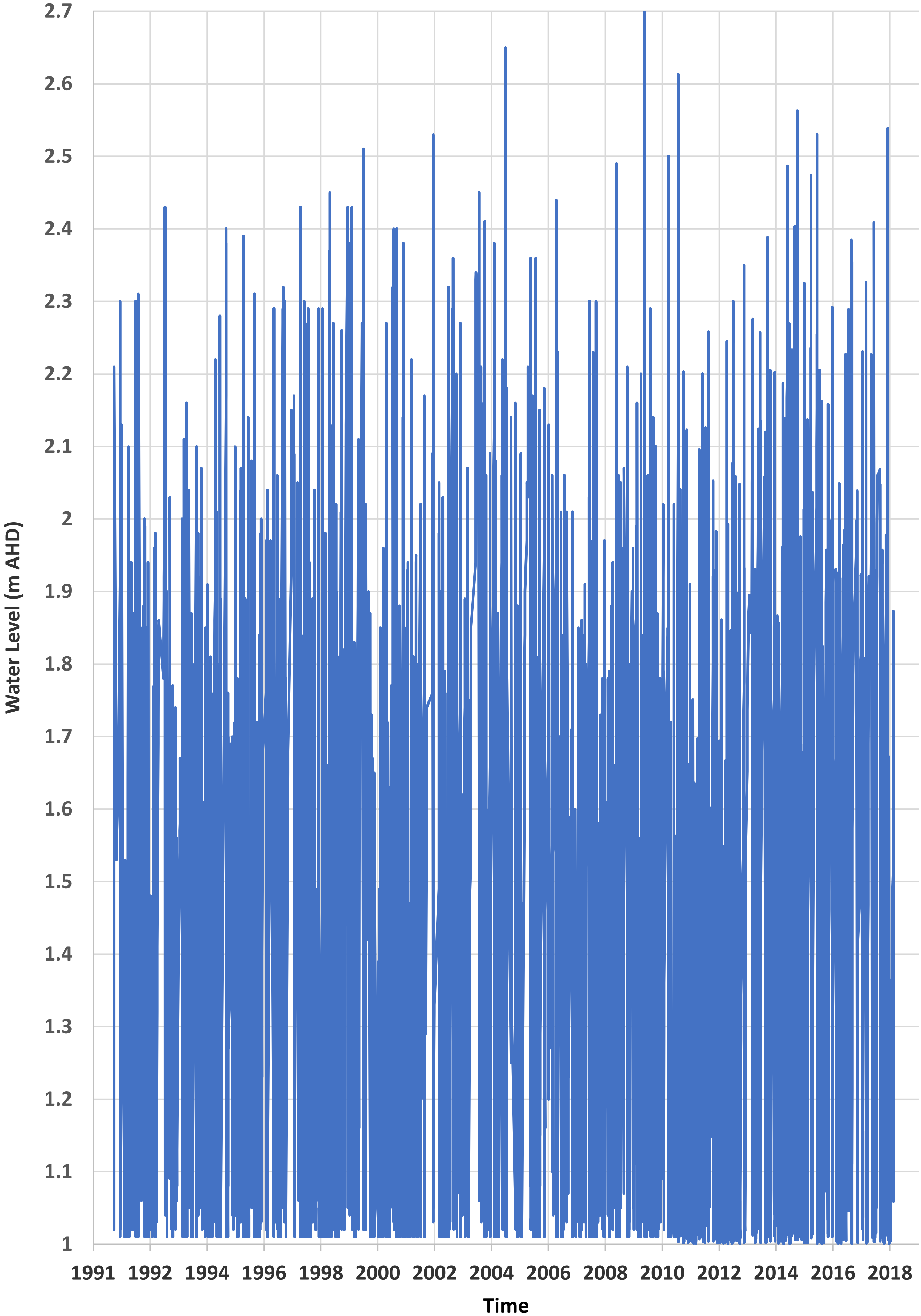
FIGURE 12
TUFLOW MODEL SCHEMATISATION

J:\Jobs\118094\ArcGIS\ArcMaps\Figure12_TUFLOW_Model_Schematisation.mxd

FIGURE 13
STREAM GAUGE RECORDS
CURL CURL GAUGE (213426)
1991 to 2019



J:\Jobs\118094\PDFs\Data_for_Figures\Figure13_Water_Level_Records_213426.xlsx

FIGURE 14
STREAM GAUGE RECORDS
GREENDALE CREEK BROOKVALE FLOW
GAUGE (213499)

