REPORT

McCarrs Creek, Mona Vale and Bayview Flood Study Review

Client:	Northern Beaches Council
Reference:	8A0433_RP-01_McCarrs Creek, Mona Vale and Bayview Flood Study Review
Revision:	04/Final For Council Adoption
Date:	07 July 2017

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HASKONING AUSTRALIA PTY LTD.

Level 1
43 Bolton Street
NSW 2300 Newcastle
Australia
Maritime & Aviation
Trade register number: ACN153656252

+61 2 49269500 **T**
Infosydney.mandw@rhdhv.com **E**
royalhaskoningdhv.com **W**

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Author(s):	Patrick Carolan, Paul Hart and Ben Patterson

Drafted by:	Patrick Carolan
Checked by:	Ben Patterson
Date / initials:	
Approved by:	Ben Patterson
Date / initials:	

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Foreword

The NSW State Government's Flood Policy provides a framework to support the sustainable use of floodplains. The Policy is specifically structured to support development of mitigation 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 with their floodplain management responsibilities.

The Policy provides for technical and financial support by the Government through the following sequential stages (refer to the Floodplain Risk Management Process overleaf):

1. Establish Floodplain Risk Management Committee (or Working Group)

Conducts a vital oversight role for the floodplain risk management process, acting as a focus and forum for discussion of key issues in formulating the management plan.

2. Flood Study

Determines the nature and extent of the flood problem.

3. Floodplain Risk Management Study

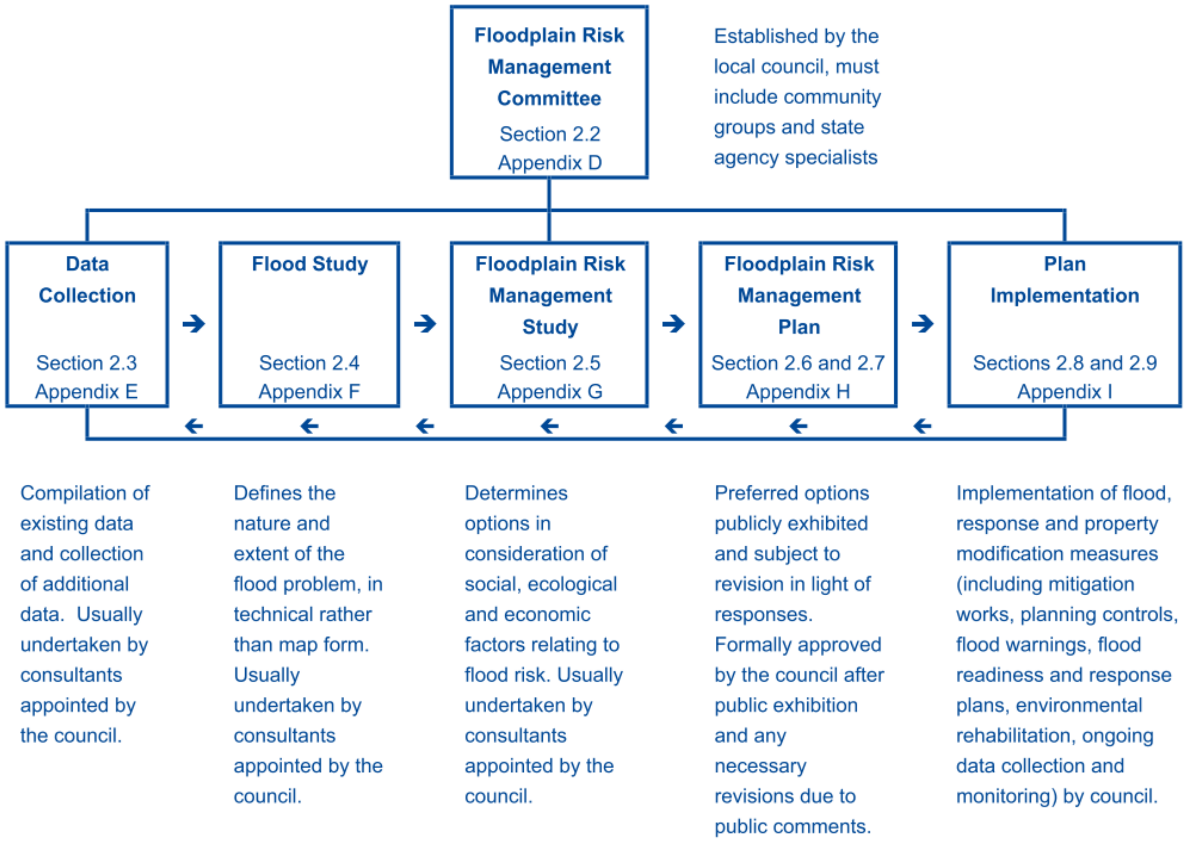
Evaluates management options for the floodplain in respect of both existing and proposed development.

4. Floodplain Risk Management Plan

Involves formal adoption by Council of a plan of management for the floodplain.

5. Implementation of the Plan

Construction of flood mitigation works to protect existing development, and use of flood risk management measures (such as development controls) to ensure new development is compatible with the flood hazard.



The Floodplain Risk Management Process (Floodplain Development Manual, 2005)

Glossary of Terms

annual exceedance probability (AEP)	AEP (measured as a percentage) is a term used to describe flood size. It is a means of describing how likely a flood is to occur in a given year. For example, a 1% AEP flood is a flood that has a 1% chance of occurring, or being exceeded, in any one year. It is also referred to as the ‘100 year ARI flood’ or ‘1 in 100 year flood’. The term 100 year ARI flood has been used in this study. See also average recurrence interval (ARI).
Australian Height Datum (AHD)	National survey datum corresponding approximately to mean sea level.
attenuation	Weakening in force or intensity
average recurrence interval (ARI)	ARI (measured in years) is a term used to describe flood size. It is the long-term average number of years between floods of a certain magnitude. For example, a 100 year ARI flood is a flood that occurs or is exceeded on average once every 100 years. The term 100 year ARI flood has been used in this study. See also annual exceedance probability (AEP).
catchment	The catchment at a particular point is the area of land that drains to that point.
design flood	A hypothetical flood representing a specific likelihood of occurrence (for example the 100yr ARI or 1% AEP flood).
development	Existing or proposed works that may or may not impact upon flooding. Typical works are filling of land, and the construction of roads, floodways and buildings.
discharge	The rate of flow of water measured in tems of vollume 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).
flood	A relatively high stream flow that overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland 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 behaviour	The pattern / characteristics / nature of a flood.
flood fringe	Land that may be affected by flooding but is not designated as floodway or flood storage.

flood hazard	The potential for damage to property or risk to persons during a flood. Flood hazard is a key tool used to determine flood severity and is used for assessing the suitability of future types of land use.The degree of flood hazard varies with circumstances across the full range of floods.
flood level	The height of the flood described either as a depth of water above a particular location (eg. 1m above a floor, yard or road) or as a depth of water related to a standard level such as Australian Height Datum (eg the flood level was 7.8 mAHD). Terms also used include flood stage and water level.
flood liable land	see flood prone land
floodplain	Land susceptible to flooding up to the probable maximum flood (PMF). Also called flood prone land. Note that the term flood liable land now covers the whole of the floodplain, not just that part below the flood planning level.
floodplain risk management study	Studies carried out in accordance with the Floodplain Development Manual (NSW Government, 2005) that assesses options for minimising the danger to life and property during floods. These measures, referred to as 'floodplain management measures / options', aim to achieve an equitable balance between environmental, social, economic, financial and engineering considerations. The outcome of a Floodplain Risk Management Study is a Floodplain Risk Management Plan.
floodplain risk management plan	The outcome of a Floodplain Risk Management Study.
flood planning levels (FPL)	The combination of flood levels and freeboards selected for planning purposes, as determined in Floodplain Risk Management Studies and incorporated in Floodplain Risk Management Plans. The concept of flood planning levels supersedes the designated flood or the flood standard used in earlier studies..
flood prone land	Land susceptible to inundation by the probable maximum flood (PMF) event. Under the merit policy, the flood prone definition should not be seen as necessarily precluding development. Floodplain Risk Management Plans should encompass all flood prone land (i.e. the entire floodplain).
flood stage	See flood level.
flood storage	Floodplain area that is important for the temporary storage of floodwaters during a flood.
flood study	A study that investigates flood behaviour, including identification of flood extents, flood levels and flood velocities for a range of flood sizes.

floodway	Those areas of the floodplain where a significant discharge of water occurs during floods. Floodways are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.
freeboard	A factor of safety usually expressed as a height above the adopted flood level thus determing the flood planning level. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
high flood hazard	For a particular size flood, there would be a possible danger to personal safety, able-bodied adults would have difficulty wading to safety, evacuation by trucks would be difficult and there would be a potential for significant structural damage to buildings.
hydraulics	The term given to the study of water flow in rivers, estuaries and coastal systems.
hydrology	The term given to the study of the rainfall-runoff process in catchments.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
low flood hazard	For a particular size flood, able-bodied adults would generally have little difficulty wading and trucks could be used to evacuate people and their possessions should it be necessary.
m AHD	metres Australian Height Datum (AHD).
m/s	metres per second. Unit used to describe the velocity of floodwaters.
m^3/s	Cubic metres per second or 'cumecs'. A unit of measurement for creek or river flows or discharges. It is the rate of flow of water measured in terms of volume per unit time.
overland flow path	The path that floodwaters can follow if they leave the confines of the main flow channel. Overland flow paths can occur through private property or along roads. Floodwaters travelling along overland flow paths, often referred to as 'overland flows', may or may not re-enter the main channel from which they left; they may be diverted to another water course.
peak flood level, flow or velocity	The maximum flood level, flow or velocity that occurs during a flood event.
probable maximum flood (PMF)	The largest flood likely to ever occur. The PMF defines the extent of flood prone land or flood liable land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with the PMF event are addressed in the current study.

probability	A statistical measure of the likely frequency or occurrence of flooding.
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall from a catchment that actually ends up as flowing water in the river or creek.
stage	See flood level.
topography	The shape of the surface features of land
velocity	The term used to describe the speed of floodwaters, usually in m/s.
water level	See flood level.

Executive Summary

BACKGROUND

Northern Beaches Council are responsible for land use planning within the Ingleside, Bayview and Mona Vale areas. The McCarrs Creek, Bayview and Mona Vale Flood Study covers 14 catchments (refer **Figure 1**), 11 of which drain into Pittwater Estuary (McCarrs Creek, Glen Cicada Creek, Gilwinga Drive, Browns Bay, BYRA, Loquat Valley, Fermoy Avenue, Bayview Park catchments, Cahill Creek including Mona Vale Main Drain, Yachtsman's Paradise and Edwin Ward Reserve catchments), 2 of which drain to the Pacific Ocean (Hill Crest catchment and the Mona Vale Golf Course Catchment).

The most relevant previous study, the Mona Vale and Bayview Flood Study, was undertaken by DHI in 1999 on behalf of Northern Beaches (Pittwater) Council using 1D MIKE11 hydraulic modelling. Since this previous flood study was carried out, significant changes in flood modelling capability have occurred, significant development across all the catchments has occurred, as well as the need to assess potential climate impacts.

The present Flood Study has been commissioned by Northern Beaches Council, with assistance from the NSW Office of Environment and Heritage (OEH). This study considers flooding from all sources: local storm runoff, creek flows as well as backwater flooding from tidal influences in the Pittwater estuary and Pacific Ocean.

This report details the results and findings of the Flood Study investigations. The key elements include:

- a description of the study area;
- a summary of available historical flood related data;
- establishment and calibration of the hydrologic and hydraulic models;
- the estimation of design flood behaviour for existing catchment conditions;
- sensitivity analysis of the model results to variation of input parameters;
- potential implications of climate change projections; and
- identification of the level of flood risk for individual properties in the catchment.

COMMUNITY CONSULTATION

A number of communication methods were employed in the community consultation process carried out for this study. This included distribution of a community consultation letter to over 4400 residents and businesses, the establishment of a web-page specifically for this study and an online questionnaire.

The primary aim of the consultation was to inform the community of the projects aims, objectives and timescales and to obtain historic flood information from the community that might benefit the study. In addition, the community consultation has in itself raised awareness of flooding issues across the study area.

A total of 48 responses were received to the online flood questionnaire of which 11 had experienced flooding. Residential flooding was reported in 8 separate events between 1988 and 2014 and included descriptions of flood mechanisms, flow paths, timing, depth, extent as well as flood photos and videos.

A Community Working Group was established including Councillors and representatives from the Community, State Emergency Services (SES), Sydney Water Corporation, Office of Environment and Heritage (OEH) and Roads and Maritime Services (RMS). The Working Group met regularly to discuss project progress and findings through the course of the Flood Study.

HYDROLOGIC AND HYDRAULIC MODELLING PROCESS

The hydrologic modelling was undertaken using a combination of the XP-RAFTS hydrological modelling software for some catchments and a direct rainfall approach within the hydraulic models for the majority of catchments. Hydraulic models were developed for all sub-catchments using a 1D/2D approach in the ESTRY/TUFLOW modelling suite. These models were calibrated to the April 1998 Event and verified against the October 1987 and January 1989 events.

The design rainfall events that were modelled were the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP design events and the Probable Maximum Precipitation (PMP). The temporal patterns for the design events were taken from Australian Rainfall and Runoff (AR&R) (Institute of Engineers Australia, 1987) and the Intensity-Frequency-Duration (IFD) data was taken from the Bureau of Meteorology's (BoM) internet-based tool. The PMP estimates were derived according to the BoM guidelines, the *Generalised Short Duration Method (BoM, 2003).*

OUTCOMES

The study outputs include design flood information such as peak flood levels and velocities, provisional flood hazard, preliminary hydraulic categorisation, preliminary flood planning extents and property classification according to Northern Beaches' Development Control Plan (DCP).

1 Introduction

1.1 Background

Northern Beaches Council (Council) are responsible for local land use planning within the McCarrs Creek, Mona Vale and Bayview catchments, and intend to prepare a floodplain risk management study and plan for these catchments. As a step towards that ultimate aim, Council have commissioned this flood study, which covers all flood prone land in the suburbs of Mona Vale, Bayview, Church Point and Ingleside and considers mainstream, overland and tidal flooding.

Mona Vale, Bayview and Church Point are relatively low lying, urbanised centres in close proximity to the Pittwater estuary. All have heavily modified drainage networks. Ingleside is a steep, rural suburb with some residential development. Importantly it includes the Ingleside Land Release Area, where significant future development plans are scheduled within the study area.

1.2 Catchment Description

The study area is situated approximately 25 kilometres north of the Sydney CBD and encompasses the suburbs of Mona Vale, Bayview, Church Point and Ingleside over 14 sub-catchments totalling an area of 18.2 square kilometres. **Figure 1** shows the study area and the 14 sub-catchments.

The Bayview and Church Point portions of the study area are characterised largely by residential urban development, while the Ingleside area predominantly consists of rural-residential zones and the Ku-rin-gai Chase National Park. Land use in the Mona Vale area is predominantly urban residential with heavily industrial area in the immediate surrounds of the Mona Vale Main Drain. Two golf courses exist within the study area, in the lower portions of the Bayview and Mona Vale areas.

The majority of the 14 sub-catchments drain through traditional pit and pipe stormwater infrastructure and open channels and outlet north into the Pittwater Estuary. The Hillcrest and Mona Vale Golf Course catchments drain through similar stormwater infrastructure to the east and directly into the Pacific Ocean. In larger storm events, bypass flows from the Hillcrest catchment cross the sub-catchment boundaries and accumulate behind Barrenjoey Road, subsequently draining into the Mona Vale Main Drain. The Cahill Creek sub-catchment adjoins the downstream reaches of the Mona Vale Main Drain, eventually draining north to the Pittwater Estuary.

The terrain in the study area ranges from approximately 180 mAHD down to sea level, where the upstream portions of the sub-catchments are generally quite steep and range from 15 - 50% in grade. The lower areas of the Cahill Creek, Mona Vale Main Drain and Mona Vale Golf Course floodplains are generally flat with grades lower than 1%. The relatively natural catchments of McCarrs Creek and Cicada Glen Creek are characterised by steep bushland with longitudinal grades along the creek in the order of 1-5%.

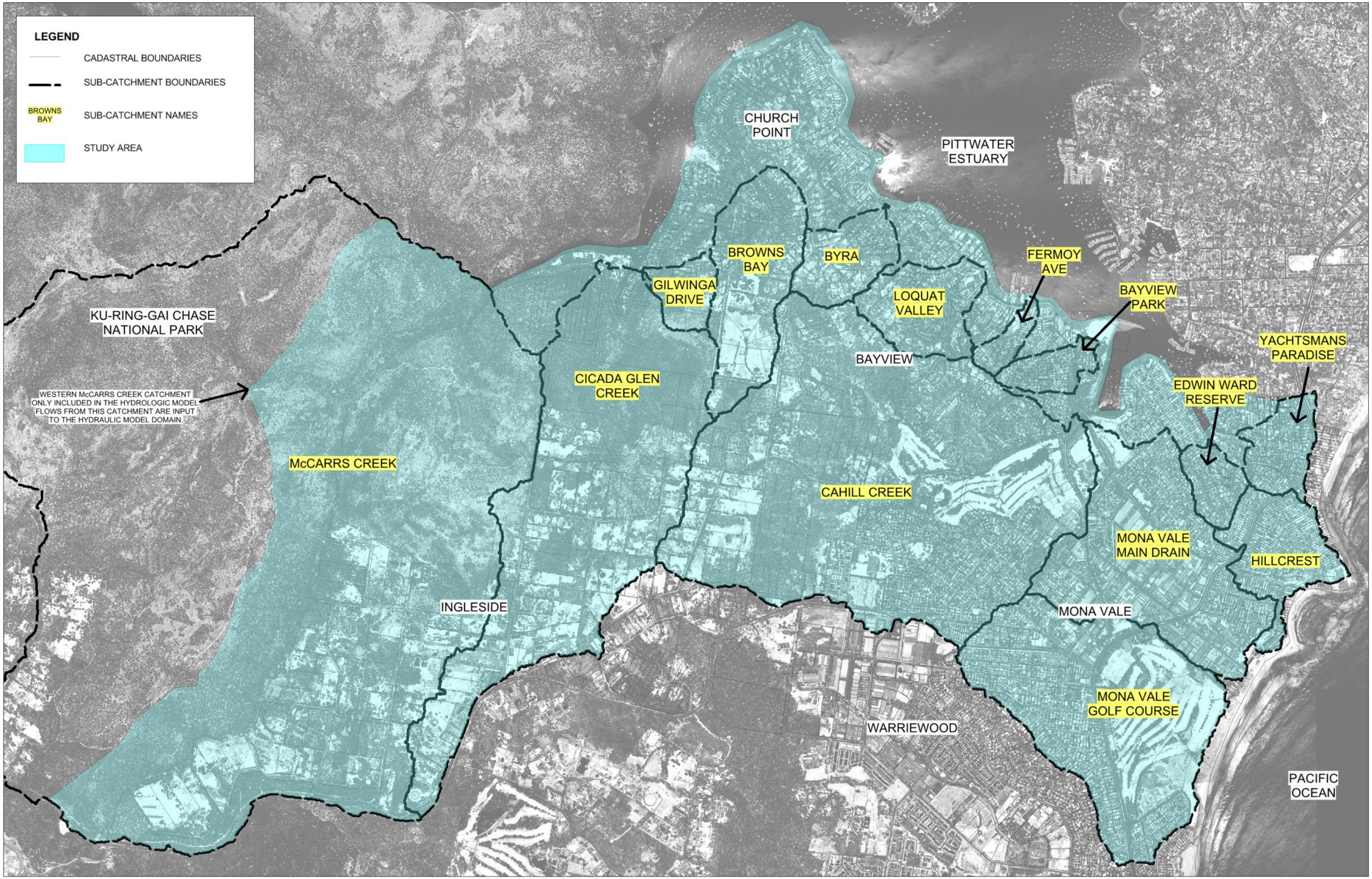


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FIGURE 1 **STUDY AREA AND SUB-CATCHMENTS**

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1.3 Study Objectives

The key objective of the Flood Study is to gain a comprehensive understanding of mainstream, overland and tidal flood risk in the above catchments. This study will form the basis of Councils future ability to undertake a Floodplain Risk Management Study and Plan and take sound flood related planning decisions for existing and future developments.

The study was developed for the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% AEP design events and the Probable Maximum Flood (PMF). The primary objectives of the study are:

- to determine the flood behaviour including design flood levels, velocities and flood extents within the 14 study catchments;
- to determine provisional residential flood planning levels and flood planning area;
- to assess the sensitivity of flood behaviour to potential climate change effects such as increases in rainfall intensities and sea level rise;
- to assess the floodplain categories in accordance with Council policy and undertake provisional hazard mapping; and
- to estimate the potential flood impact of the Ingleside Land Release.

This report details the results and findings of the Flood Study investigations. The key elements include:

- a description of the study area;
- a summary of available historical flood related data;
- establishment and calibration of the hydrologic and hydraulic models;
- the estimation of design flood behaviour for existing catchment conditions;
- sensitivity analysis of the model results to variation of input parameters;
- potential implications of climate change projections;
- to identify the level of flood risk for individual properties in the catchment; and
- description of the potential flood impact of the Ingleside Land Release.