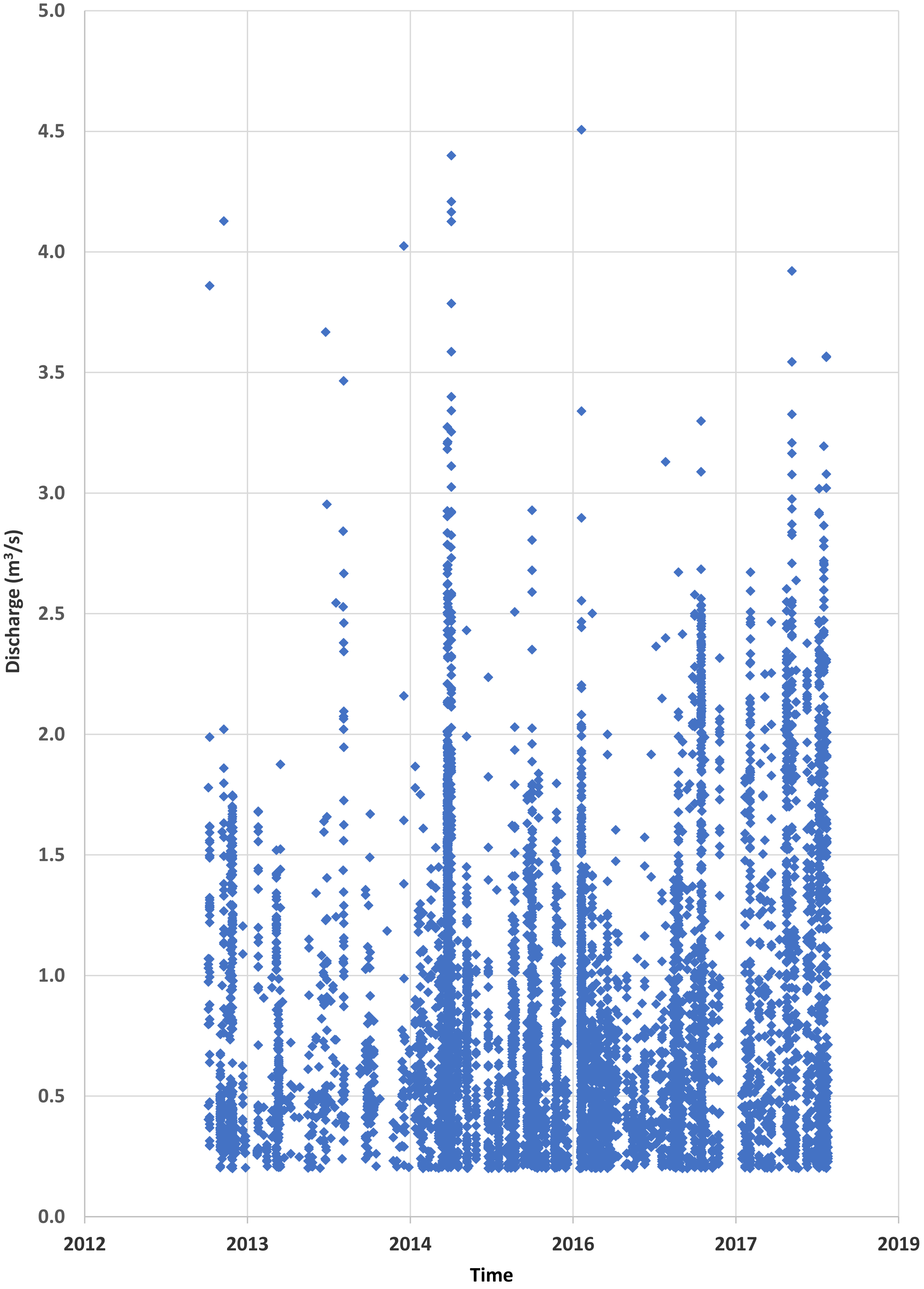


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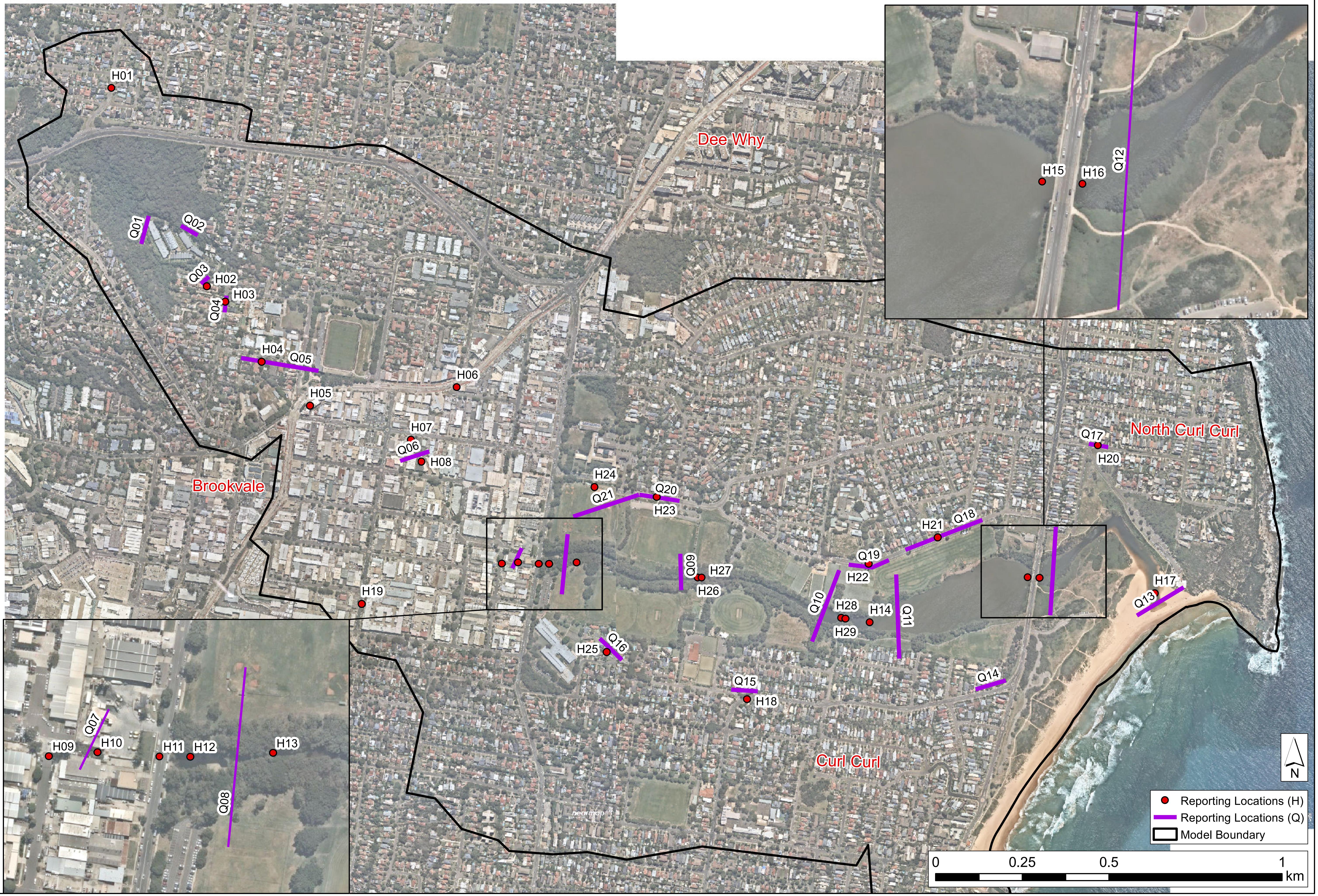
FIGURE 15
REPORTING LOCATIONS