1.4 Justification for this Study

This Flood Study has been undertaken for the following reasons:

- None of the study catchments, with the exception of the Cahill Creek Catchment, have previously been subject to detailed flood modelling or investigation;
- The Mona Vale - Bayview Flood Study (2002), which studied the Cahill Creek Catchment, is over 10 years old and was undertaken using 1D hydraulic modelling only;
- Recent completion of the 2015 Pittwater Estuary Mapping of Sea Level Rise Impacts Study and the 2013 Pittwater Overland Flow Flood Study highlighted both potential flood risk across the study area and the incompleteness of current knowledge; and
- A need to understand the potential flood risk impacts from the Ingleside Land Release.

1.5 About This Report

This report documents the Study's objectives, results and recommendations.

Section 1 introduces the study.

Section 2 provides an overview of the approach adopted to complete the study.

Section 3 outlines the community consultation program undertaken.

Section 4 details the development of computer models.

Section 5 details the model calibration and validation process.

Section 6 presents the design flood conditions.

Section 7 discusses model sensitivity.

Section 8 reviews relevant literature relating to climate change within the study area.

Section 9 outlines appropriate development controls in the sub-catchments.

2 Study Approach

2.1 Available Data

The data used for this study is presented and discussed in the following sections.

2.1.1 Previous Studies

Mona Vale - Bayview Flood Study, DHI (2002)

This report details the Flood Study undertaken by DHI on behalf of Northern Beaches (Pittwater) Council for the Mona Vale Main Drain and Cahill Creek sub-catchments. Hydrological modelling was undertaken using RDII, (DHI, 2001), which was an update to DHI's hydrological model, MOUSENAM. One-dimensional hydraulic modelling was undertaken using MIKE11, (DHI, 2001). Calibration of these models was undertaken for a storm event in April of 1998. Two events in January 1989 and October 1987 were used to verify the model calibration.

Pittwater Overland Flow Mapping and Flood Study, Cardno (2013)

This study identified properties and areas potentially affected by overland flow rather than "mainstream" flooding. A full dynamic two-dimensional (2D) SOBEK hydraulic model was used to define the overland flow behaviour under existing conditions and climate change scenarios. A range of flood events were considered, including the 5, 20, 100 year ARI and PMF events. Major hydraulic structures were included in the hydraulic modelling which used a 5 x 5 metre cell size.

2.1.2 Topographic Data

Airborne Light Detection and Ranging (LiDAR) survey of the catchment and its surrounds was provided for the study by Northern Beaches Council. This data was collected by two different sources in 2007. RHDHV also provided LiDAR for the study from their Land and Property Information (LPI) database. This dataset was collected in May of 2011.

The 2011 LiDAR dataset was utilised in this study as the time of survey allowed for more accurate representation of the current topographical features in the catchment. The accuracy of this dataset is reported as follows:

- Spatial Accuracy Horizontal = 0.8m
- Spatial Accuracy Vertical = 0.3m
- The accuracy of Aerial Laser Scanning (ALS) data can be influenced by the presence of open water or vegetation (tree shrub or canopy) at the time of the survey.

2.1.3 Pit and Pipe Data

Northern Beaches Council provided an asset database including the locations and dimensions of the majority of stormwater pits and pipes within the study area. The depths from the ground surface to pipe inverts were also included in the database for most entries.

2.1.4 Stream Flow Gauge

The Manly Hydraulics Laboratory (MHL) has been operating a flow gauge on the Darley Street Tributary of the Mona Vale Creek since July 2013. Details of this gauge are presented in **Table 2-1**, below.

Table 2-1: Details of the MHL flow gauge

Station Number	Station Name	Operating Authority	Location	Record Start Date	Record Close Date	Type
2134101	Darley Street, Mona Vale Stormwater	MHL	Mona Vale Main Drain Catchment. North Darley St.	2/07/13	Ongoing	Records discharge in l/s every 5 mins.

Since installation of the flow gauge, there have only been a number of very small events, limiting the gauge's usefulness for the purposes of this study. The largest event on record was equivalent to less than a 1 exceedance year (EY) event (i.e. less than a 1 year recurrence interval).

2.1.5 Historical Flood Level Data

Historical flood level data was obtained from previous reports provided by Northern Beaches Council. For the storm events in 1987, 1989 and 1999, surveyed flood mark information was sourced from the DHI (2002) Mona Vale - Bayview Flood Study. The tabulated flood mark data from this study is presented in **Section 5.1**.

2.1.6 Historical Rainfall Data

Table 2-2 presents the rainfall stations within the vicinity of the study area.

Table 2-2: Details of Rainfall Gauges Within the Vicinity of the Study Area

Station Number	Station Name	Operating Authority	Location	Elevation (m AHD)	Record Start Date	Record Close Date	Type
566145	Avalon Rain (Live)	MHL	Approx. 4km north of study area	-	27/06/94	Ongoing	TBR*/ Pluviometer
566146	Mona Vale Rain (Live)	MHL	Mona Vale Main Drain Catchment	-	27/06/94	Ongoing	TBR*/ Pluviometer
2134111	Narrabeen Creek / Warriewood Rain	MHL	Approx. 1.5km south of study area	-	15/05/98	18/9/2010	TBR*/ Pluviometer
66141	Mona Vale Golf Club	BoM	Mona Vale Golf Club	10	1/02/69	Ongoing	Daily
66183	Ingleside Animal Welfare League	BoM	Cicada Glen Creek Catchment	160	1/01/84	31/12/12	Daily
66059	Terry Hills	BoM	500m south of study area	199	3/01/08	Ongoing	1 min

**TBR – Tipping Bucket Recorder*

2.1.7 Previous Models

1D (MIKE 11) Hydraulics Model

A 1D MIKE11 hydraulic model was developed by DHI in 2002 as part of the Mona Vale and Bayview Flood Study. Some topography data has been extracted from the model and included in this study. The majority of data extracted from this model was situated on the Bayview Golf Course.

2D (SOBEK) Hydraulics Model

This model was developed by Cardno as part of the Pittwater Overland Flow Mapping and Flood Study, Cardno (2013). The SOBEK model uses a 5m x 5m grid with levels assigned by LiDAR data acquired in 2009. The model provided a useful initial cross check of flood results.

2.1.8 Detailed Survey

Detailed survey was gathered by Mepsteads Associates in 2014 as part of this study. A survey brief was prepared following the review of all available data and identification of critical data gaps. Given the size of the total study area, a compromise was made between the survey cost and appropriate

The survey covered a sample number of pits (up to 30) including pit cover, inlet levels, pipe sizes and inlet type. This sample survey was used to verify that the assumptions (outlined in **Section 4.2.5**) used on the pit and pipe asset database were reasonable. In addition, a number of culvert structures were surveyed as well as the thalweg (centreline) and overbanks of a number of open channels.

2.2 Site Inspections

A number of site inspections were undertaken by RHDHV staff during the course of the study to gain an appreciation of local features influencing flooding behaviour. Some of the key observations to be accounted for during the site inspections included:

- Presence of local structural hydraulic controls such as embankments and kerbs that may have an impact on overland flooding behaviour;
- Confirmation of the location and configuration of the stormwater drainage pits and outlets;
- Land use types and vegetation characteristics; and
- Location of existing development and infrastructure on the floodplain.

This visual assessment was useful for defining hydraulic properties within the hydraulic model (such as hydraulic roughness) and ground-truthing of topographic features identified from survey.

2.3 Community Consultation

The success of a floodplain management plan hinges on its acceptance by the community, residents within the study area, and other stake-holders. This can be achieved by involving the local community at all stages of the decision-making process. This includes the collection of their ideas and knowledge on flood behaviour in the study area, together with discussing the issues and outcomes of the study with them.

The key elements of the consultation process in undertaking the flood study have included:

- Issue of an online questionnaire to obtain historical flood data and community perspective on flooding issues;
- Involvement of community representatives on the Floodplain Management Working Group; and
- Public exhibition of Draft the Report and community information session.

These elements are discussed in further detail in **Section 3**.

2.4 Development of Computer Models

2.4.1 Hydrological Model

Traditionally, for the purpose of the Flood Study, a hydrologic model is developed to simulate the rate of storm runoff from the catchment. The output from the hydrologic model is a series of flow hydrographs at selected locations such as at stormwater drainage pit inlets, which form the inflow boundaries to the hydraulic model.

In recent years, the advancement in computer technology has enabled the use of the direct rainfall approach as a viable alternative. With the direct rainfall method the design rainfall is applied directly to the individual cells of the 2D hydraulic model.

This is particularly useful for overland flow studies where model results are desired in areas with very small contributing catchments. This study has adopted both a traditional approach (i.e. using a hydrologic and hydraulic model) and the direct-rainfall approach to model different parts of the study area. Details of both model's development are discussed in **Section 4.**

2.4.2 Hydraulic Model

Three TUFLOW hydraulic models (discussed in **Section 4.2**) were developed for this study. The models include:

- two-dimensional (2D) representation of the 14 sub-catchments, covering an area of approximately 18.6 km^2 (complete coverage of the total study area); and
- one-dimensional (1D) representation of the stormwater pipe network.

The hydraulic models were applied to determine flood levels, velocities and depths across the study area for historical and design events.

2.5 Calibration and Sensitivity Testing

The hydrodynamic model was primarily calibrated to the April 1998 flood event to establish the values of key model parameters and confirm that the models were capable of adequately simulating real flood events. The following criteria are generally used to determine the suitability of historical events to use for calibration or validation:

- The availability, completeness and quality of rainfall and flood level event data;
- The amount of reliable data collected during the historical flood information survey; and
- The variability of events – preferably events would cover a range of flood severity.

The available historical information highlighted only one flood with sufficient data to potentially support a calibration process – the April 1998 event. However, flood information relating to the October 1989 and October 1987 events has also been used to aid the model calibration and validation process.

The calibration and validation of the model is presented in **Section 5**. A series of sensitivity tests were also carried out to evaluate the model. These tests were conducted to examine the performance of the models and determine the relative importance of different hydrological and hydrodynamic factors. The sensitivity testing of the model is detailed in **Section 7**.

2.6 Establishing Design Flood Conditions

Design floods are statistical-based events which have a particular probability of occurrence. For example, the 1% Annual Exceedance Probability (AEP) event, which is sometimes referred to as the 100 year Average Recurrence Interval (ARI) flood, is the best estimate of a flood with a peak discharge that has a 1% (i.e. 1 in 100) chance of occurring in any one year. For the Flood Study catchments, design floods were based on design rainfall estimates according to Australian Rainfall and Runoff (IEAust, 1987).

The design flood conditions form the basis for floodplain management in the catchments and in particular design flood planning levels for future development controls. The predicted design flood conditions are presented in **Section 6**.

2.7 Flood Model Results Presentation

Design flood result presentation was undertaken using output from the hydrodynamic model. Figures were produced showing estimated water depth and velocity for each of the design events. The figures present the peak value of each parameter. Provisional flood hazard and hydraulic categories derived from the hydrodynamic model results are also presented.

2.7.1 Map Filtering

Map filtering is a required component for producing flood mapping for this study as the direct rainfall model applies water to the entire domain of the model. In order to produce realistic flood extents, the criteria outlined in **Table 2-3** have been applied to the maps for events up to the 1% AEP Event. **Table** 2-4 presents the criteria applied to the maps for events larger than the 1% AEP Event. An outcome of the filtering process is that “puddles” may become evident within the flood extents. Puddles within the model results can occur for the following reasons:

1. The direct rainfall modelling approach is reliant on the model DEM to determine flood results. The LiDAR data utilised to create the model DEM is filtered by the data supplier, using an algorithm to remove buildings and vegetation from the survey. This filtering process can result in depressions in the DEM which do not reflect actual ground conditions.

2. Underground carparks may formulate puddles from the direct application of rainfall to these cells.

3. Legitimate depressions in the topography such as roadway sag points will collect flows, with no obvious signs of an overland flow path, nevertheless these areas are considered worthy of including in the flood maps.

The handling of puddle flood results is discussed in **Table 2-3** and **Table 2-4** and further below.

Table 2-3: Map Filtering Criteria for Events up to the 1% AEP Event

Filtering Criteria	Justification
Depth ≥ 0.15m	Depths above 0.15m are considered significant and contribute to the flood extent. Depths below 0.15m are only considered significant where flood waters have an associated significant velocity (refer below).
Depth below 0.15m with a velocity depth product > 0.3m²/s	Includes significant flowpaths under 0.15m of depth in the mapping.
Depth below 0.15m with a velocity depth product between 0.025m²/s and 0.3m²/s	These areas are considered local stormwater and were removed from the flood study mapping. The local stormwater extents were provided to Council for management thorugh the stormwater clause of the DCP.
"Puddles" less than 100m² removed from the flood extents	Excludes insignificant "puddles" from direct rain model results.

Table 2-4: Map Filtering Criteria for Events Greater than the 1% AEP Event

Filtering Criteria	Justification
Depth ≥ 0.15m	Depths above 0.15m are considered significant and contribute to the flood extent.
"Puddles" less than 200m² removed from the flood extents	Excludes insignificant "puddles" from direct rain model results.

Further minor editing of flood results was required following the map filtering outlined in **Table 2-3** and **Table** 2-4 to achieve the final flood results. The following additional measures were applied to model outputs:

- Puddles isolated around buildings were removed from the flood extents. These were attributed to the filtering of LiDAR data to remove buildings from the DEM. Areas similarly affected by the filtering of vegetation from the DEM were also removed from the flood extents unless an overland flow path was evident.
- Puddles isolated on underground carparks were removed from the flood extents unless significant flood depth on the adjoining roadway was observed. These areas were attributed to direct rainfall applied to the underground carpark down ramp rather than flooding.

The flood model outputs are described in **Section 6.3** and presented in **Appendix A**.

3 Community Consultation

3.1 Introduction

Community consultation is extremely important in the Flood Study process. A range of consultation and communication methods have been utilised with the following aims:

- Inform the community that the study is being undertaken and its aims and objectives;
- Utilise the communities flood knowledge within the Flood Study; and
- Inform and discuss the Flood Study results with the community and gain their confidence in the findings, and to raise awareness of flooding in the community.

3.2 Property Owner Letter and Online Survey

On 26 September 2014 Council sent all known residential and business addresses within the study area an initial community consultation letter. In total 4,443 letters were sent. A copy of the letter can be found in **Appendix B**.

The purpose of the letter was to inform the community of the study and its aims and objectives, and to request that members of the community submit flood information and knowledge to assist the formulation of the study. The letter also sought community representatives to be part of a McCarrs Creek, Mona Vale and Bayview flood study community working group.

To assist the community in submitting flood information and knowledge, an online questionnaire / survey was set-up using the 'Survey Monkey' software. A link to the survey was included in the letter and on Council's website. A total of 48 completed survey responses were received by 30th October 2014. This represents a 1% return rate to date. Most of the flood information that has been received is anecdotal in nature and therefore has been treated with caution and be subject to verification prior to adoption. A graphical summary of answers to key questions is provided in **Appendix B**.

The following flood related information was collated:

- A property on Eastview Road has experienced annual flooding since its current resident moved in 4 years ago. Flooding occurs under the house and is generally up to 7cm deep. The floodwater flows down the hill and floods the neighbour's property before exiting onto McCarrs Creek Road. The source of flooding is described as heavy rain combined with poor public drainage.
- A property on Minkara Road, Bayview was flooded in 1988. The floodwater flowed through the ground floor level of the property at a depth of approx. 5cm. Water inundated the property for 1 day. This is the only known flood event at the property in the past 26 years. The owner blames the flooding on Council for altering the footpath level outside the property. No flooding has been experienced since 1988 when the footpath level was adjusted by council.
- A property owner on Rednal Street, Mona Vale, who has lived there for 7 years alluded to possible tidal flooding of their garden in 2009 and 2011 but stated they were unsure if flooding had occurred at the dwelling.

- The garden and driveway of a property in Church Point have flooded every year since 2010. The owner has lived at the property for 15 years, which suggests no flooding occurred between 2000 – 2010. Floodwaters flow down the hill onto the property and pond for approx. 1 week. A total of 4 flood videos have been provided to Council.
- The garage of a property on Pittwater Road, Bayview experienced flooding in 2014. The floodwater flowed from the back to the front under the house and ponded for 2 days. The source of flooding is described as overland flow.
- Flooding in Pamela Reserve was reported to have occurred in 2012. The water overtopped the drainage pipe that runs through the park and flowed down the side of a property in Pamela Crescent, Bayview. Blockage by tree debris is reported to be the cause of flooding. The creek in the eastern part of the reserve was flowing fast.
- A property on Crescent Road, Newport experienced a near miss in 2013 and had to sandbag the front door and garage to prevent flow on the street from entering the property. The cause of flooding is reported to be a street drain with insufficient capacity.
- The garage, garden and driveway of a property on Seabeach Avenue has experienced flooding in 2008, 2010 and 2012. Flood depth in the garage is reported to be 20cm. The cause of flooding is described as poor maintenance of the Sea Road near Kennards.
- The owners of a property in Church Point, reported very fast overland flow in 2014 from neighbouring properties across the Council strip and onto the road. The flooding was approx. 25cm deep, 10m wide and flowed for 4 hours.
- A property on Rednal Street, Mona Vale, reported that a King tide caused tidal inundation of their backyard in 2014. This was the only flooding reported in the 3 years the owners have lived at the property.
- Properties along Pittwater Road, Mona Vale are reported to flood whenever there is heavy downpours. Water enters about 6 shop fronts, under the doors, up to a depth of 15-20cms. The flood mechanism is high tide preventing drainage and car wash from Pittwater Road. The water normally recedes with 2 hours.

3.3 Individual Stakeholder Engagement

The following individual project stakeholders have been contacted during the data collection stage of the project.

- **Bayview Golf Course –** Mr David Stone from Bayview Golf Course accompanied Paul Hart, Patrick Carolan and David Mepstead on a tour of the golf course on 9 September 2014. Mr Stone provided some topographic information of the golf course and indicated flood prone areas and flow directions. Mr Stone indicated that the golf course would be interested in participating in the McCarrs Creek, Mona Vale and Bayview flood working group.
- **Mona Vale Golf Course** – Mr Andy Hugill was contacted and access to the golf course arranged for the site visit on 9 September 2014. Flood photos of the golf course from February 1990 have been received via Council.
- **RMS** – RMS were contacted regarding the Mona Vale Road upgrade and widening project. Detailed design drawings of the scheme were received via RMS' design consultants GHD.

3.4 Website

Information on the Flood Study was posted on Councils website (link below) and feedback encouraged via a link to the online flood survey.

http://www.pittwater.nsw.gov.au/environment/natural_hazards/flooding/where_does_it_flood/mona_valebayview

3.5 Community Working Group

Council sought nominations for (via the property owner letter and website) and formed a Community Working Group.

The aim of the working group has been to act as a forum for the discussion of technical, social, economic and environmental issues in an advisory role to Council.

The working group members were:

- 2 x Councillors (Acting as Chairperson)
- Pittwater / Northern Beaches Council Officers (Manager, Catchment Management and Climate Change and
  A/Principal Officer, Floodplain Management)
- 2 x Citizen Representatives
- Bayview Golf Club
- Bayview Church Point Residents Association
- Mona Vale Chamber of Commerce
- NSW Office of Environment and Heritage (OEH)
- State Emergency Service – Warringah/Pittwater Unit and Sydney Northern Region
- Sydney Water
- Roads and Maritime Services

The group has met 3 times (so far) during the flood study, on the following dates:

- 12 February 2015. A presentation on the Flood Study aims and objectives and work thus far were given by Northern Beaches Council and Royal HaskoningDHV.
- 7 May 2015. A presentation on the hydrologic and hydraulic model build and calibration was given by Royal HaskoningDHV.
- 13 August 2015. A presentation on some initial draft Flood Study results was given by Royal HaskoningDHV.

3.6 Ground Truthing and Door Knocking

3.6.1 Ground Truthing Exercise

Preliminary flood maps were produced for the 10% and 1% AEP Events and the PMF Event, using the assumptions documented in this working paper. RHDHV staff undertook a ground truthing exercise with Council on the 6th May 2015 to verify these results against ground features. 14 areas of the floodplain were inspected across the study area. While flooding behaviour of the preliminary design events was generally found to be in line with expectations from the ground truthing exercise, a number of additional features were identified as part of this process which were included in the design event modelling.

3.7 Public Exhibition and Community Information Sessions

The draft flood study report was placed on public exhibition from the 29th February to the 8th of April, 2016. During this time, the community were encouraged to complete submissions to raise any concerns related to the flood study. Along with Council, RHDHV conducted one-on-one information sessions with residents to provide the public with further information and take on board comments from the public. Submissions were collated, resulting in some changes to the flood results presentation (outlined in **Section 2.7**) from the draft flood study results. A response to submissions memorandum was provided to Council with the revised flood results mapping, outlining the key changes for flood results and property affectation.

4 Model Development

For the purpose of a Flood Study, hydrologic and hydraulic models are commonly developed to assess a catchment's flood behaviour.

The **hydrologic model** simulates a catchment's rainfall-runoff processes, estimating stormwater flows that can be used for input into a hydraulic model.

The **hydraulic model** simulates the physical behaviour of water flowing in overland flow paths, watercourses and urban drainage networks and is a useful tool for estimating discharges, flood levels, flow velocities and flood hazard.

As outlined previously, recent developments in computer technology have enabled the efficient use of direct-rainfall modelling. This method combines the two modelling processes by applying rainfall directly to each cell in a two-dimensional hydraulic model. This study has adopted both a traditional approach (i.e. using a hydrologic and hydraulic model) and the direct-rainfall approach to model different parts of the study area (refer to **Section 4.2.1**).

The study area can be broken into three model areas (Refer **Figure 2**) based on catchment characteristics:

1. Model Area 1 – The 'Rural' Catchments. This area encompasses the McCarrs Creek and Cicada Glen Creek catchments, which are characterised as having little existing development area, with predominately rural lands, natural bushland and watercourses. A traditional modelling approach has been taken for these catchments, where both a hydrologic and hydraulic model have been developed using the XP-RAFTS and TUFLOW/ESTRY software packages respectively.

2. Model Area 2 – The 'Pittwater' Catchments. This area includes all of the small, steep highly urbanised catchments that drain into the southern foreshore of the Pittwater estuary. The direct-rainfall approach has been applied to these catchments, as it is an effective way of modelling overland flow paths and discharge to the drainage network in heavily urbanised areas. The TUFLOW/ESTRY software package was used for this area.

3. Model Area 3 – The 'Urban' Catchments. This model area consists of the majority of the study area, including the Cahill Creek and Mona Vale Main Drain floodplains and the Mona Vale Golf Course catchment. The direct-rainfall approach has been applied for this model area for the same reasons given above. The TUFLOW/ESTRY software package was also used for this area.

The following general steps have been undertaken in the development of the hydraulic model:

1. Delineation of the model topography catchment boundaries and drainage networks.

2. Inclusion of other physical characteristics such drainage features (i.e. bridges, stormwater pipes and channels and roughness values).

3. Review of hydrographic data from historic events for inclusion in the modelling (rainfall records, gauged flow and flood levels).

4. Calibration to a number of historic flood events (calibration refers to the adjustment of model parameters within reasonable limits, to best match modelled results to observed historical data).

5. Verification of the model against a number of historic flood events (verification refers to testing the models performance to other historic events without further adjustment to the model parameters).

6. Sensitivity Analysis of the model parameters to measure dependence of the results upon model assumptions.

Once model development is completed, it can be used for the following purposes:

- Establishing design flood conditions;
- Determining levels for flood planning control; and
- Modelling development or flood management options to assess hydraulic impacts.

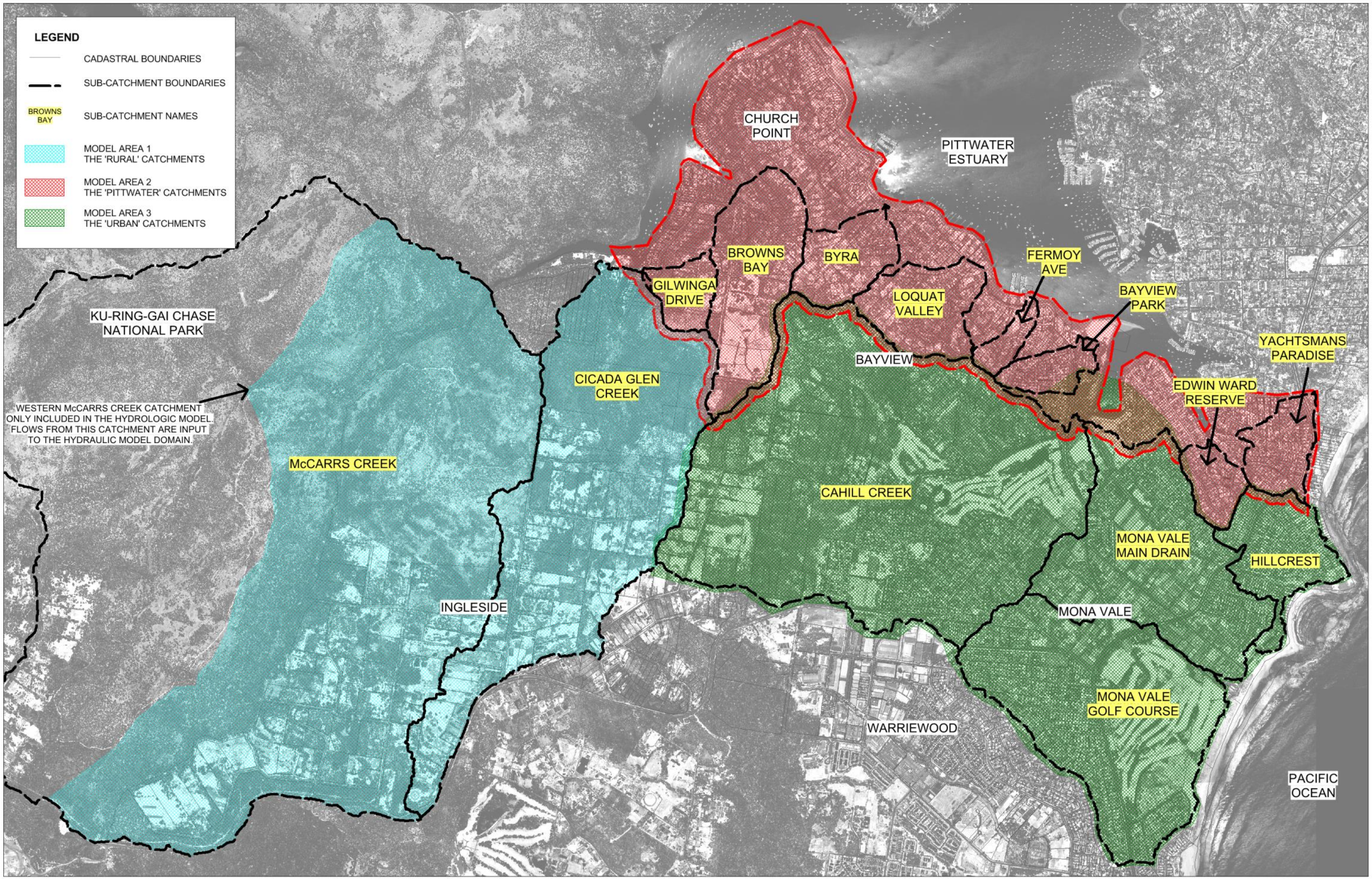

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FIGURE 2 **HYDRAULIC MODEL DOMAIN AREAS**

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4.1 Hydrological Model

The hydrologic model simulates the rate at which rainfall runs off the catchment. The amount of rainfall runoff from the catchment is dependent on:

- The catchment slope, area roughness due to, vegetation or buildings and fences as well as, and other characteristics;
- Variations in the distribution, intensity and total depth of rainfall; and
- The antecedent conditions (dryness/wetness) of the catchment.

Hydrological modelling was undertaken using the XP RAFTS software package to establish inflow boundaries to the Model Area 1 TUFLOW model. A direct rainfall approach was adopted for Model Area 2 and Model Area 3. The hydrological parameters and approach for both of these methods is discussed in the following sections.

4.1.1 Catchment Delineation

The study area drains an area of approximately 18.6 km^2 through a number of channels, floodplains and drainage networks both north to the Pittwater estuary and east to the Pacific Ocean. The study area can be broken into 14 main sub-catchments (refer **Figure 2**), with some remaining minor overland flow areas in Model Area 2. **Table 4-1** lists these sub-catchments, their associated hydrologic/hydraulic model domain and provides the area each sub-catchment drains.

4.1.2 Rainfall Data

Rainfall information is the primary input and driver of the hydrological model. Rainfall characteristics for both historical and design events are described by:

- Rainfall depth – the depth of rainfall occurring across a catchment surface over a defined period; and
- Temporal pattern – the temporal (time varying) spatial distribution of rainfall depth at a certain time intervals over the duration of the rainfall event.

Both of these properties can vary spatially across the catchment.

The procedure for defining these properties is different for historical and design events. For historical events, the recorded hyetographs at continuous rainfall gauges provide the observed rainfall depth and temporal pattern. Where only daily read gauges are available within a catchment, significant assumptions regarding the temporal pattern may need to be made.

For design events, rainfall depths have been derived by the estimation of intensity frequency duration (IFD) design rainfall curves for the catchment by Engineers Australia. Standard procedures for derivation of these curves are outlined in AR&R (1987). AR&R (1987) also defines standard temporal patterns for use in design flood estimation.

4.1.3 Rainfall Losses and Catchment Roughness

The rainfall losses are a significant calibration parameter within the hydrologic/hydraulic model and have a major influence on peak flows and runoff volumes within the models. An initial and continuing loss model has been used in both the hydrological modelling and the direct rainfall approach to hydraulic modelling.

Initial losses describe the depth of rainfall that does not contribute to runoff in the initial part of a storm event due to interception and infiltration. These loss values help to describe the catchments antecedent conditions.

Continuing losses are the rate of rainfall that does not contribute to surface runoff once the initial loss has been satisfied, usually caused by ongoing previous surface infiltration.

The catchment and slope roughness parameters govern the speed with which the runoff will travel, influencing the hydrological response of the model. For calibration purposes, generally loss rate and catchment roughness parameters are adjusted to match observed flooding.

The catchment roughness and rainfall loss parameters adopted from the calibration process are discussed in **Section 3**.

4.2 Hydraulic Model

The overland flow regime in urban environments is generally characterised by complex varying flow paths. Road networks often convey a considerable portion of floodwaters due to the hydraulic efficiency of the impervious road surface compared with developed areas consisting of buildings and fences, which act to constrict or redistribute flows. Flow in urban environments is also conveyed through underground pipe networks that drain to downstream watercourses or bodies of water (such as the Pittwater estuary or out to sea). These drainage networks can often be the main escape route for floodwaters for some areas. Given this complex flooding environment, an integrated 1D/2D model approach is prudent for the study area.

TUFLOW/ESTRY is an integrated 1D/2D hydrodynamic model that is commonly used to analyse urban drainage systems. The following approach was applied to the TUFLOW model development:

- Surface flows were simulated within the model domain using TUFLOW's 2D unsteady flow algorithm. This simulation is informed by hydraulic roughness parameters and the model DEM, both of which were established over the study area.
- The pit and pipe drainage system was established in the model as a 1D ESTRY network. The 1D network is integrated (or linked) with the 2D model domain at the pit locations. This enables water to enter the pipe system from the 2D model (i.e. inflows into a pit) and in some cases surcharge from the pipe system to the 2D model domain.

4.2.1 Model Extents and Layout

Considerations for the hydraulic model development included:

- The physical nature of sub-catchments including land use, topography and drainage features;
- Controlling features such as embankments, culverts and bridges;
- The availability and location of calibration data;
- The level of accuracy required to meet the study's objectives; and
- Computational constraints.

As mentioned above**,** the study area has been broken into three model areas based on the similar nature of the sub-catchments within these model areas. It should also be noted that the only available calibration data for the study is contained within Model Area 3 (the 'Urban Catchments' around Mona Vale area).

Table 4-1 provides a description of each of the model areas and lists their sub-catchments and areas.

Table 4-1: Hydraulic Model Areas

Model Area		Sub-Catchments	Catchment Area	Total Hydraulic Model Area
			km^2	km^2
1	**The 'Rural' Catchments:** Characterised by predominantly natural, bushland catchments with some rural development. Natural watercourses are central to these catchments.	McCarrs Creek Cicada Glen Creek	9.9 2.3	7.6 *
2	**The 'Pittwater' Catchments:** Characterised by a number of small and steep, heavily urbanised catchments and overland flow paths that drain north into the Pittwater Estuary.	Gilwinga Drive Browns Bay BYRA Loquat Valley Fermoy Avenue Bayview Park Edwin Ward Reserve Yachtsman's Paradise	0.1 0.6 0.3 0.3 0.1 0.1 0.1 0.2	4.1 **
3	**The 'Urban' Catchments:** Consists of the larger urbanised catchments with generally flatter floodplains downstream. These catchments drain to both the Pittwater estuary and east to the Pacific Ocean.	Cahill Creek Mona Vale Main Drain Hillcrest Mona Vale Golf Course	3.4 1.1 0.4 1.4	6.9

* Western portion of McCarrs Creek Catchment is not included in hydraulic model.
** Model Area 2 also consists of small steep overland flow areas that are not considered significant sub-catchments for the purpose of this study.

A TUFLOW 2D domain model resolution of 3m was adopted for the study area. It should be noted that TUFLOW samples elevation points at the cell centres, mid-sides and corners, so a 3m cell size provides a Digital Elevation Model (DEM) resolution of approximately 1.5m. The 3m cell size was chosen for the study area as it provides the best compromise between computational efficiency and model accuracy for the study area.

4.2.2 Topography

A high resolution DEM has been developed utilising the following data:

- LiDAR data acquired in May of 2011 (acquired from LPI by RHDHV);
- Previously surveyed information acquired as part of the DHI Study (2002); and
- Detailed survey acquired by Mepsteads & Associates surveyors as part of this study (survey was completed in early 2015).

The DEM derived for this study was built using the LiDAR data as a base input to the TUFLOW model with all additional survey information incorporated using the software's Z-Shape functions. This allows model edits to be easily tracked, complexity to be added to the model over time and guarantees the editing of model cells irrespective of the grid origin and orientation (i.e. using breaklines). Where survey was difficult to collect due to dense vegetation, channel centrelines were lowered in the DEM by a small amount (up to 0.5m), to allow for extra channel capacity that is not represented in the filtered LiDAR data.

Figure 3, **Figure 4** and **Figure 5** show the resultant model DEM's for Model Areas 1, 2 and 3 respectively.

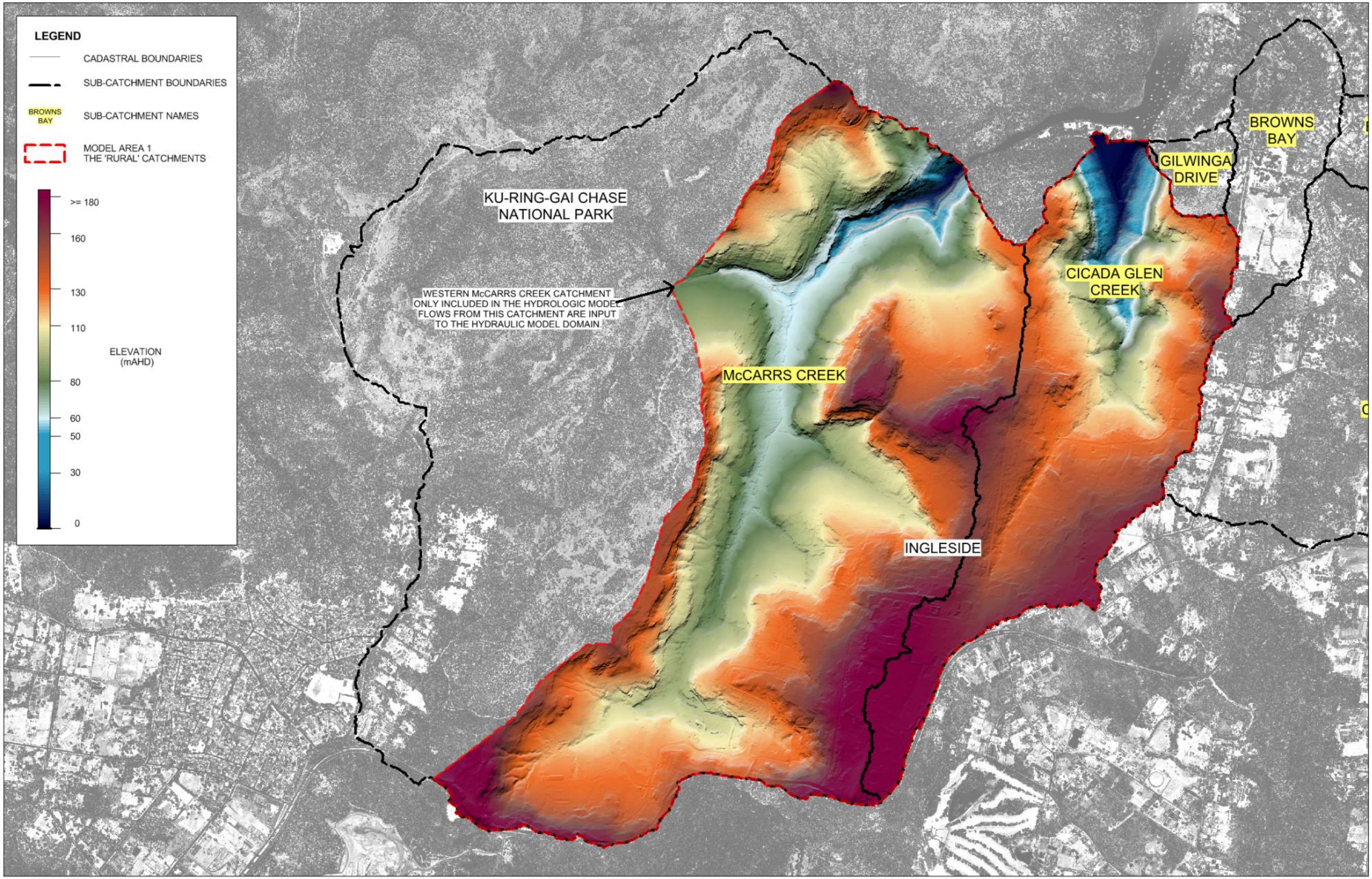

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FIGURE 3 DIGITAL ELEVATION MODEL (MODEL AREA 1)

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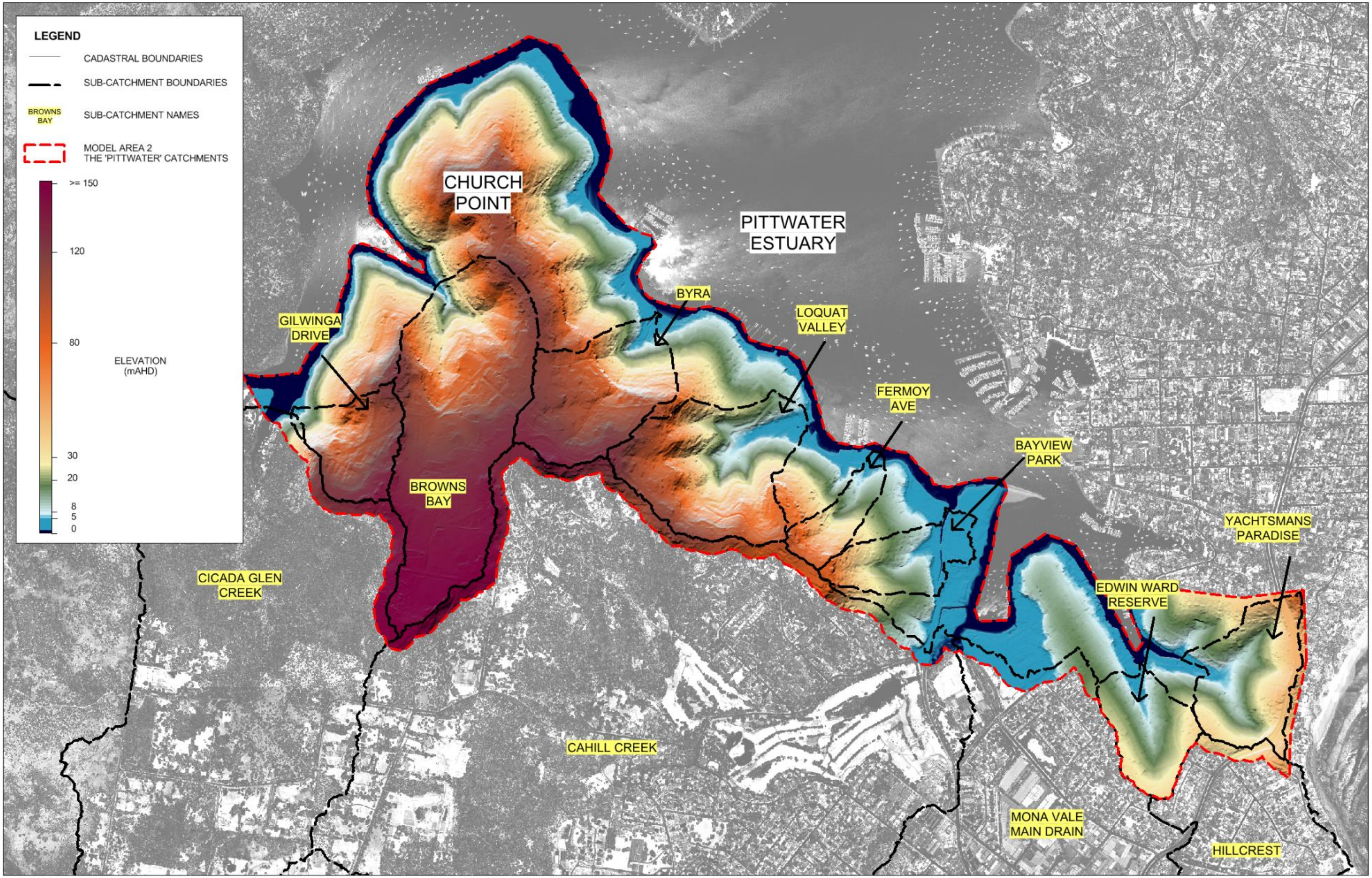