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PIPE CAPACITY ASSESSMENT

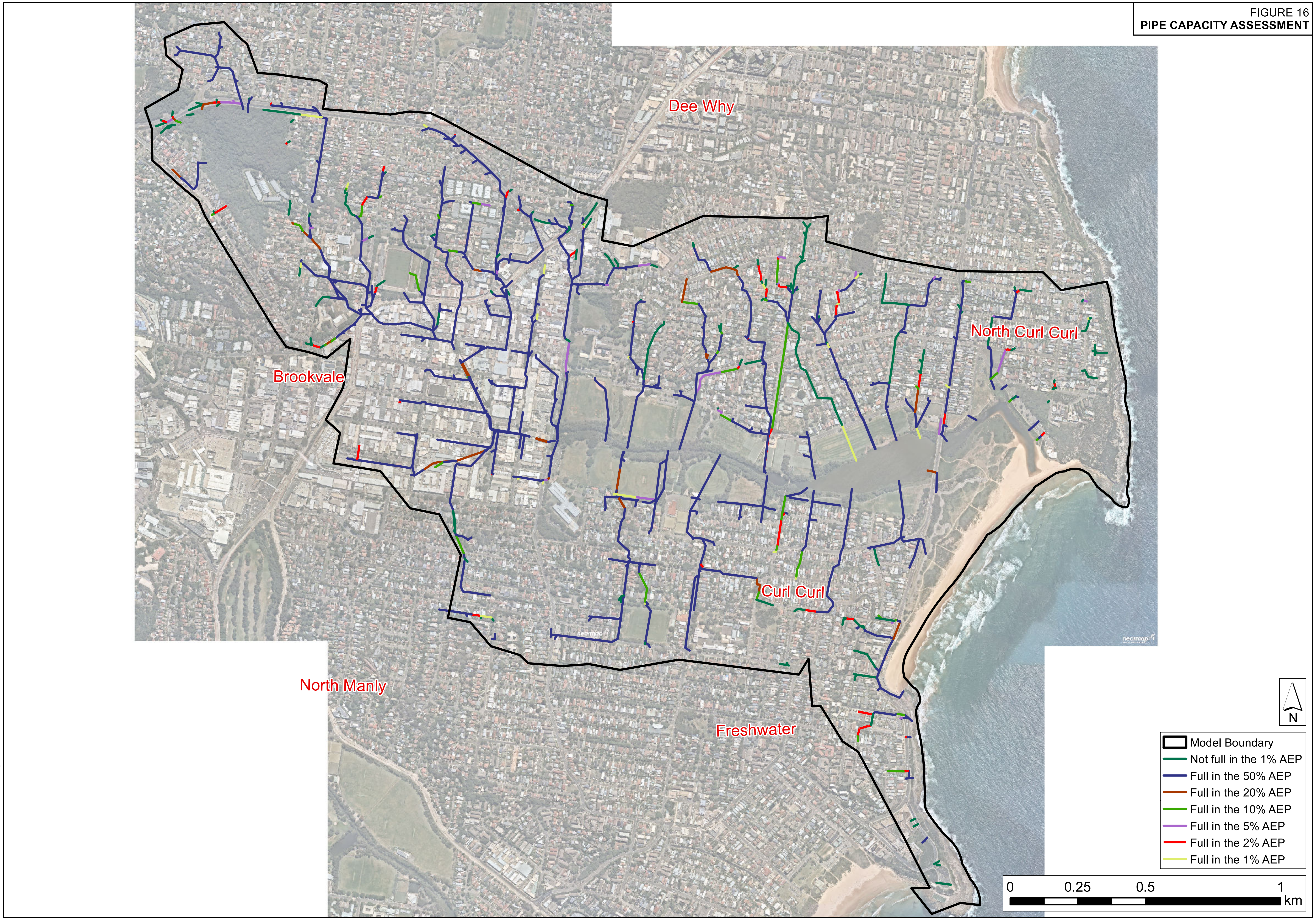