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FIGURE 4 DIGITAL ELEVATION MODEL (MODEL AREA 2)

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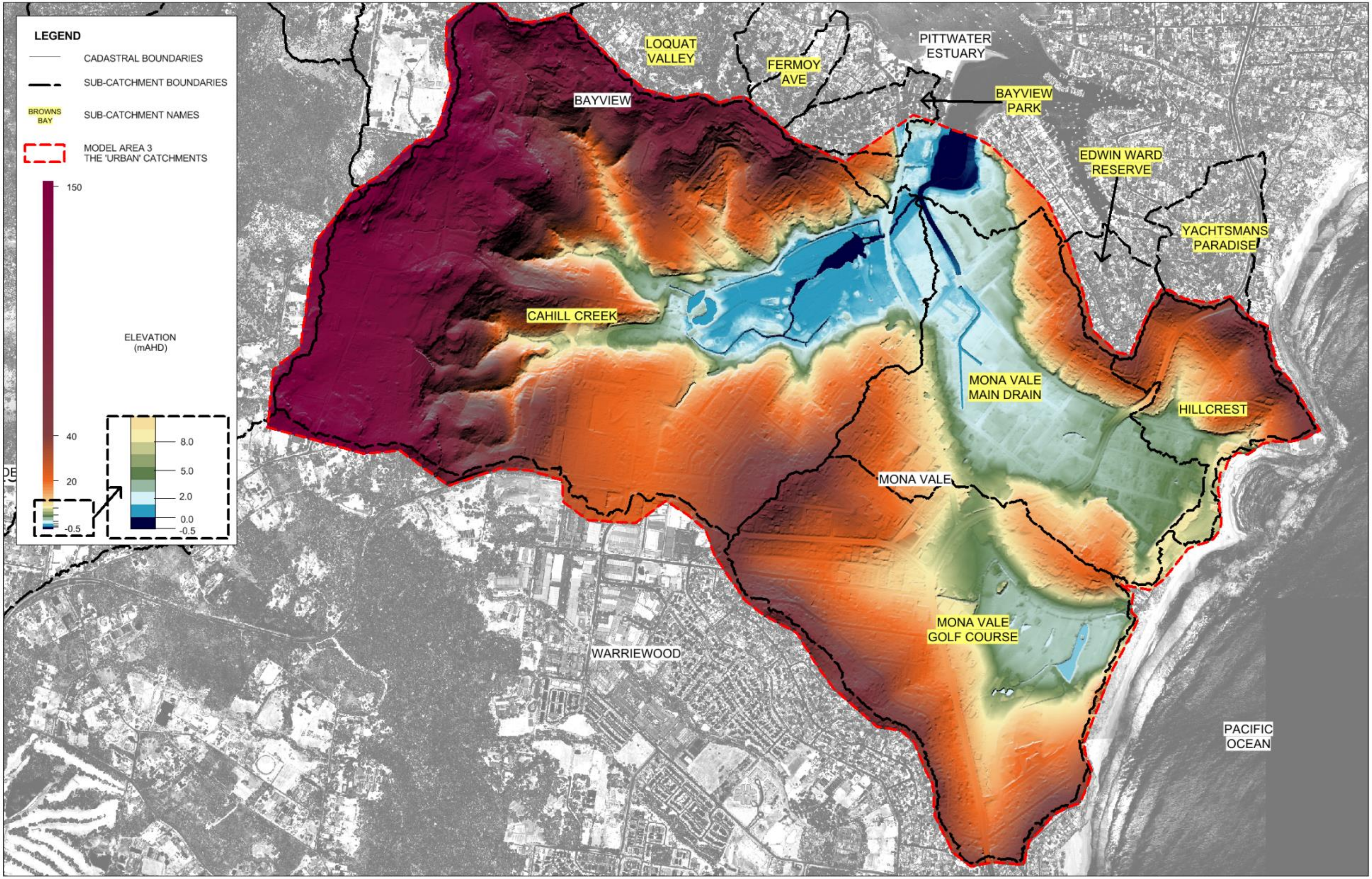

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FIGURE 5 DIGITAL ELEVATION MODEL (MODEL AREA 3)

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4.2.3 Hydraulic Roughness

The development of the hydraulic model required the assignment of different hydraulic roughness values to different areas of the model based on land-use types. Aerial photography and cadastral information have been used to delineate different land-uses (such as forested areas, cleared land, road reserve, residential allotments etc.) for modelling the variation in flow resistance.

The hydraulic roughness is one of the most important calibration parameters within the hydraulic model and has a major influence on flow routing and flood levels. The roughness values adopted from the calibration process are discussed in **Section 3**.

4.2.4 Modelling of Buildings

Both residential and industrial buildings can act to restrict overland flow, where the industrial buildings are likely to provide a greater disruption. To account for the flow disruption caused by buildings, the following assumptions were made for the hydraulic model:

- Residential dwellings were modelled using higher roughness values for the entire allotment (documented in Section **5.2.4**). This also accounts for flow disruptions associated with fences and other potential debris in residential allotments.
- Industrial buildings in the Mona Vale Main Drain catchment were excluded from the model grid (i.e. fully blocked out for flow) with independent calculations of flow input to the model to account for their excluded catchment area from the direct rainfall model.

4.2.5 Drainage System

The study requires the modelling of the trunk drainage system in each catchment. Council provided information where available on the existing drainage system. This data comprised a GIS layer of pit and pipe locations, along with surveyed details including pipe sizes, pit and pipe depths and pit inlet structures. No information was available for pipe invert levels or pit inlet levels and some local gaps in the data exist where survey of pit depths had not been possible.

Detailed survey was conducted for a number of pits and pipes in the downstream areas of each catchment as part of this study. This survey was used to verify the relative accuracy of assumptions made with the available GIS based pit and pipe data. The following assumptions were made to convert the GIS drainage information into a format reasonable for inclusion in the hydraulic model:

- Pit cover levels were assumed to be equal to the LiDAR surface level at the same location (survey of all pit cover levels in the study area was deemed inappropriate given the size of the study and its objectives).
- In areas where the depth to pipe invert was not available, depths were assumed to equal the pipe size plus an allowance for standard pipe cover (e.g. 600mm).
- These assumptions were then cross-checked against the DEM elevations to take account of any local topographic features and to maintain minimum cover levels. Assumed invert levels were also checked to maintain upstream and downstream pipe gradients, where appropriate.

For this study, the entire trunk drainage network was included in the hydraulic model, where this was defined as pipes with a larger diameter (or box culverts with a greater width) than 450mm. As the study area contains a number of locations that would drain poorly without the inclusion of the entire pipe network, some areas of the model included pipes smaller than 450mm.

The pipe network, represented as a 1D layer in the model, is dynamically linked to the 2D domain at specified pit locations for inflow and surcharge. Pit inlet capacities at each pit have been exaggerated in the model for the following reasons:

1. Only the trunk drainage system has been included in the hydraulic model therefore the exaggeration of pit inlets allows more flow to enter the trunk system in the downstream areas, where in reality it would already be in the underground network (that is, collected in upstream drainage network which is omitted from the model).

2. One of the study's objectives is to produce a map of pipe capacities within the trunk drainage system. By exaggerating the pit inlet capacities, the trunk system is assumed to be 'pipe constrained' and the true pipe capacity of each area of the trunk system can be established.

The modelled trunk drainage system, watercourses included in the modelling and the remaining drainage system (excluded from the model) are shown in **Figure 6** for the study area.

4.2.6 Boundary Conditions

The downstream model limit for each of the hydraulic model areas is as follows:

- Model Area 1 and 2 – The downstream model limit for the rural catchments and Pittwater foreshore models corresponds to the water level in the Pittwater estuary.
- Model Area 3 – Half of the 'Urban' model sub-catchments drain north to the Pittwater estuary and half drain east to the Pacific Ocean at Mona Vale Beach.

Tidal boundary conditions have been predicted for each of the calibration events using tidal constituents for the Pittwater area. For the design events, model boundaries are discussed in **Section 6.2**.

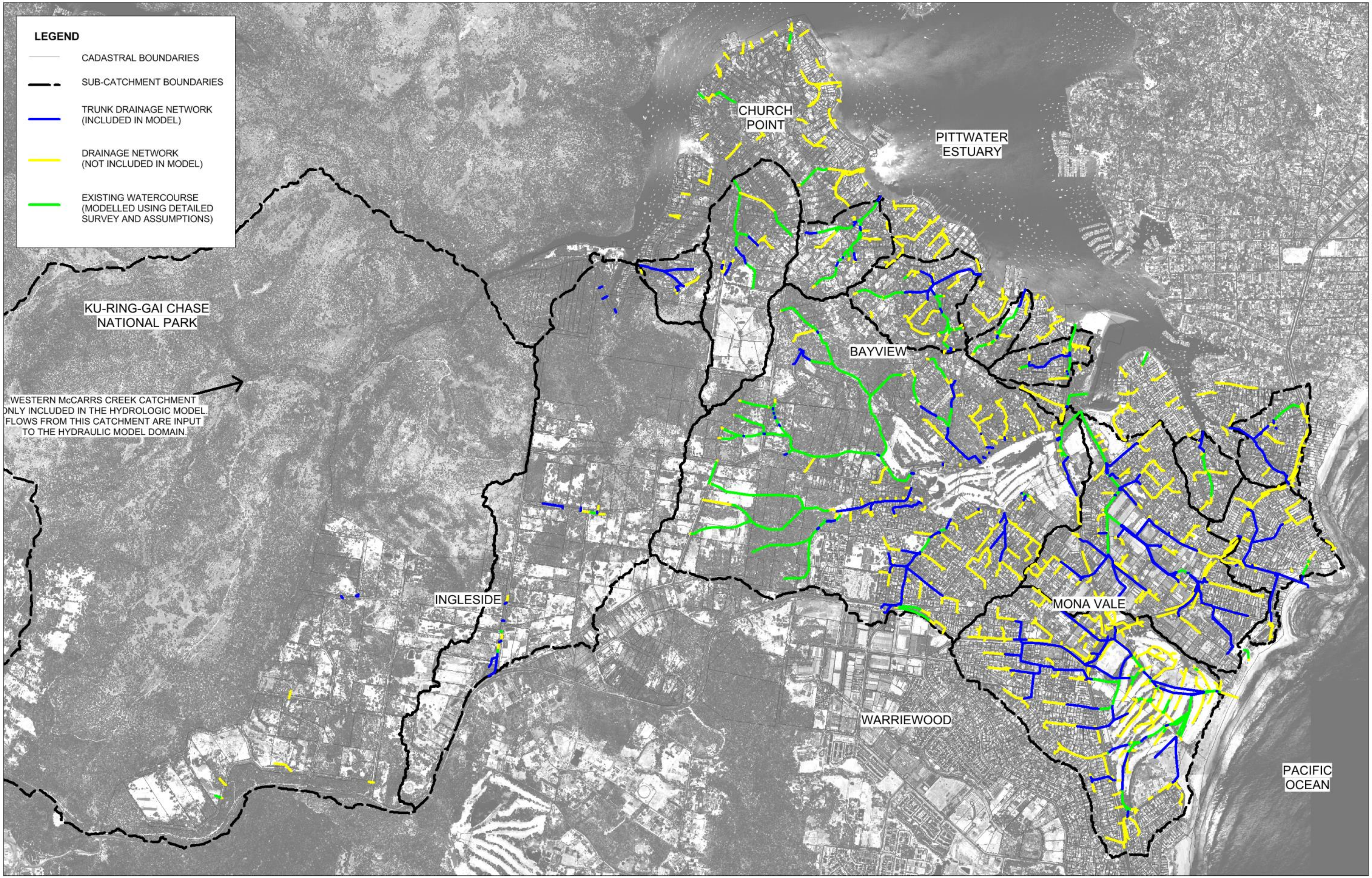

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FIGURE 6 DRAINAGE NETWORK FEATURES

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5 Hydraulic Model Calibration

5.1 Selection of Calibration Events

The selection of suitable historical events for calibration of computer models is largely dependent on available historical flood information. Ideally, the calibration and validation process should cover a range of flood magnitudes to demonstrate the suitability of a model for the range of design event magnitudes to be considered.

Significant flooding in the study area has occurred on numerous occasions, with the most severe events in recent times including 1977, 1987, 1989, 1990 and 1998. These events were documented in the Mona Vale - Bayview Flood Study undertaken by DHI in 2002.

The Manly Hydraulics Laboratory (MHL) have been operating a flow gauge on the Darley Road Tributary of Mona Vale Creek since July 2013 and a tipping bucket rainfall gauge at Mona Vale since June 1994. Since installation of the flow gauge, there has only been a number of very small events, limiting the gauge's usefulness for calibration of the hydrological models. No new historical flood events have been identified as part of this study.

Table 5-1 identifies the three main events used for calibration/verification in the DHI Study (2002).

Table 5-1: Historical Rainfall Events (DHI, 2002)

Day/Month	Year	Total Depth	Total Duration	Estimated AEP
24 October	1987	99 mm	2 hours	5%
5/6 January	1989	154 mm	24 hours	<20%
10 April	1998	65 mm	1 hour	<5%

No stream flow gauges were operational in any of the three events noted in **Table 5-1**. Surveyed flood marks for these events were gathered as part of the previous DHI Study (2002). These are presented in **Table 5-2** and are located in **Figure 7** . As all of the surveyed flood marks available lie within Model Area 3, this model has been further developed during the calibration exercise. Parameter values chosen during this calibration process have been utilised in other parts of the study area for similar land use types.

Table 5-2: Historical Flood Marks (DHI, 2002)

Street Name	Description	Year	Month	Date	Level (mAHD)
Barrenjoey Rd	Circular mark on back side fence	1998	**Apr***	**10**	**3.52**
Barrenjoey Rd	Line mark on back side fence	1987	Oct	24	3.68
Barrenjoey Rd	2cm above garage floor level	1998	**Apr***	**10**	**3.51**
Barrenjoey Rd	Top of brick foundation wall	1987	Oct	24	3.53
Parkland Rd	Ground level half way across back yard	1998	**Apr***	**10**	**1.82**
Parkland Rd	Level on tree on creek channel opposite back yard	1998	**Apr***	**10**	**1.42**
Parkland Rd	Water level on back fence	1998	**May***	**18**	**1.02**
Parkland Rd	Debris mark 1.5 brick courses up back of building	1998	**Apr***	**10**	**1.64**
Samuel St	Debris level on back fence	1989	Jan	5	5.71
Samuel St	Debris level on front wall near front door	1989	Jan	5	5.90
Old Samuel St	Debris level on garden shed	1989	Jan	5	7.54
Old Samuel St	Water level on fence between 4 & 6 Old Samuel St	1989	Jan	5	7.56
Darley St	Debris level on garage door	1998	**Apr***	**10**	**4.77**
Darley St	Debris level on side fence adjacent Apex Park	1998	**Apr***	**10**	**4.74**
Darley St	Water level in back yard near garages	1987	Oct	24	4.75

*Note: Values shown in **bold** were used for model calibration

Of the three main events presented in **Table 5-2**, the April 1998 event was chosen for hydraulic model calibration, with the verification exercise undertaken for the 1987 and 1989 events.

5.1.1 Accuracy of Flood Marks

The surveyed flood marks listed in **Table 5-2** have a relatively unknown level of accuracy. These marks were collected for the DHI study in 2002 predominately from photographs or anecdotal information of the historic events. The accuracy of this information could be affected by a number of factors such as debris marks being overstated from flood waves or misguided anecdotal information. As such, a match of +/- 150mm is considered a reasonable target for the observed and modelled flood levels in the calibration exercise.

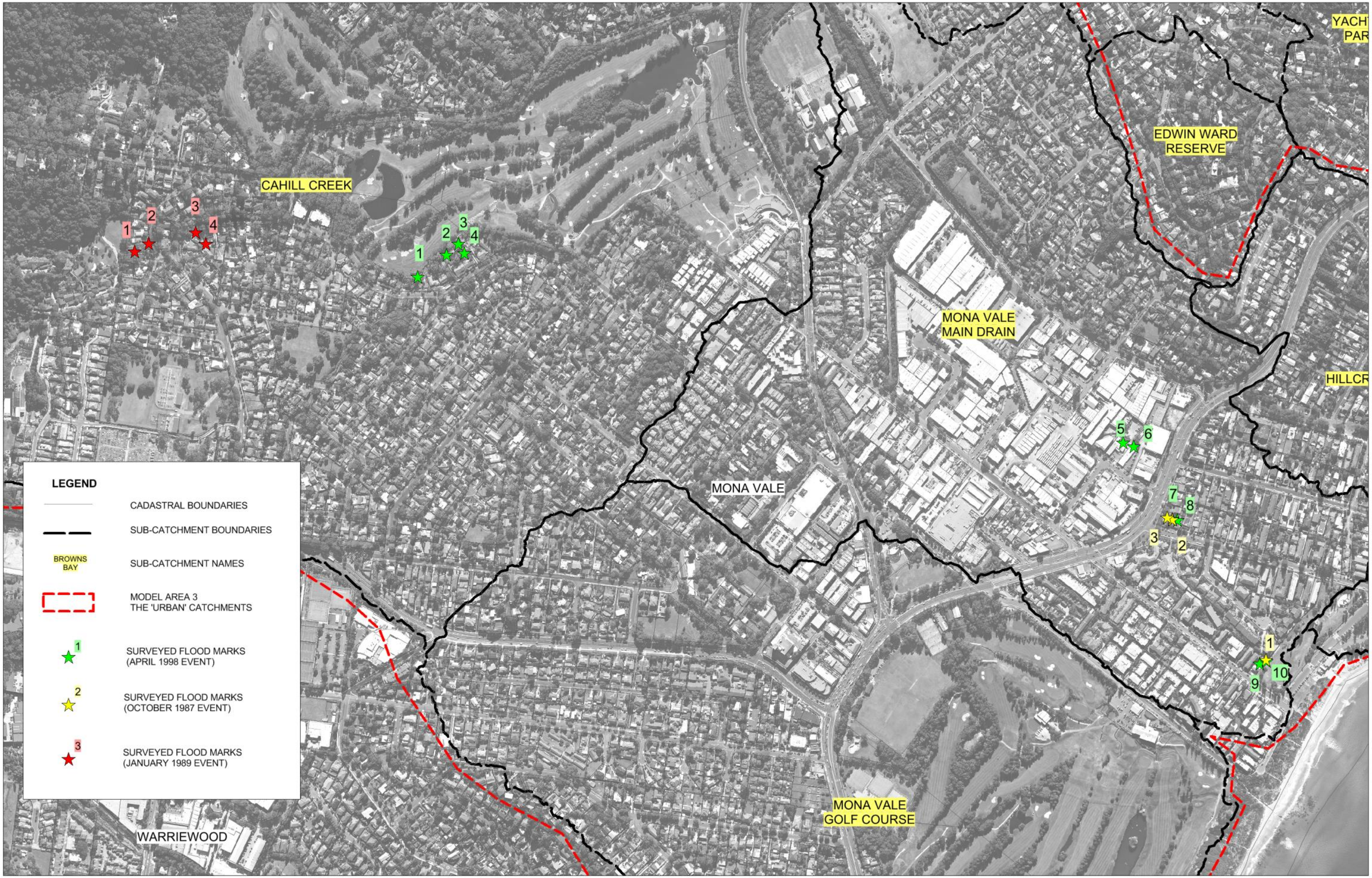

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FIGURE 7 SURVEYED FLOOD MARKS

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5.2 April 1998 Model Calibration

A calibration run was undertaken for the model using the April 1998 flood event as this event had the most rainfall and flood level data available. All data for this event was sourced from the Mona Vale – Bayview Flood Study (DHI, 2002).

5.2.1 Recorded Flood Levels

Recorded flood levels for the April 1998 flood event (provided in the DHI flood study of 2002) were located in three different areas of the floodplain. In the Cahill Creek catchment, recorded peak flood levels were available for the rear of properties on Parkland Road adjoining the Bayview Golf Course. In the Mona Vale Main Drain catchment, peak flood levels were available for the properties in Darley Street East, Seabeach Avenue and Barrenjoey Road and in the light industrial areas adjacent to Polo Avenue.

5.2.2 Rainfall Data

Several pluviometers were installed in the Pittwater local government area in 1995. The rainfall records for these stations were used for both the temporal pattern and total rainfall depth aspects of the 1998 storm event. The Warriewood STP gauge temporal pattern was applied to the eastern sub-catchments of Model Area 3 and the Middle Creek gauge temporal pattern was applied to the western sub-catchments. **Figure 8** shows the derived rainfall isohyets for the April 1998 event.