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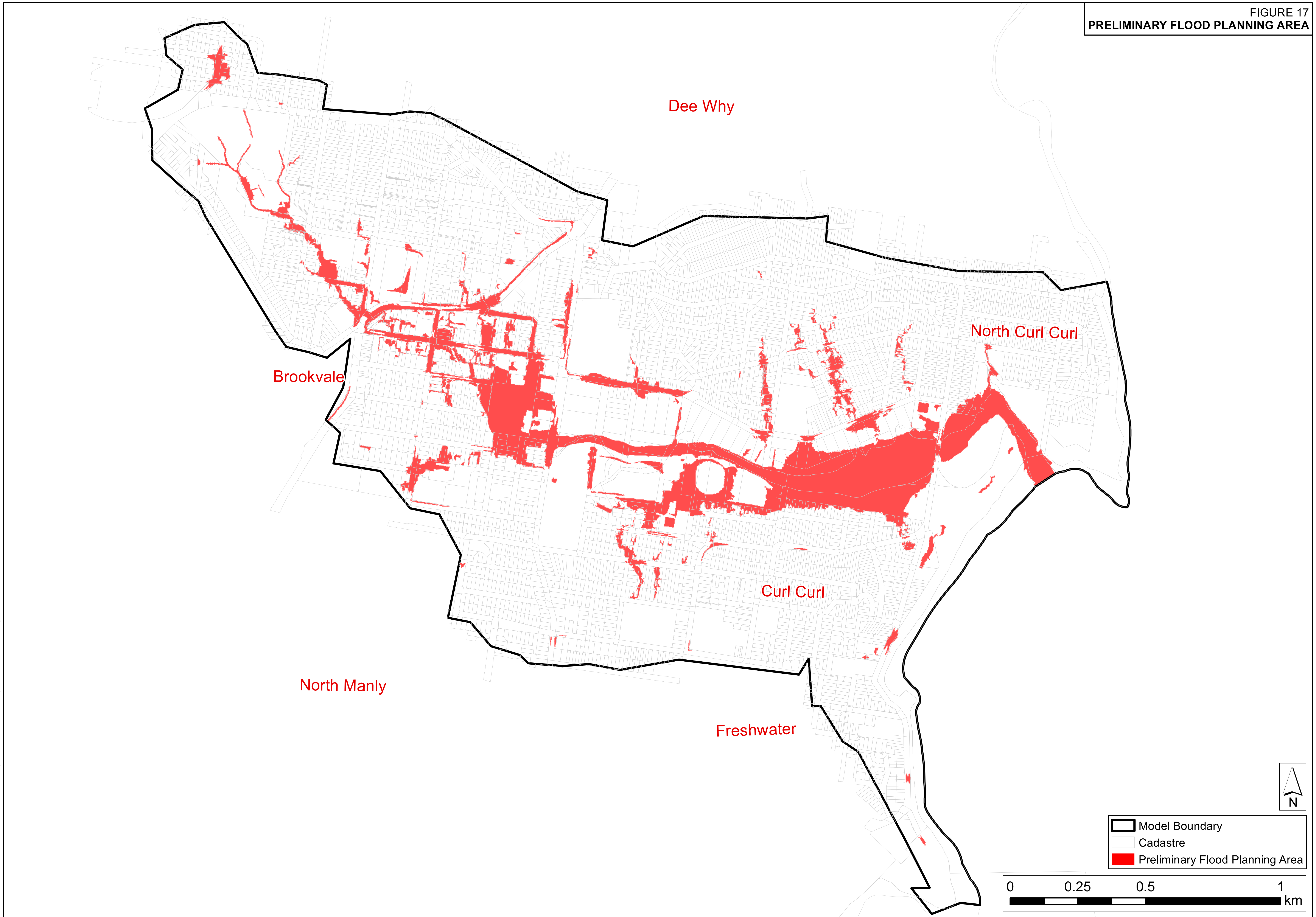
FIGURE 17
PRELIMINARY FLOOD PLANNING AREA



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## APPENDIX A.	ARR2019 Metadata

Appendix A

ATTENTION: This site was updated recently, changing some of the functionality. Please see the changelog (./changelog) for further information

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	151.284
Latitude	-33.764
Selected Regions (clear)	
River Region	show
ARF Parameters	show
Storm Losses	show
Temporal Patterns	show
Areal Temporal Patterns	show
BOM IFDs	show
Median Preburst Depths and Ratios	show
10% Preburst Depths	show
25% Preburst Depths	show
75% Preburst Depths	show
90% Preburst Depths	show
Interim Climate Change Factors	show
Probability Neutral Burst Initial Loss (./nsw_specific)	show

Data

River Region

Division	South East Coast (NSW)
River Number	13
River Name	Sydney Coast-Georges River

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2016_v1

ARF Parameters

A R F = M i n { 1 , [1 – a (A r e a b – c log 10 D u r a t i o n) D u r a t i o n – d + e A r e a f D u r a t i o n g (0.3 + log 10 A E P) + h 10 i A r e a D u r a t i o n 1440 (0.3 + log 10 A E P)] }

Zone	a	b	c	d	e	f	g	h	i
SE Coast	0.06	0.361	0.0	0.317	8.11e-05	0.651	0.0	0.0	0.0

Short Duration ARF

A R F = M i n [1 , 1 – 0.287 (A r e a 0.265 – 0.439 log 10 (D u r a t i o n)) . D u r a t i o n – 0.36 + 2.26 x 10 – 3 x A r e a 0.226 . D u r a t i o n 0.125 (0.3 + log 10 (A E P)) + 0.0141 x A r e a 0.213 x 10 – 0.021 (D u r a t i o n – 180) 2 1440 (0.3 + log 10 (A E P))]

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2016_v1

Storm Losses

Note: Burst Loss = Storm Loss - Preburst

Note: These losses are only for rural use and are **NOT FOR DIRECT USE** in urban areas

Note: As this point is in NSW the advice provided on losses and pre-burst on the NSW Specific Tab of the ARR Data Hub (./nsw_specific) is to be considered. In NSW losses are derived considering a hierarchy of approaches depending on the available loss information. The continuing storm loss information from the ARR Datahub provided below should only be used where relevant under the loss hierarchy (level 5) and where used is to be multiplied by the factor of 0.4.

ID	17135.0
Storm Initial Losses (mm)	28.0
Storm Continuing Losses (mm/h)	1.6

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2016_v1

Temporal Patterns | Download (.zip) (static/temporal_patterns/TP/ECsouth.zip)

code	ECsouth
Label	East Coast South

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2016_v2

Areal Temporal Patterns | Download (.zip) (./static/temporal_patterns/Areal/Areal_ECsouth.zip)

code	ECsouth
arealabel	East Coast South

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2016_v2

BOM IFDs

Click here (http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016&coordinate_type=dd&latitude=-33.764433133&longitude=151.283701599&sdmin=true&sdhr=true to obtain the IFD depths for catchment centroid from the BoM website