5.2.3 Downstream Boundary Conditions

No recorded tailwater data was available for the 1998 event. As such, a tidal water level boundary was predicted for the period of the April 1998 storm event using available tidal constituents for Pittwater and was applied to both the ocean outlet of the Mona Vale Golf Course catchment and the Pittwater estuary.

The predicted Pittwater time series for the April 1998 event is presented in **Plate 5-1**.

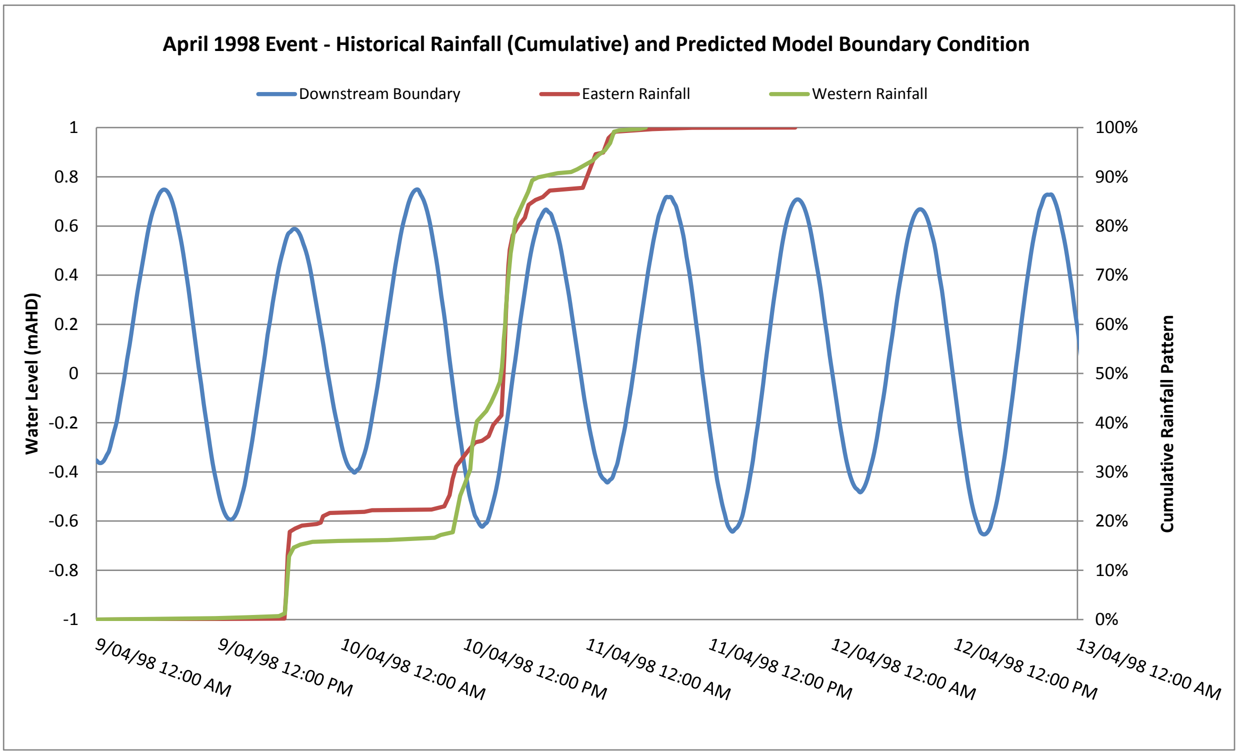

***Plate 5-1:** Recorded Rainfall Pattern and Predicted Model Boundary Condition (1998 Event)*

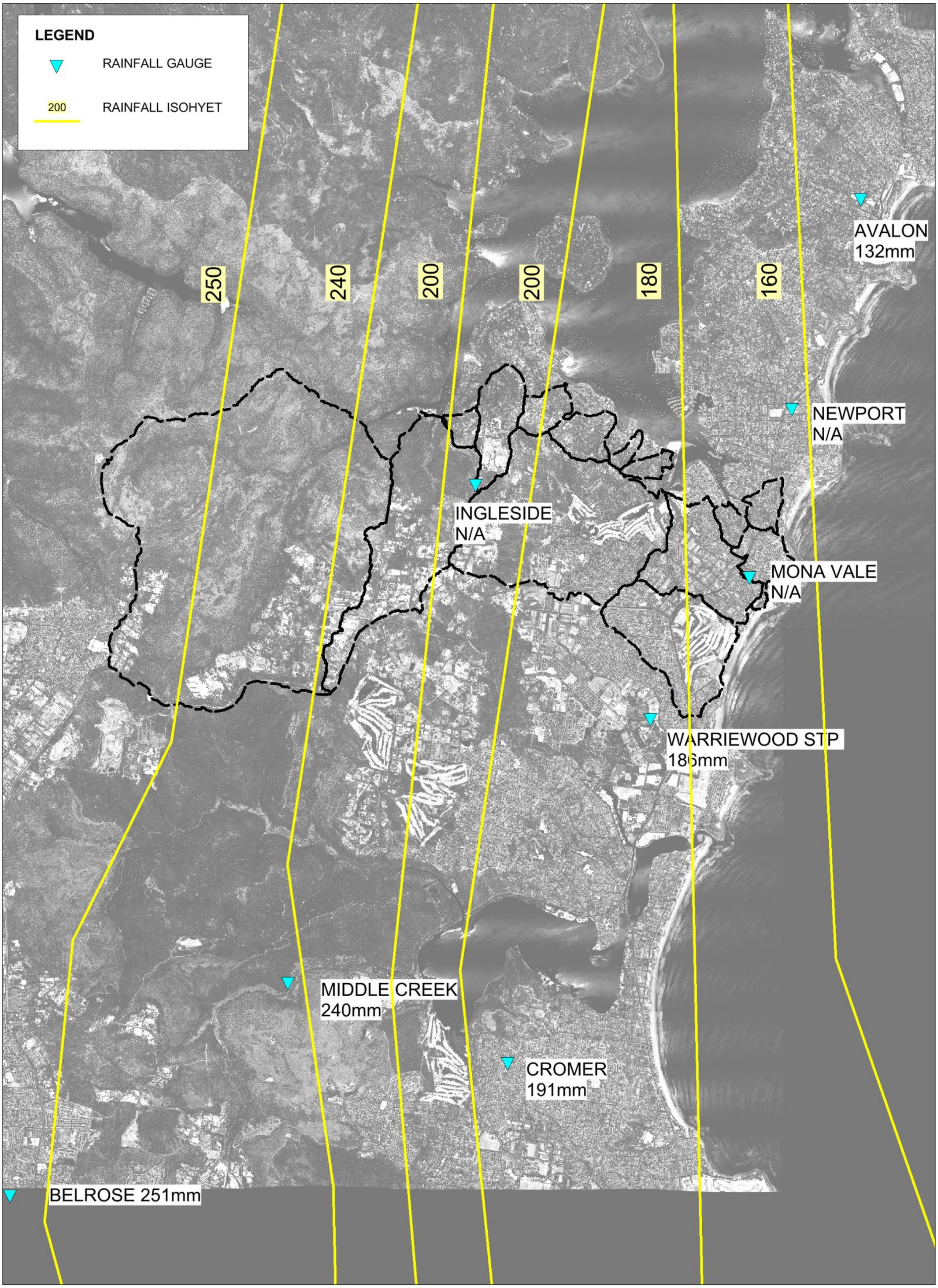

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FIGURE 8 **RAINFALL ISOHYETS (1998 EVENT)**

NORTH

0 0.75 1.5km
Approx. Scale

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5.2.4 Adopted Model Parameters

Initial estimates of rainfall losses and hydraulic roughness were assigned to different land uses in the study area following an initial site inspection. These parameters were then adjusted within reasonable limits to achieve a reasonable fit between modelled and historic flood levels. The land use areas adopted for the 1998 calibration event are shown in **Figure 9**. The associated rainfall losses and Manning's roughness values for these land use types are presented in **Table 5-3**.

Table 5-3: Model Material Types, Roughness and Rainfall Losses

Material Type		Hydraulic Roughness	Initial Loss	Continuing Loss
(-)		(n)	(mm)	(mm/hr)
1	Road Reserve	0.020	4	0.25
2	Rural Allotment	0.050	24	2.40
3	Medium Density Allotment	0.200	13	1.30
4	High Density Allotment	0.400	8.6	0.80
5	Industrial Areas	0.050	4	0.25
6	Densely Vegetated Areas	0.100	35	4.50
7	Grassed Areas	0.040	25	2.50
8	Open Water	0.020	0	0.00
9	Vegetated Allotment	0.100	31	4.10
10	Vegetated Channel	0.100	25	2.50
11	Concrete Lined Channel	0.025	1.5	0.00
12	Industrial Channel	0.100	1.5	0.00
13	Medium Density Allotment (Sandy Soil)	0.200	25	2.00
14	High Density Allotment (Sandy Soil)	0.400	20	1.50
15	Grassed Area (Sandy Soil)	0.040	30	2.50

Table 5-3 presents the adopted model parameters required to achieve calibration to the April 1998 flood event (discussed in **Section 5.2.5**). The following items are notable in the table:

- Residential allotments have the highest hydraulic roughness values (0.2 to 0.4) to account for flow disruptions caused by buildings and fences.
- Industrial areas have lower hydraulic roughness as the buildings in these parts of the floodplain have been blocked from the model DEM.
- Losses for material types 13-15 were desired based on need to reduce runoff flow volumes in the Mona Vale Main Drain and Hillcrest Catchments to match historical behaviour. The increase in rainfall losses for these material types is attributed to sandy soils in this area.

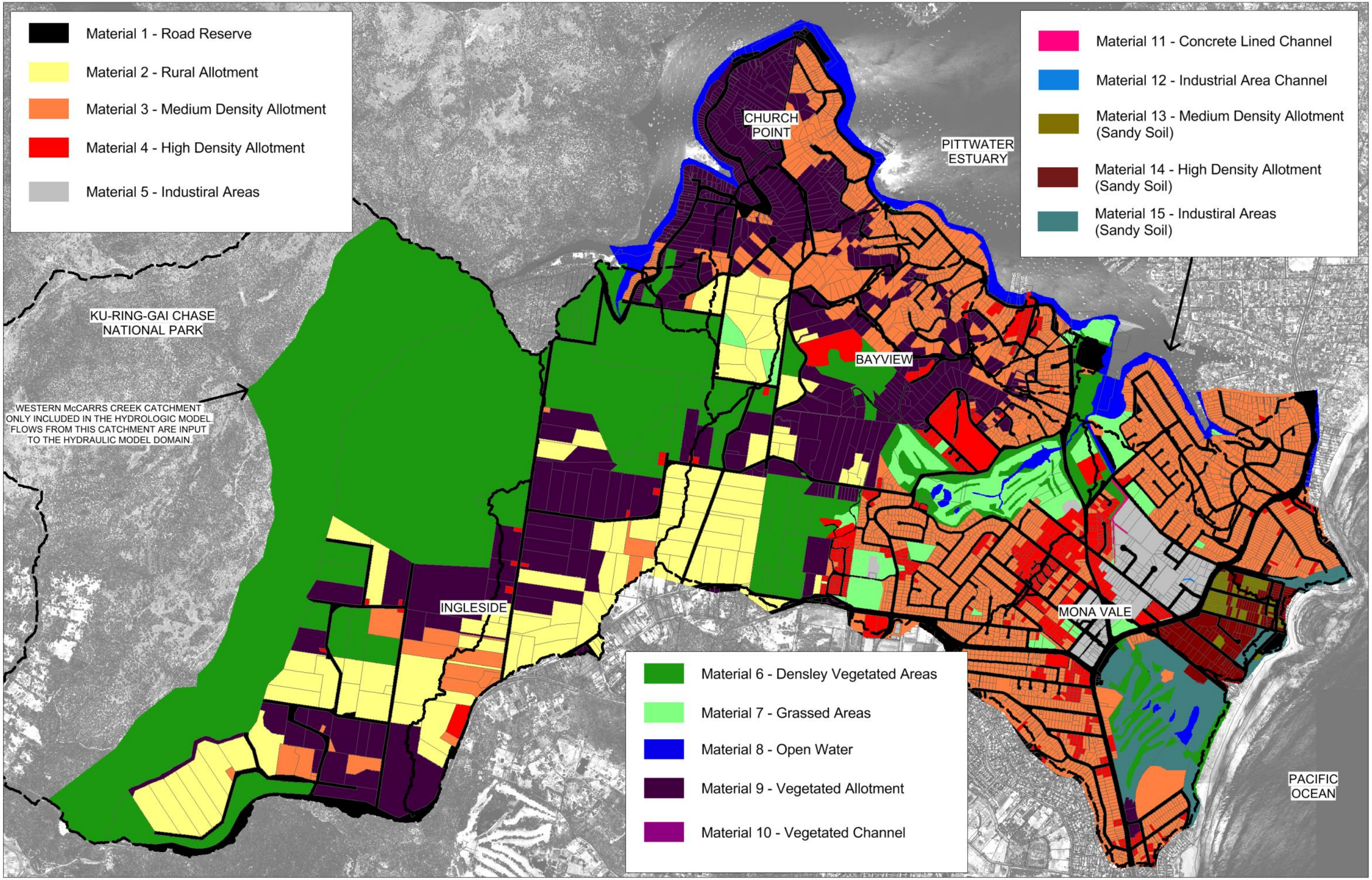

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FIGURE 9 **HYDRAULIC MODEL MATERIAL TYPES**

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0 375 750m
Approx. Scale

NORTH

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5.2.5 Observed and Simulated Flood Behaviour

Comparisons between the recorded and predicted flood levels for the April 1998 flood event are presented in **Table 5-4**.

Table 5-4: 1998 Event Calibration Results

Flood Mark	Surveyed Level (m AHD)	Modelled Level (m AHD)	Difference (m)	Comment
1	**1.42**	**1.82**	**0.40**	**Flood mark considered unreliable***
2	1.82	1.71	-0.11	
3	**1.02**	**1.70**	**0.68**	**Flood mark considered unreliable ***
4	1.64	1.69	0.05	
5	3.16	3.07	-0.09	
6	3.22	3.24	0.02	
7	3.52	3.72	0.20	
8	3.51	3.72	0.21	
9	4.77	4.81	0.04	
10	4.74	4.80	0.06	

** given the vicinity of flood marks with different level*

Table 5-4 shows that the modelled flood level at 6 of the 10 flood marks is within the 150mm target range of the surveyed level, providing a good level of confidence in the models performance. Modelled levels at flood marks 7 and 8 are both 200m above the surveyed level, which is above the target level but not unacceptable. Flood marks 1 and 3 are considered unreliable given the vicinity of other nearby recorded flood marks with differing levels.

5.3 October 1987 Model Verification

A verification run was undertaken for the model using the October 1987 flood event, as this was a regionally significant event and produced the highest recorded flood levels in the Mona Vale Main Drain catchment. All data for this event was sourced from the Mona Vale – Bayview Flood Study (DHI, 2002).

5.3.1 Recorded Flood Levels

Recorded flood levels for the October 1987 flood event (DHI, 2002) were located within the Mona Vale Main Drain catchment upstream of the industrial areas. Peak flood levels were available for properties on Barrenjoey Road and Darley Street.

5.3.2 Rainfall Data

Only one pluviometer near the study area was available for the October 1987 event, the Warriewood STP. The temporal pattern of this gauge was applied to the entire domain of the hydraulic model and rainfall totals for each of the eastern and western sub-catchments were scaled using the rainfall Isohyets shown in **Figure 10**. These were derived from rainfall depths for the event from a number of daily rainfall gauges.

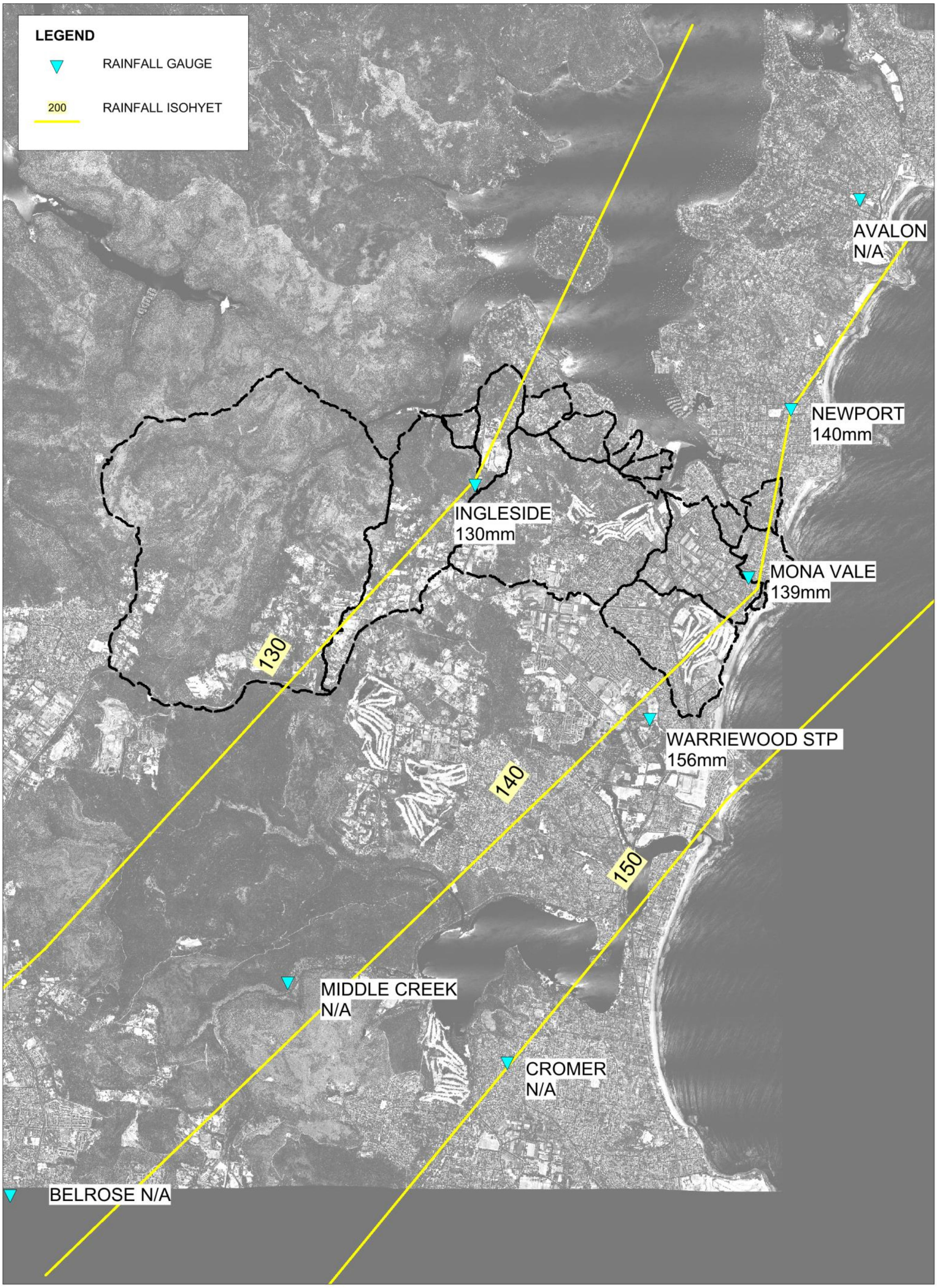

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FIGURE 10 RAINFALL ISOHYETS (1987 EVENT)

NORTH

0 0.75 1.5km
Approx. Scale

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5.3.3 Downstream Boundary Conditions

No recorded tailwater data was available for the October 1987 event. As such, a tidal water level boundary was predicted for the period of the storm event using available tidal constituents for Pittwater and was applied to both the ocean outlet of the Mona Vale Golf Course catchment and the Pittwater estuary.

The predicted Pittwater time series for the October 1987 event is presented in **Plate 5-2**.

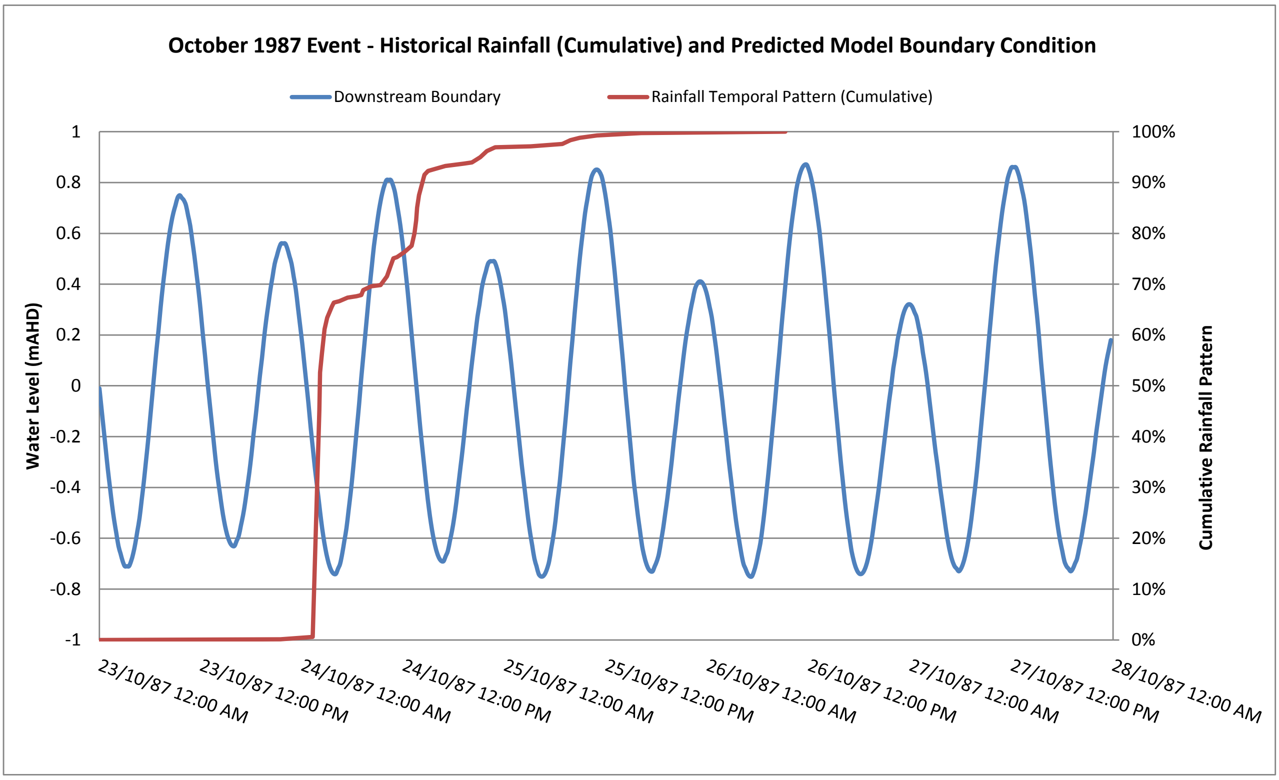

***Plate 5-2:** Recorded Rainfall Pattern and Predicted Model Boundary Condition (1987 Event)*

5.3.4 Observed and Simulated Flood Behaviour

Comparisons between the recorded and predicted flood levels for the October 1987 flood event are presented in **Table 5-5.**

Table 5-5: 1987 Event Verification Results

Flood Mark	Surveyed Level (m AHD)	Modelled Level (m AHD)	Difference (m)	Comment
1	4.75	4.80	0.05	
2	**3.68**	**3.69**	**0.01**	**Flood Marks 2 and 3 have some discrepancy – modelled levels are both controlled by Barrenjoey Rd (and are the same)**
3	3.53	3.69	0.16	

Table 5-5 shows that for the 1987 flood event the model calibrates accurately, within or very close to the 150mm target accuracy, for all 3 surveyed levels.

5.4 January 1989 Model Verification

The January 1989 flood event was also used to verify the model calibration. No pluviograph data was available for this event, however it was still deemed appropriate to include this event in the model verification exercise as flood marks were available in the Cahill Creek catchment.

5.4.1 Recorded Flood Levels

Recorded flood levels for the January 1989 flood event (DHI, 2002) were located in the Cahill Creek catchment around the Samuel Street area. Little information is available for other areas suggesting that the event may not have been significant in other areas of the catchment.

5.4.2 Rainfall Data

A similar approach to the DHI Study (2002) was taken to derive rainfall for the 1989 verification event. The temporal pattern of the October 1987 event was adopted for this event as no pluviograph data was available and the two events were similar in magnitude. The rainfall Isohyets derived from the available daily rainfall gauges are shown in **Figure 11**.

5.4.3 Downstream Boundary Conditions

No recorded tailwater data was available for the October 1989 event. As such, a tidal water level boundary was predicted for the period of the storm event using available tidal constituents for Pittwater and was applied to both the ocean outlet of the Mona Vale Golf Course catchment and the Pittwater estuary.

The predicted Pittwater time series for the January 1989 event is presented in **Plate 5-3.**

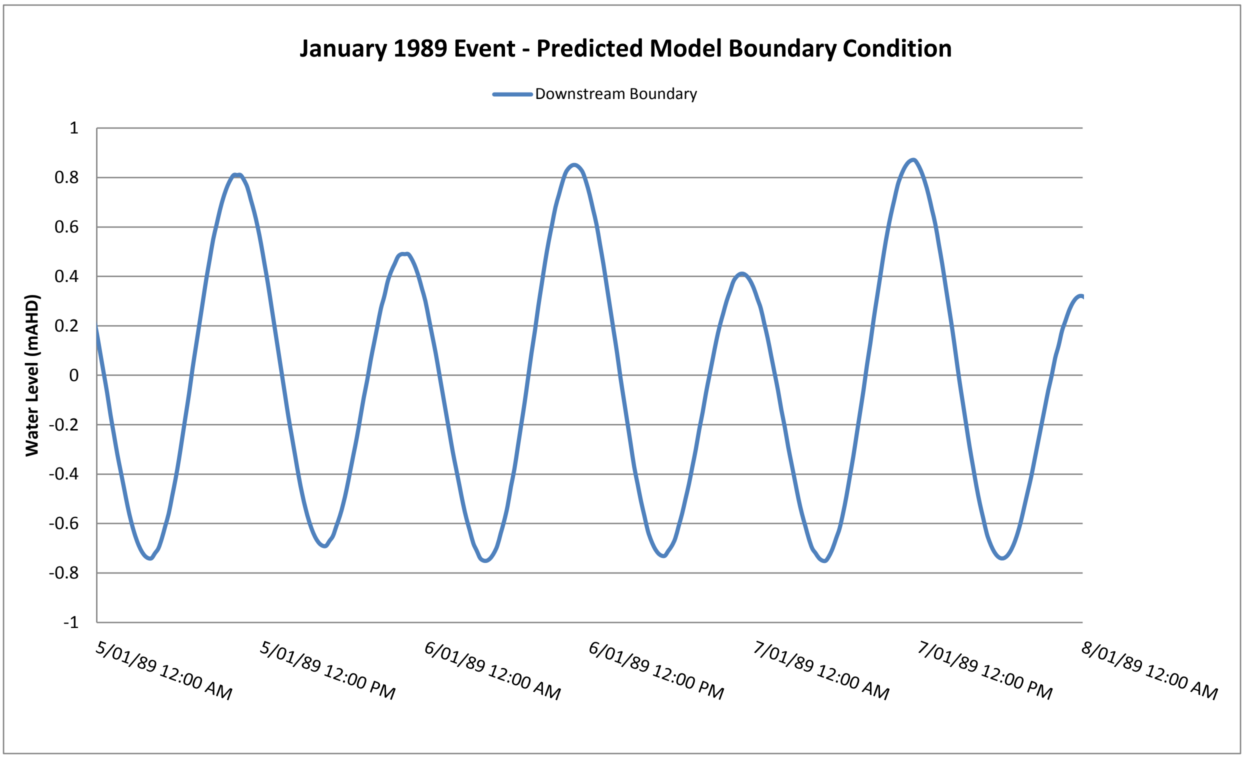

***Plate 5-3** Predicted Model Boundary Condition (1989 Event)*

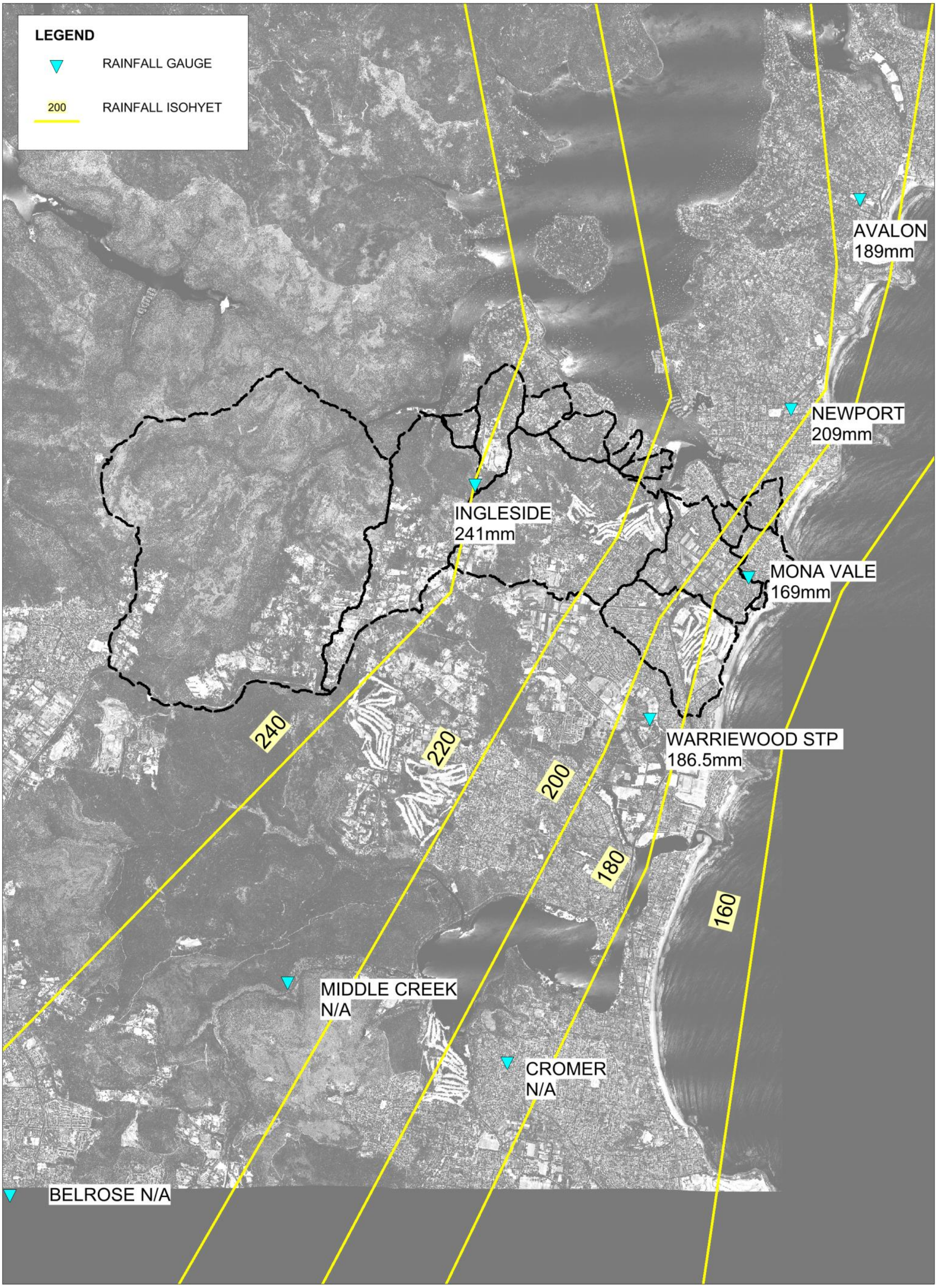

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FIGURE 11 **RAINFALL ISOHYETS (1989 EVENT)**

NORTH

0 0.75 1.5km
Approx. Scale

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5.4.4 Observed and Simulated Flood Behaviour

Comparisons between the recorded and predicted flood levels for the January 1989 flood event are presented in **Table 5-6**.

Table 5-6: 1989 Event Verification Results

Flood Mark	Surveyed Level (m AHD)	Modelled Level (m AHD)	Difference (m)	Comment
1	**7.54**	**8.23**	**0.69**	**Water level hard to match due to steep flood gradient in the model.**
2	7.56	7.59	0.03	
3	5.90	6.01	0.11	
4	5.71	5.77	0.06	

Table 5-6 shows that for the 1989 flood event the model calibrates accurately, within or very close to the 150mm target accuracy, for 3 of the surveyed levels (flood marks 2 – 4). Flood mark 1 is considered unreliable as the surveyed flood level is lower than flood mark 2, which is downstream of flood mark 1.

5.5 Calibration Summary

Hydraulic model calibration and verification has been achieved against a range of surveyed / observed flood marks across 3 historic flood events. The hydraulic models are considered to accurately representing the physical and hydraulic nature of the catchments and will provide a reasonable tool for estimating flooding in the design events. As historical flood data is not available for all catchments in the study area, sensitivity analysis of model parameters enables the model limitations to be understood. All sensitivity analyses carried out for this study are document in **Section 7**.

6 Design Event Modelling

6.1 Design Rainfall

Design rainfall parameters are derived from standard procedures defined in AR&R (1987) which are based on statistical analysis of recorded rainfall data across Australia. The derivation of location specific design rainfall parameters (e.g. rainfall depth and temporal pattern) for the Study Area is presented below.

6.1.1 Rainfall Depths

Design rainfall depths are based on the generation of intensity-frequency-duration (IFD) design rainfall curves utilising the procedures outlined in AR&R (1987). These curves provide rainfall depths for various design magnitudes (up to the 0.2% AEP) and for durations from 5 minutes to 72 hours.

The Probable Maximum Precipitation (PMP) is used in deriving the Probable Maximum Flood (PMF) event. The theoretical definition of the PMP is "the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location at a certain time of year" (AR&R, 1987). The ARI of a PMP/PMF event ranges between 10^4 and 10^7 years and is beyond the "credible limit of extrapolation". That is, it is not possible to use rainfall depths determined for the more frequent events (100 year ARI and less) to extrapolate to the PMP. The PMP has been estimated using the Generalised Short Duration Method (GSDM) derived by the Bureau of Meteorology (2003).

A range of storm durations were modelled in order to identify the critical storm duration for design event flooding in the catchment. Design durations considered included the 0.25-hour, 0.5-hour, 0.75-hour, 1-hour, 1.5-hour, 2-hour, 3-hour, 4.5-hour, 6-hour and 9-hour durations.

Table 6-1 shows an excerpt of the average design rainfall intensities based on AR&R (1987) adopted for the modelled events and **Table 6-2** shows the adopted rainfall intensities for the PMF event.

Table 6-1: Average Design Rainfall Intensities (mm/hr) - AR&R 1987

Duration (hours)	Design Event Frequency				
	20% AEP	5% AEP	1% AEP	0.5% AEP	0.2% AEP
0.25	108.2	140.3	182.2	200.6	225.3
0.5	78.2	102.7	134.8	149.0	168.1
0.75	63.5	84.1	111.2	123.2	139.4
1	54.5	72.5	96.4	107.0	121.4
1.5	43.0	57.1	75.7	83.9	95.1
2	36.2	48.0	63.5	70.4	79.7
3	28.4	37.5	49.5	54.8	61.9
4.5	22.2	29.2	38.5	42.6	48.1
6	18.7	24.5	32.2	35.6	40.2
9	14.6	19.1	25.1	27.7	31.2

Table 6-2: Design Rainfall Intensities – PMF Event (mm/hr) – GSDM BoM 2003

Duration (hours)	0.25	0.5	0.75	1	1.5	2	2.5	3	4	5	6
Rainfall Intensity (mm/hr)	600	440	375	330	280	250	220	200	175	150	135

Areal Reduction Factor

The areal reduction factor takes into account the unlikelihood that larger catchments will experience rainfall of the same design intensity (eg 1% AEP) over the entire area. Areal reduction factors typically apply to catchments significantly larger than those considered in this Flood Study and no reduction factor is required even for the largest catchment in the study area (9.9km^2). This is confirmed from the historic catchment rainfall events which showed reasonably intense rainfall occurring over the entire catchment.

6.1.2 Temporal Patterns

The IFD data presented in **Table 6-1** provides for the average intensity that occurs over a given storm duration. Temporal patterns are required to define what percentage of the total rainfall depth occurs over a given time interval throughout the storm duration. The temporal patterns adopted in the current study are based on the standard patterns presented in AR&R (1987).

The same temporal pattern has been applied across the whole catchment. This assumes that the design rainfall occurs simultaneously across each of the modelled sub-catchments. The direction of a storm and relative timing of rainfall across the catchment may be determined for historical events if sufficient data exists, however, from a design perspective for catchments of this size the same pattern across the catchment is generally adopted.

6.1.3 Rainfall Losses

The rainfall losses adopted for the design floods were the same as those used for model calibration and verification. Refer to **Table 5-3** of **Section 5** for the rainfall losses applied in model calibration.

6.2 Design Boundary Conditions

The downstream model limit for each of the hydraulic model areas is as follows:

- Model Area 1 and 2 – The downstream model limit for the rural catchments and Pittwater foreshore models corresponds to the water level in the Pittwater estuary.
- Model Area 3 – Half of the 'Urban' model sub-catchments drain north to the Pittwater Estuary and half drain east to the Pacific Ocean at Mona Vale Beach.

Each of the two boundary levels have been derived through a review of studies completed in the catchment.

6.2.1 Ocean Water Levels

The *Coastal Risk Management Guide* (DECCW, 2010a) is an authoritative source of design ocean water level information for NSW. DECCW (2010a) estimated that the 100 year ARI still water level (excluding wave setup) offshore of the Newcastle to Wollongong area (and thus including offshore of Pittwater LGA) was 1.44m AHD at present. Design water levels were also given for other AEP events (0.02%. 0.05%, 0.1%, 1%, 10% and 50%).

At a shoreline where there are breaking waves offshore, wave setup can increase still water levels. These levels are not specifically quantified in the latest coastline hazard study for Pittwater LGA (WorleyParsons, 2012), and have therefore been estimated from first principles.

Wave setup at a shoreline is typically about 15% of the breaking significant wave height. Based on Shand et al (2011), the 100 year ARI significant wave height offshore of Sydney for a 6 hour duration (a suitable duration to use to have the likelihood of coinciding with high tide) is 8.0m. Approximating this as a breaking wave height, this gives wave setup as 15% of this or 1.2m, and thus a total 100 year ARI water level including wave setup of 2.6m AHD could be assumed.

In the *Flood Risk Management Guide: incorporating sea level rise benchmarks in flood risk assessments* (DECCW, 2010b), it was noted that a conservative assumption for the 100 year ARI elevated water level at the ocean boundary for a catchment that drains directly to the ocean would be 2.6m AHD (that is, including wave setup effects). However, they noted that detailed site-specific analyses of elevated water levels at estuary entrances was appropriate, and may provide a potentially lower (less conservative) water level.

It is therefore reiterated that tailwater levels in the order of 2.6m AHD can only be potentially realised for stormwater outlets that discharge at back beach areas, which are landward of the surf zone. If outlets have a finite depth of water located seaward of the outlet in the design event, the magnitude of the water level including wave setup would be smaller than 2.6m AHD. It is also important to note that the 2.6m AHD water level including wave setup level does <u>not</u> propagate into the Pittwater waterway.

6.2.2 Estuary Water Levels

In the Pittwater estuary, there are variances in local water levels compared to the ocean, mainly due to local wind setup effects. Wave setup is also significantly lower in Pittwater estuary compared to offshore due to lower wave heights. There have been numerous studies investigating water levels in Pittwater estuary since 1991, and the latest study is Cardno (2011). Cardno (2011) determined design water levels at 37 locations around the Pittwater estuary (see **Figure A1** of **Attachment A**).

With reference to **Figure A1** and **Table A1**, locations 16 through 21 are within the study area. Given these locations, the Fork Junction on McCarrs Creek wind setup was applied to the estuary model boundaries as this gives the greatest possible wind setup in the study area.

6.2.3 Design Event Peak Levels

A tidal boundary was applied to the model with a peak level for each event as presented in **Table 6-2.**

Table 6-3: Peak Tailwater Levels for Design Events

Event	Pittwater Estuary Peak Level	Ocean Boundary Peak Level
	(mAHD)	(mAHD)
20% AEP Event	1.36	1.90
10% AEP Event	1.40	2.10
5% AEP Event	1.43	2.25
2% AEP Event	1.44	2.45
1% AEP Event	1.50	2.60
0.5% AEP Event	1.55	2.75
0.2% AEP Event	1.60	3.00
PMF Event	1.75	3.25

6.3 Design Results

6.3.1 Critical Storm Durations

A range of design event durations were simulated to determine the critical duration for flooding throughout the study area. Generally, shorter duration events are critical in the upstream, smaller and steeper sub-catchment areas. Longer duration events are critical in the lower catchment areas where storage effects become evident. The critical storm durations for the 1% AEP and PMF events are presented in **Figures 12 and 13**.

The design flood results are presented in a flood mapping series in **Appendix A**, which is comprised of an envelope of the critical storm shown durations in **Table 6-4**.