Layer Info

Time Accessed	25 September 2019 12:07PM

Median Preburst Depths and Ratios

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	7.0 (0.211)	7.7 (0.177)	5.6 (0.110)	3.5 (0.060)	2.1 (0.030)	1.0 (0.013)
90 (1.5)	13.2 (0.351)	9.5 (0.193)	6.5 (0.113)	3.6 (0.055)	2.1 (0.028)	1.0 (0.012)
120 (2.0)	12.8 (0.311)	7.7 (0.144)	6.6 (0.106)	5.6 (0.079)	4.2 (0.051)	3.2 (0.034)
180 (3.0)	5.3 (0.114)	6.5 (0.107)	6.3 (0.090)	6.2 (0.076)	6.9 (0.073)	7.4 (0.070)
360 (6.0)	7.3 (0.122)	11.0 (0.141)	13.4 (0.147)	15.7 (0.150)	18.9 (0.152)	13.0 (0.094)
720 (12.0)	6.2 (0.080)	11.0 (0.106)	14.1 (0.115)	17.1 (0.120)	26.1 (0.153)	30.3 (0.157)
1080 (18.0)	1.8 (0.020)	9.3 (0.075)	14.3 (0.096)	19.1 (0.110)	22.2 (0.107)	28.3 (0.120)
1440 (24.0)	1.8 (0.017)	5.6 (0.039)	8.1 (0.048)	10.5 (0.053)	21.1 (0.088)	26.9 (0.099)
2160 (36.0)	0.0 (0.000)	1.8 (0.010)	2.9 (0.014)	4.0 (0.017)	9.4 (0.033)	11.5 (0.035)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	1.0 (0.003)	1.7 (0.005)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	1.2 (0.003)	2.2 (0.005)

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

10% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
90 (1.5)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

25% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.000)	0.2 (0.005)	0.1 (0.002)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
90 (1.5)	0.0 (0.001)	0.5 (0.010)	0.2 (0.004)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.1 (0.003)	0.1 (0.001)	0.0 (0.001)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.4 (0.003)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	1.2 (0.006)	2.2 (0.009)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.4 (0.002)	0.7 (0.003)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	25 September 2019 12:07PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

75% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	40.6 (1.232)	37.9 (0.873)	36.4 (0.717)	34.9 (0.602)	28.4 (0.418)	23.5 (0.310)
90 (1.5)	36.7 (0.976)	39.7 (0.809)	35.1 (0.613)	30.6 (0.469)	27.8 (0.364)	25.7 (0.302)
120 (2.0)	53.4 (1.298)	37.7 (0.703)	34.8 (0.558)	32.0 (0.450)	35.9 (0.431)	38.7 (0.417)
180 (3.0)	33.4 (0.713)	45.5 (0.749)	43.8 (0.619)	42.2 (0.522)	57.0 (0.601)	68.1 (0.642)
360 (6.0)	45.2 (0.759)	54.8 (0.705)	61.1 (0.671)	67.2 (0.641)	86.5 (0.698)	90.0 (0.646)
720 (12.0)	30.1 (0.386)	43.5 (0.419)	52.3 (0.426)	60.8 (0.426)	67.0 (0.393)	76.8 (0.398)
1080 (18.0)	22.9 (0.248)	34.7 (0.279)	42.5 (0.286)	50.0 (0.288)	71.4 (0.343)	82.2 (0.348)
1440 (24.0)	24.9 (0.240)	31.9 (0.225)	36.4 (0.214)	40.8 (0.205)	67.1 (0.280)	74.8 (0.275)
2160 (36.0)	5.0 (0.041)	15.4 (0.091)	22.3 (0.109)	28.9 (0.120)	51.5 (0.178)	60.1 (0.183)
2880 (48.0)	12.3 (0.089)	12.5 (0.065)	12.7 (0.055)	12.8 (0.047)	22.7 (0.070)	30.2 (0.082)
4320 (72.0)	0.0 (0.000)	0.3 (0.001)	0.5 (0.002)	0.7 (0.002)	20.9 (0.056)	33.8 (0.080)

Layer Info

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Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

90% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	95.5 (2.894)	94.8 (2.181)	96.9 (1.909)	98.9 (1.705)	103.6 (1.525)	107.1 (1.415)
90 (1.5)	72.3 (1.924)	105.2 (2.142)	103.3 (1.805)	101.4 (1.552)	113.6 (1.488)	122.8 (1.443)
120 (2.0)	88.3 (2.148)	94.6 (1.766)	98.1 (1.575)	101.5 (1.428)	108.2 (1.301)	113.2 (1.220)
180 (3.0)	86.3 (1.843)	102.2 (1.680)	108.8 (1.537)	115.1 (1.423)	126.6 (1.334)	135.2 (1.274)
360 (6.0)	78.0 (1.310)	89.9 (1.156)	97.7 (1.073)	105.2 (1.005)	158.1 (1.276)	175.3 (1.257)
720 (12.0)	62.3 (0.799)	86.2 (0.831)	102.0 (0.831)	117.2 (0.821)	133.6 (0.783)	148.3 (0.768)
1080 (18.0)	47.4 (0.514)	64.9 (0.522)	76.5 (0.516)	87.6 (0.505)	138.3 (0.664)	153.9 (0.651)
1440 (24.0)	58.2 (0.559)	70.3 (0.496)	78.3 (0.461)	86.1 (0.432)	118.4 (0.493)	130.7 (0.480)
2160 (36.0)	32.3 (0.263)	44.7 (0.263)	52.9 (0.259)	60.7 (0.253)	100.0 (0.346)	115.6 (0.352)
2880 (48.0)	26.6 (0.194)	39.1 (0.205)	47.5 (0.206)	55.4 (0.204)	76.8 (0.236)	92.7 (0.252)
4320 (72.0)	10.1 (0.064)	23.0 (0.104)	31.5 (0.118)	39.6 (0.127)	62.7 (0.168)	82.0 (0.195)

Layer Info

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Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

Interim Climate Change Factors

	RCP 4.5	RCP6	**RCP 8.5**
2030	**0.869 (4.3%)**	0.783 (3.9%)	**0.983 (4.9%)**
2040	**1.057 (5.3%)**	1.014 (5.1%)	**1.349 (6.8%)**
2050	**1.272 (6.4%)**	1.236 (6.2%)	**1.773 (9.0%)**
2060	**1.488 (7.5%)**	1.458 (7.4%)	**2.237 (11.5%)**
2070	**1.676 (8.5%)**	1.691 (8.6%)	**2.722 (14.2%)**
2080	**1.810 (9.2%)**	1.944 (9.9%)	**3.209 (16.9%)**
2090	**1.862 (9.5%)**	2.227 (11.5%)	**3.679 (19.7%)**

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Note	ARR recommends the use of RCP4.5 and RCP 8.5 values. These have been updated to the values that can be found on the climate change in Australia website.

Probability Neutral Burst Initial Loss

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	12.3	8.1	9.0	8.7	8.6	6.6
90 (1.5)	11.8	8.1	9.5	9.7	9.5	7.0
120 (2.0)	13.4	9.0	10.2	10.0	10.1	6.0
180 (3.0)	13.8	9.3	10.6	10.1	8.9	4.3
360 (6.0)	13.2	8.6	8.8	8.1	9.1	3.7
720 (12.0)	17.6	12.2	12.2	10.6	11.9	3.1
1080 (18.0)	18.3	13.5	14.6	12.0	13.3	3.8
1440 (24.0)	21.5	15.5	15.8	13.8	14.6	4.4
2160 (36.0)	24.2	18.4	18.4	15.9	16.6	6.9
2880 (48.0)	27.2	22.0	21.1	22.9	19.4	9.5
4320 (72.0)	29.4	25.5	25.5	25.7	21.8	10.5

Layer Info

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Note	As this point is in NSW the advice provided on losses and pre-burst on the NSW Specific Tab of the ARR Data Hub (./nsw_specific) is to be considered. In NSW losses are derived considering a hierarchy of approaches depending on the available loss information. Probability neutral burst initial loss values for NSW are to be used in place of the standard initial loss and pre-burst as per the losses hierarchy.

Download TXT (downloads/5fb0a5b7-6a43-43e2-ba3c-4f0a7b4d1096.txt)

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