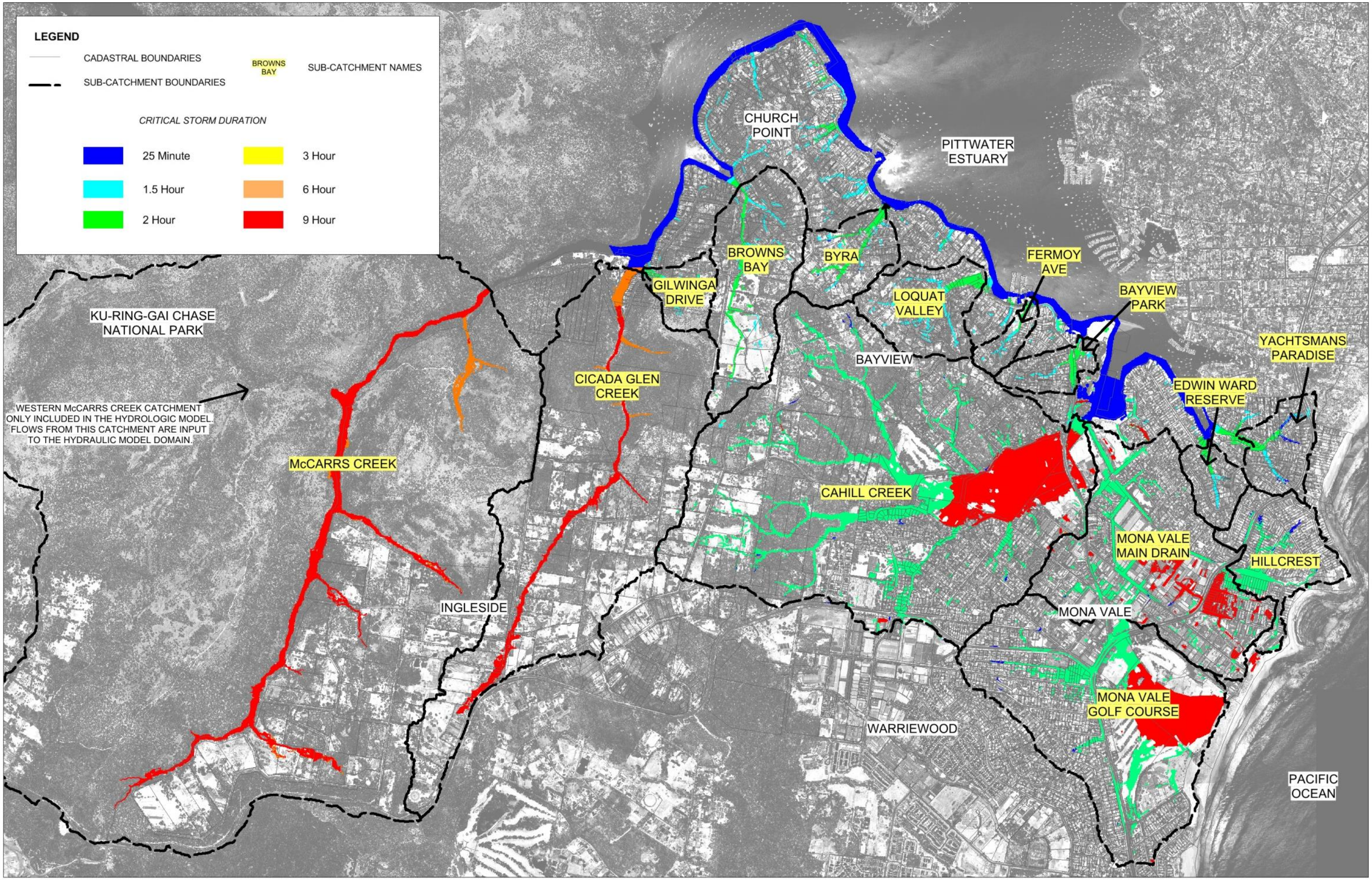

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FIGURE 12 **CRITICAL STORM DURATIONS (1% AEP EVENT)**

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Approx. Scale

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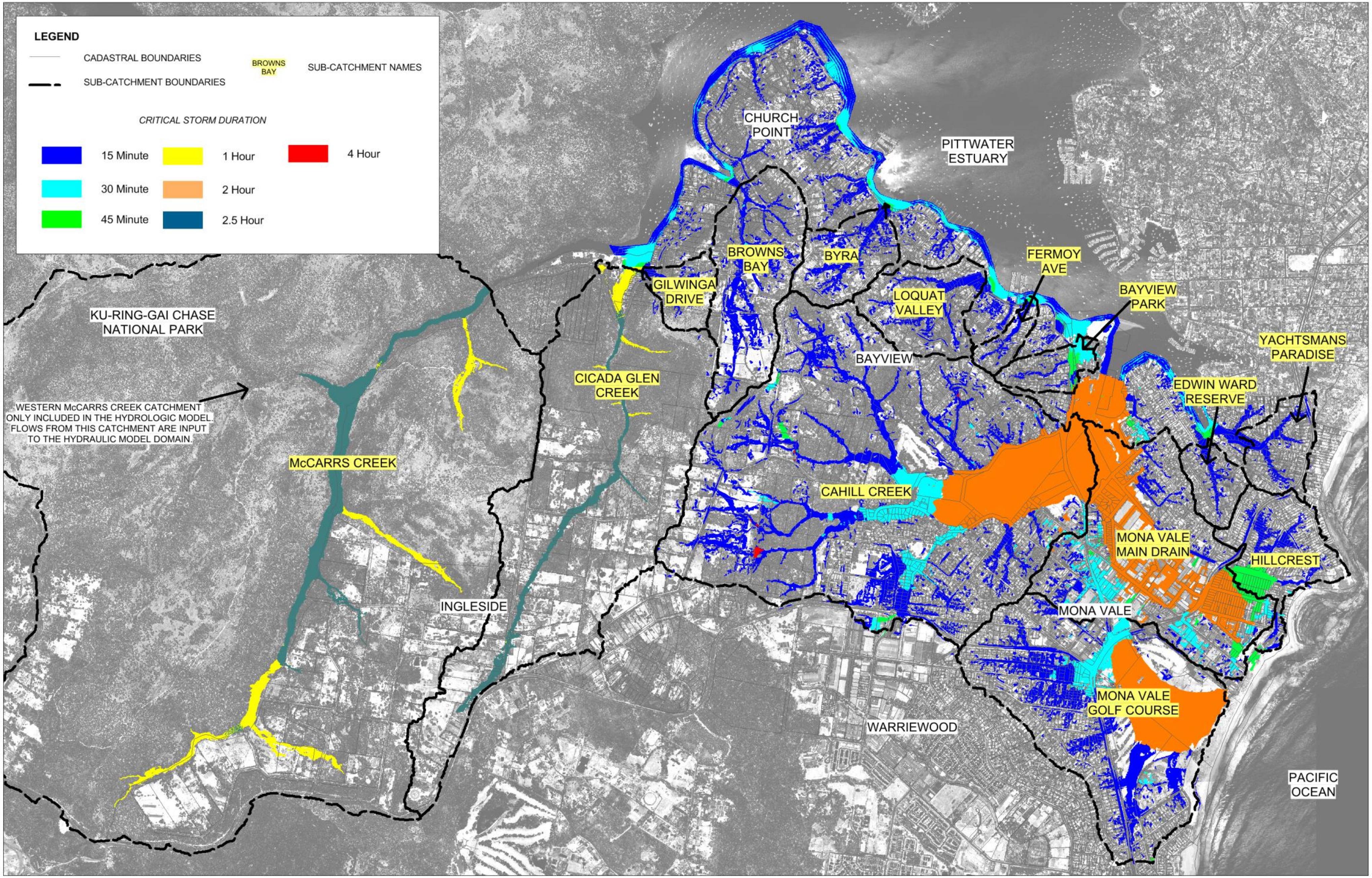

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FIGURE 12 **CRITICAL STORM DURATIONS (PMF EVENT)**

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Table 6-4: Critical Storm Durations

Model Area	Critical Storm Durations
Model Area 1 – The 'Rural' Catchments	• 6 hour • 9 hour
Model Area 2 – The 'Pittwater' Catchments	• 25 minute • 1 hour • 1.5 hour • 2 hour • 3 hour
Model Area 3 – The 'Urban' Catchments	• 25 minute • 1 hour • 2 hour • 6 hour • 9hour

6.3.2 Peak Flood Depths

A summary of peak flood depths at key locations is shown in **Table 6-5** through **Table 6-7** below, and the placement of these locations is shown in **Figures 14** to **16**. Estimated peak flood depths and flood level labels are presented on **Figures A1** through **A8**. The values reported below are the maximum values from an envelope of scenarios as described in **Section 6.3.1**.

Table 6-5: Peak Flood Depths – Model Area 1 – The 'Rural' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 1 – The 'Rural' Catchments – Peak Flood Depths (m)									
1	West Wirreanda Rd	0.85	0.89	0.93	0.96	0.99	1.03	1.06	1.62
2	East Wirreanda Rd	1.60	1.64	1.69	1.74	1.78	1.82	1.81	2.51
3	Wirreanda Rd North	1.20	1.30	1.42	1.52	1.60	1.69	1.78	3.16
4	Wirreanda Rd North	0.69	0.74	0.80	0.85	0.90	0.94	1.04	1.69
5	McCarrs Creek Rd	1.43	1.55	1.67	1.76	1.84	1.92	2.06	3.94
6	McCarrs Creek Rd	2.50	2.66	2.84	3.00	3.14	3.27	3.43	5.59
7	McCarrs Creek Rd	0.58	0.64	0.66	0.71	0.75	0.79	0.90	1.56
8	McCarrs Creek	1.02	1.13	1.25	1.37	1.47	1.57	1.53	3.42
9	Sophie Avenue	0.11	0.12	0.13	0.14	0.15	0.16	0.16	0.29
10	Chiltern Road	0.10	0.11	0.12	0.14	0.15	0.16	0.17	0.34
11	Glen Cicada Creek	1.28	1.31	1.35	1.38	1.41	1.44	1.54	1.90
12	Cicada Glen Rd	1.72	1.76	1.80	1.83	1.86	1.88	2.14	2.40
13	McCarrs Creek Rd	N/A	N/A	0.06	0.11	0.14	0.17	0.22	0.79

Table 6-6: Peak Flood Depths – Model Area 2 – The 'Pittwater' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
	Model Area 2 – The 'Pittwater' Catchments – Peak Flood Depths (m)								
1	McCarrs Creek Rd	0.14	0.15	0.17	0.21	0.22	0.24	0.28	0.41
2	Barcoola Place	0.35	0.37	0.40	0.42	0.42	0.43	0.47	0.52
3	Gilwinga Drive	0.11	0.16	0.22	0.24	0.29	0.32	0.34	0.48
4	McCarrs Creek Rd	0.12	0.16	0.20	0.24	0.26	0.29	0.31	0.67
5	McCarrs Creek Rd	N/A	0.06	0.07	0.08	0.09	0.10	0.11	0.15
6	Kananook Avenue	0.20	0.23	0.27	0.32	0.30	0.31	0.36	0.48
7	Kananook Avenue	0.11	0.13	0.15	0.18	0.18	0.20	0.23	0.28
8	Pittwater Rd	0.06	0.07	0.08	0.10	0.10	0.11	0.12	0.17
9	Pittwater Rd	0.15	0.18	0.21	0.24	0.25	0.28	0.31	0.41
10	Clive Crescent	0.07	0.08	0.09	0.09	0.11	0.12	0.11	0.17
11	Jendi Avenue	0.22	0.24	0.27	0.28	0.30	0.31	0.32	0.40
12	Jendi Avenue	0.08	0.09	0.13	0.15	0.16	0.17	0.19	0.29
13	Loquat Valley Rd	0.11	0.14	0.20	0.21	0.25	0.28	0.27	0.41
14	Kookaburra Close	N/A	0.15	0.16	0.17	0.16	0.17	0.17	0.20
15	Pittwater Rd	N/A	N/A	0.10	0.15	0.17	0.20	0.24	0.45
16	Pittwater Rd	0.08	0.08	0.09	0.10	0.11	0.11	0.12	0.16
17	Pittwater Rd	0.09	0.09	0.09	0.12	0.10	0.10	0.12	0.23
18	Gerroa Avenue	0.12	0.13	0.15	0.15	0.18	0.19	0.21	0.32
19	Pittwater Rd	0.26	0.33	0.41	0.48	0.54	0.58	0.64	0.95
20	Pittwater Rd	0.18	0.18	0.19	0.20	0.21	0.21	0.22	0.27
21	The Esplande	0.35	0.36	0.38	0.38	0.39	0.40	0.41	0.50
22	Rednal Street	0.71	0.36	0.39	0.41	0.44	0.46	1.70	0.79
23	Crescent Rd	0.23	0.24	0.25	0.27	0.27	0.27	0.30	0.44
24	Suncrest Avenue	0.17	0.18	0.19	0.20	0.22	0.23	0.25	0.32
25	Crescent Rd	0.18	0.19	0.22	0.24	0.26	0.29	0.31	0.46
26	Yachtsmans Paradise	0.68	0.74	0.81	0.86	0.89	0.94	0.99	1.33
27	Crescent Rd	0.16	0.18	0.20	0.21	0.23	0.24	0.26	0.40

Table 6-7: Peak Flood Depths – Model Area 3 – The 'Urban' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 3 – The Urban Catchments – Peak Flood Depths (m)									
1	Peninsula Gdns Retirement	1.92	1.99	2.08	2.19	2.30	2.41	2.52	3.27
2	Old Samuel Street	0.36	0.48	0.60	0.74	0.88	1.01	1.15	2.02
3	Samuel Street	0.17	0.24	0.31	0.40	0.48	0.56	0.65	1.30
4	Parkland Road	0.18	0.26	0.35	0.46	0.57	0.65	0.78	1.54
5	Cabbage Tree Road	0.45	0.53	0.59	0.68	0.79	0.79	0.90	1.64
6	Annam Road	0.34	0.38	0.40	0.42	0.45	0.46	0.49	0.74
7	Annam Road	0.14	0.17	0.19	0.20	0.21	0.23	0.24	0.40
8	Annam Road	0.15	0.19	0.24	0.30	0.36	0.40	0.46	0.93
9	Cabbage Tree Road	0.07	0.18	0.27	0.39	0.51	0.62	0.80	1.72
10	Bayview Golf Course	0.88	0.99	1.08	1.20	1.32	1.46	1.64	2.54
11	Mona Vale Road	0.20	0.21	0.23	0.24	0.25	0.26	0.27	0.46
12	Samuel Street	0.38	0.43	0.46	0.49	0.51	0.52	0.54	0.75
13	Marie Crescent	0.81	0.95	1.08	1.22	1.34	1.43	1.56	2.37
14	Siobhan Place	0.28	0.29	0.34	0.53	0.63	0.72	0.82	1.51
15	Parkland Road	0.09	0.13	0.20	0.28	0.35	0.41	0.47	1.08
16	Waratah Street	0.08	0.09	0.10	0.11	0.11	0.11	0.12	0.27
17	Waratah Street	0.13	0.14	0.15	0.16	0.17	0.17	0.18	0.29
18	Cnr Maxwell St & Parkland Road	0.26	0.30	0.35	0.38	0.41	0.48	0.52	0.79
19	Wilmette Place	0.64	0.69	0.74	0.78	0.81	0.84	0.87	1.52
20	Grandview Parade	0.19	0.20	0.21	0.21	0.22	0.22	0.23	0.36
21	Orana Road	0.17	0.18	0.20	0.22	0.26	0.28	0.30	0.45
22	Bassett Street	0.33	0.41	0.51	0.61	0.69	0.75	0.83	1.33
23	Surfview Road	0.12	0.13	0.15	0.17	0.18	0.19	0.22	0.38
24	Seabeach Avenue	0.19	0.23	0.35	0.45	0.55	0.63	0.76	1.19
25	Heath Street	0.28	0.29	0.37	0.47	0.57	0.67	0.80	1.23
26	Polo Avenue	0.21	0.24	0.28	0.33	0.35	0.37	0.47	1.24

27	Tengah Crescent	0.47	0.56	0.68	0.76	0.82	0.86	0.92	1.70
28	Bassett Street	0.17	0.19	0.22	0.25	0.33	0.39	0.52	1.62
29	Mona Street	0.43	0.44	0.45	0.48	0.56	0.63	0.77	1.86
30	Barrenjoey Road	0.66	0.68	0.70	0.71	0.72	0.72	0.74	0.89
31	Barrenjoey Road	0.52	0.53	0.55	0.56	0.57	0.59	0.60	0.72
32	Brinawa Street	0.13	0.14	0.15	0.16	0.17	0.19	0.21	0.45
33	Brinawa Street	0.46	0.50	0.54	0.58	0.62	0.65	0.69	1.02
34	Vineyard Street	0.15	0.16	0.17	0.18	0.19	0.21	0.22	0.34
35	Cnr Mona Vale Rd & Pittwater Rd	0.32	0.34	0.37	0.39	0.41	0.44	0.47	0.64
36	Cnr Rowan St & Pittwater Rd	0.21	0.24	0.27	0.30	0.32	0.36	0.39	0.65
37	Pittwater Road	0.41	0.42	0.44	0.45	0.46	0.49	0.50	0.58
38	Mona Vale Golf Course	0.75	0.88	1.05	1.27	1.43	1.60	1.81	2.93

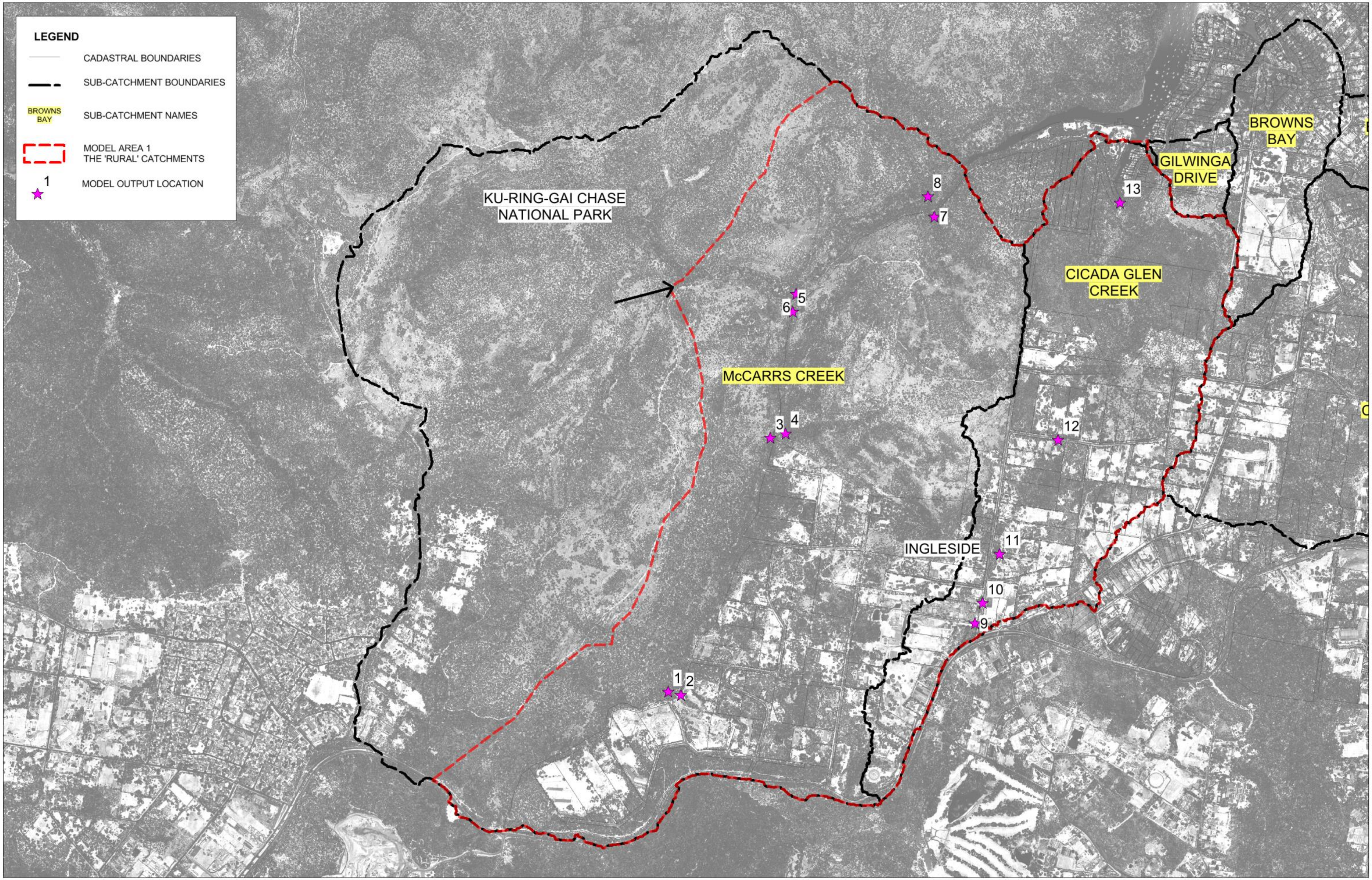

FIGURE 14 **MODEL OUTPUT LOCATIONS (MODEL AREA 1)**

PAGE SIZE A4 SCALE 1:25,000

0 375 750m
Approx. Scale

NORTH

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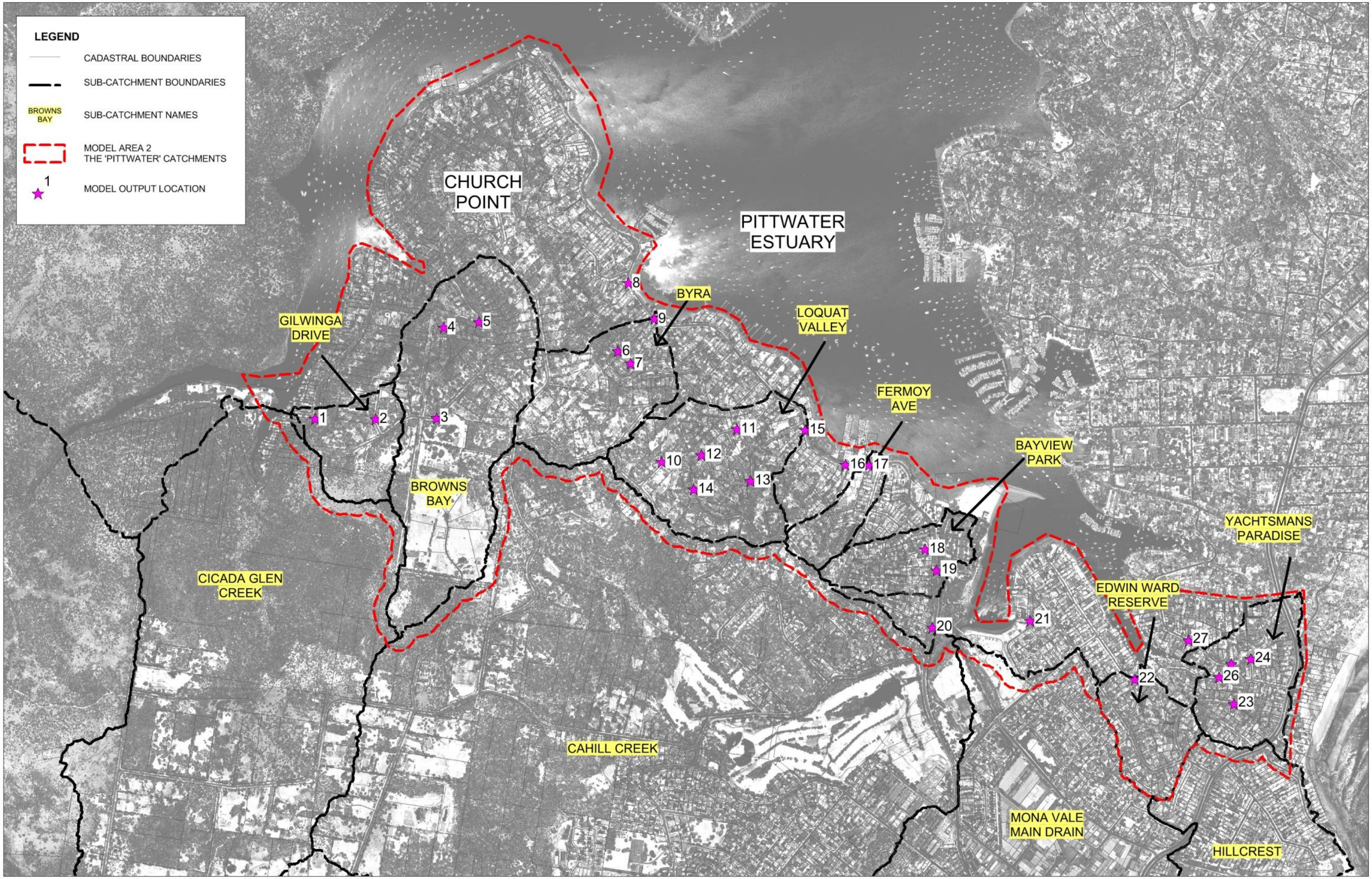

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FIGURE 15 MODEL OUTPUT LOCATIONS (MODEL AREA 2)

PAGE SIZE A4 SCALE 1:20,000

0 250 500m
Approx. Scale

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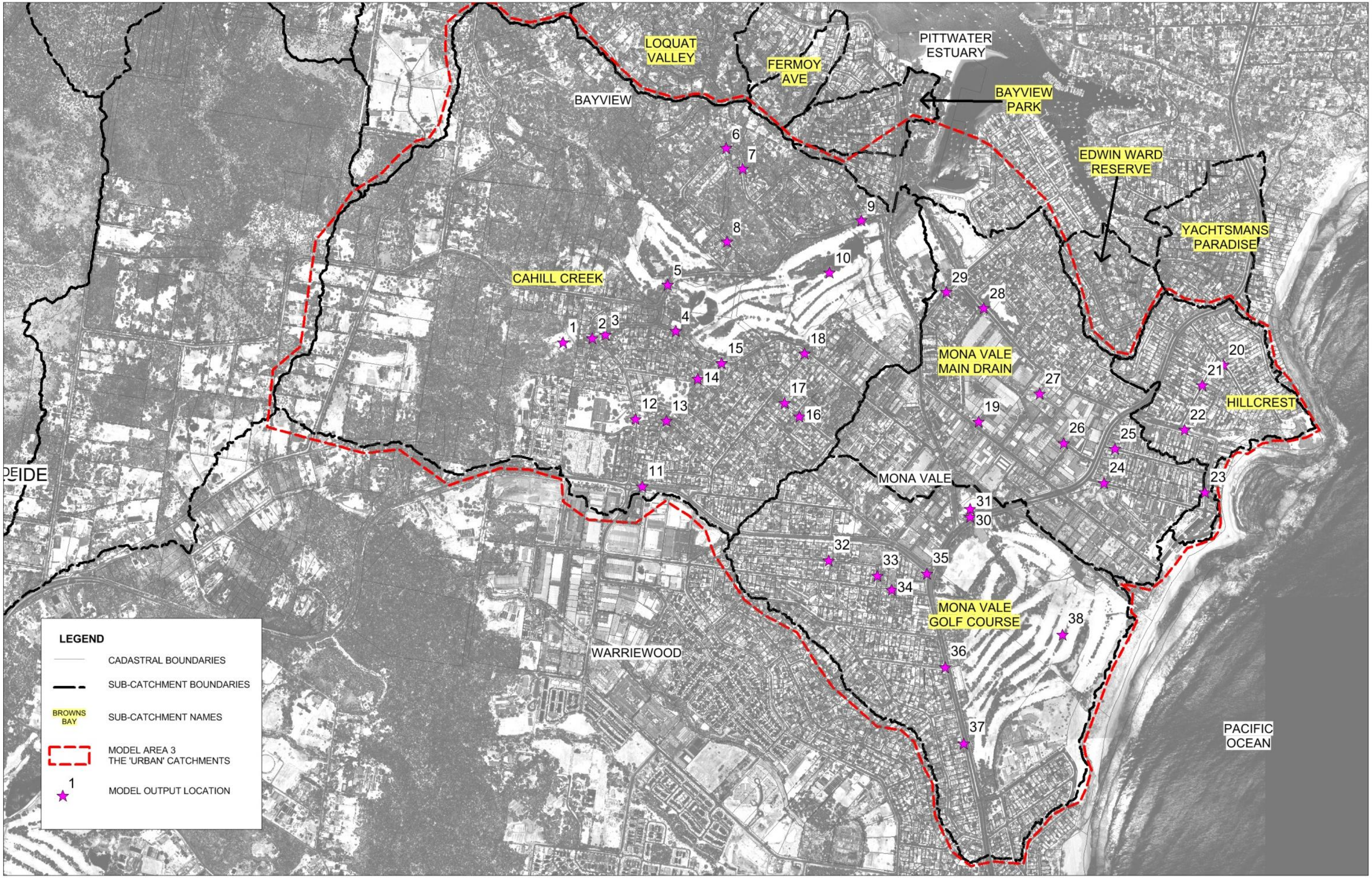

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FIGURE 16 MODEL OUTPUT LOCATIONS (MODEL AREA 3)

PAGE SIZE A4 SCALE 1:20,000

0 250 500m
Approx. Scale

NORTH

Disclaimer: While all reasonable care has been taken to ensure the information contained on this map is up to date and accurate, this map contains information from a variety of sources - no warranty is given that the information on this map is free from error or omission. Any reliance placed on such information shall be the sole risk of the user. Please verify the accuracy of all information prior to using it. This map is not a design document.

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6.3.3 Peak Flood Velocities

A summary of peak flood velocities at key locations is shown in **Table 6-8** through **Table 6-10** below, and the placement of these locations is shown in **Figures 14** to **16**. Estimated peak flood velocity is presented on **Figures A9** to **A12**. The values reported below are the maximum values from an envelope of scenarios as described in **Section 6.3.1**.

Table 6-8: Peak Flood Velocities – Model Area 1 – The 'Rural' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 1 – The Rural Catchments – Peak Flood Velocity (m/s)									
1	West Wirreanda Rd	0.31	0.35	0.41	0.45	0.49	0.54	0.59	1.36
2	East Wirreanda Rd	0.17	0.20	0.23	0.26	0.29	0.33	0.36	0.96
3	Wirreanda Rd North	1.47	1.50	1.54	1.56	1.59	1.61	1.65	2.08
4	Wirreanda Rd North	1.05	1.12	1.22	1.28	1.34	1.40	1.49	2.30
5	McCarrs Creek Rd	0.75	0.83	0.86	0.91	0.97	1.02	1.09	1.65
6	McCarrs Creek Rd	1.36	1.46	1.59	1.69	1.77	1.84	1.92	2.31
7	McCarrs Creek Rd	2.41	2.70	2.82	3.00	3.23	3.44	3.72	5.77
8	McCarrs Creek	2.73	2.93	3.16	3.33	3.50	3.66	3.84	5.98
9	Sophie Avenue	0.63	0.66	0.70	0.72	0.75	0.76	0.79	1.08
10	Chiltern Road	2.08	2.24	2.45	2.55	2.59	2.65	2.79	4.19
11	Glen Cicada Creek	0.23	0.26	0.30	0.35	0.39	0.43	0.48	1.38
12	Cicada Glen Rd	0.47	0.51	0.57	0.63	0.68	0.72	0.79	1.52
13	McCarrs Creek Rd	N/A	N/A	1.71	2.94	3.44	3.63	3.86	5.34

Table 6-9: Peak Flood Velocities – Model Area 2 – The 'Pittwater' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 2 – The Pittwater Catchments – Peak Flood Velocity (m/s)									
1	McCarrs Creek Rd	0.14	0.15	0.17	0.21	0.22	0.24	0.28	6.13
2	Barcoola Place	0.35	0.37	0.40	0.42	0.42	0.43	0.47	0.38
3	Gilwinga Drive	0.11	0.16	0.22	0.24	0.29	0.32	0.34	5.93
4	McCarrs Creek Rd	0.12	0.16	0.20	0.24	0.26	0.29	0.31	4.54
5	McCarrs Creek Rd	N/A	0.06	0.07	0.08	0.09	0.10	0.11	3.72
6	Kananook Avenue	0.20	0.23	0.27	0.32	0.30	0.31	0.36	1.10
7	Kananook Avenue	0.11	0.13	0.15	0.18	0.18	0.20	0.23	2.83
8	Pittwater Rd	0.06	0.07	0.08	0.10	0.10	0.11	0.12	3.77
9	Pittwater Rd	0.15	0.18	0.21	0.24	0.25	0.28	0.31	2.76
10	Clive Crescent	0.07	0.08	0.09	0.09	0.11	0.12	0.11	2.37
11	Jendi Avenue	0.22	0.24	0.27	0.28	0.30	0.31	0.32	0.86
12	Jendi Avenue	0.08	0.09	0.13	0.15	0.16	0.17	0.19	2.51
13	Loquat Valley Rd	0.11	0.14	0.20	0.21	0.25	0.28	0.27	2.24
14	Kookaburra Close	N/A	0.15	0.16	0.17	0.16	0.17	0.17	1.35
15	Pittwater Rd	N/A	N/A	0.10	0.15	0.17	0.20	0.24	2.46
16	Pittwater Rd	0.08	0.08	0.09	0.10	0.11	0.11	0.12	2.39
17	Pittwater Rd	0.09	0.09	0.09	0.12	0.10	0.10	0.12	1.79
18	Gerroa Avenue	0.12	0.13	0.15	0.15	0.18	0.19	0.21	2.39
19	Pittwater Rd	0.26	0.33	0.41	0.48	0.54	0.58	0.64	0.09
20	Pittwater Rd	0.18	0.18	0.19	0.20	0.21	0.21	0.22	0.92
21	The Esplande	0.35	0.36	0.38	0.38	0.39	0.40	0.41	0.12
22	Rednal Street	0.71	0.36	0.39	0.41	0.44	0.46	1.70	0.66
23	Crescent Rd	0.23	0.24	0.25	0.27	0.27	0.27	0.30	2.09
24	Suncrest Avenue	0.17	0.18	0.19	0.20	0.22	0.23	0.25	0.85
25	Crescent Rd	0.18	0.19	0.22	0.24	0.26	0.29	0.31	1.62
26	Yachtsmans Paradise	0.68	0.74	0.81	0.86	0.89	0.94	0.99	1.00
27	Crescent Rd	0.16	0.18	0.20	0.21	0.23	0.24	0.26	1.21

Table 6-10: Peak Flood Velocities – Model Area 3 – The 'Urban' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 3 – The Urban Catchments – Peak Flood Velocity (m/s)									
1	Peninsula Gdns Retirement	0.63	0.63	0.61	0.59	0.58	0.61	0.64	1.06
2	Old Samuel Street	0.77	0.91	1.04	1.18	1.30	1.39	1.46	1.91
3	Samuel Street	0.87	1.12	1.32	1.52	1.69	1.82	1.92	2.23
4	Parkland Road	0.74	1.14	1.39	1.51	1.74	1.89	2.05	2.94
5	Cabbage Tree Road	1.46	1.44	1.77	1.95	2.04	2.10	2.19	2.79
6	Annam Road	0.28	0.32	0.37	0.39	0.42	0.44	0.47	0.75
7	Annam Road	0.44	0.50	0.55	0.59	0.62	0.68	0.72	1.13
8	Annam Road	1.35	1.48	1.54	1.64	1.79	1.82	1.98	3.13
9	Cabbage Tree Road	0.69	0.79	0.93	0.99	1.02	0.86	0.93	2.23
10	Bayview Golf Course	0.13	0.13	0.13	0.14	0.14	0.14	0.15	0.42
11	Mona Vale Road	N/A	0.16	0.20	0.24	0.27	0.31	0.35	0.51
12	Samuel Street	0.39	0.41	0.45	0.46	0.46	0.46	0.46	0.50
13	Marie Crescent	0.16	0.16	0.16	0.19	0.24	0.30	0.37	0.98
14	Siobhan Place	N/A	N/A	0.13	0.42	0.54	0.69	0.85	2.31
15	Parkland Road	0.42	0.56	0.87	1.23	1.41	1.57	1.70	2.14
16	Waratah Street	1.05	1.13	1.20	1.25	1.30	1.36	1.41	2.28
17	Waratah Street	1.00	1.08	1.11	1.19	1.29	1.40	1.49	1.92
18	Cnr Maxwell St & Parkland Road	1.15	1.21	1.34	1.44	1.51	1.59	1.69	2.54
19	Wilmette Place	0.22	0.23	0.25	0.21	0.22	0.25	0.28	0.91
20	Grandview Parade	0.25	0.27	0.29	0.31	0.32	0.34	0.37	0.67
21	Orana Road	2.14	2.37	2.64	2.81	2.81	2.96	2.97	3.71
22	Bassett Street	0.18	0.18	0.18	0.19	0.19	0.19	0.22	0.48
23	Surfview Road	0.30	0.34	0.37	0.38	0.41	0.43	0.45	0.79
24	Seabeach Avenue	N/A	N/A	0.14	0.18	0.19	0.21	0.23	0.43
25	Heath Street	0.12	0.13	0.16	0.18	0.20	0.21	0.22	0.37

26	Polo Avenue	0.27	0.26	0.27	0.26	0.26	0.27	0.37	1.09
27	Tengah Crescent	N/A	0.13	0.13	0.14	0.12	0.12	0.13	0.47
28	Bassett Street	0.24	0.32	0.38	0.43	0.50	0.53	0.57	0.78
29	Mona Street	N/A	N/A	N/A	0.10	0.10	0.12	0.13	0.27
30	Barrenjoey Road	0.14	0.15	0.18	0.19	0.20	0.21	0.21	0.27
31	Barrenjoey Road	0.59	0.61	0.63	0.63	0.63	0.64	0.65	0.97
32	Brinawa Street	1.75	1.85	1.97	2.07	2.16	2.25	2.31	3.18
33	Brinawa Street	0.35	0.35	0.37	0.40	0.40	0.41	0.42	0.42
34	Vineyard Street	0.87	0.88	0.93	1.00	1.07	1.12	1.21	2.09
35	Cnr Mona Vale Rd & Pittwater Rd	0.68	0.76	0.88	0.96	0.98	0.96	1.01	1.01
36	Cnr Rowan St & Pittwater Rd	0.96	0.98	0.98	0.98	0.98	0.98	0.98	1.19
37	Pittwater Road	0.16	0.16	0.17	0.17	0.19	0.19	0.22	0.40
38	Mona Vale Golf Course	0.26	0.29	0.33	0.36	0.39	0.42	0.46	0.72

6.3.4 Design Hydrographs

Design hydrographs were output from the model results for the 20%, 50% and 100% AEP Events and the PMF Event at numerous locations across the study area. Hydrographs for these events are presented in **Appendix C**.

6.3.5 Provisional Flood Hazard

The NSW Government's Floodplain Development Manual (2005) defines flood hazard categories as follows:

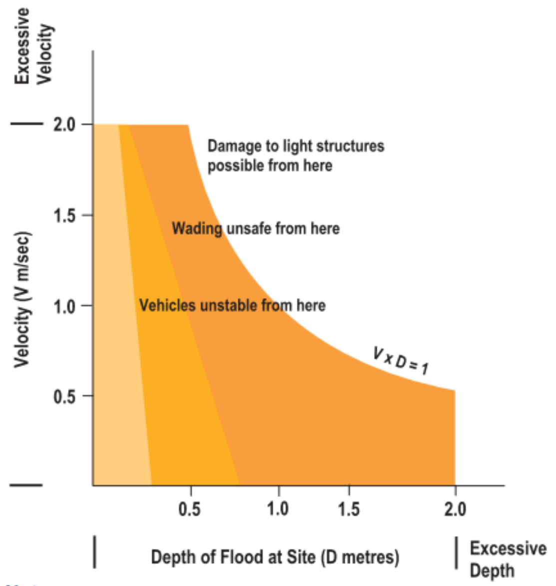
- **High hazard** – possible danger to personal safety; evacuation by trucks is difficult; able-bodied adults would have difficulty in wading to safety; potential for significant structural damage to buildings; and
- **Low hazard** – should it be necessary, trucks could evacuate people and their possessions; able-bodied adults would have little difficulty in wading to safety.

The key factors influencing flood hazard or risk are:

- Size of the Flood
- Rate of Rise - Effective Warning Time
- Community Awareness
- Flood Depth and Velocity
- Duration of Inundation
- Obstructions to Flow
- Access and Evacuation

The provisional flood hazard level is often determined on the basis of the predicted flood depth and velocity. This is conveniently done through the analysis of flood model results. A high flood depth will cause a hazardous situation while a low depth may only cause an inconvenience. High flood velocities are dangerous and may cause structural damage while low velocities have no major threat.

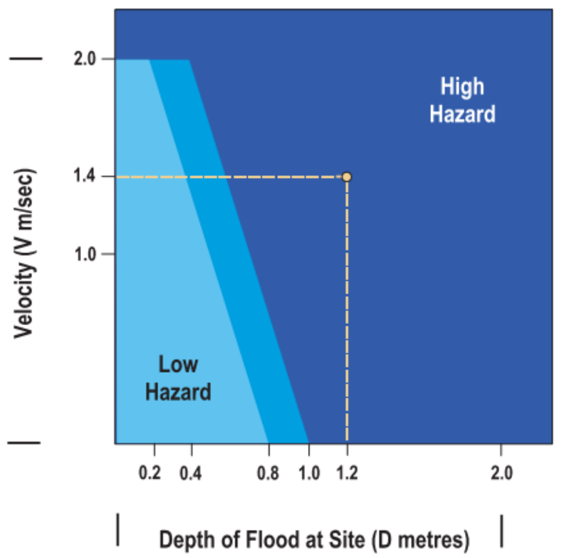
Figures L1 and L2 in the Floodplain Development Manual (NSW Government, 2005) are used to determine provisional hazard categorisations within flood liable land. These figures are reproduced in **Plate 6-1.**



Notes

1. At velocities in excess of 2.0 m/s, the stability of foundations and poles can be affected by scour. Also, grass and earth surfaces begin to scour and can become rough and unstable
2. The velocity of floodwaters passing between buildings can produce a hazard, which may not be apparent if only the average velocity is considered. For instance, the velocity of floodwaters in a model test has risen from an average of 1 m/sec to 3 m/sec between houses.
3. Vehicle instability is initially by buoyancy.
4. At floodwater depths in excess of 2.0 meters and even at low velocities, there can be damage to light-framed buildings from water pressure, flotation and debris impact.

Derived from laboratory testing and flood conditions which caused damage.



Notes

The degree of hazard may be either -

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

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

Example:

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

Plate 6-1: Provisional Flood Hazard Categorisation (NSW Government, 2005)

The provisional hydraulic hazard is included in the mapping series for each simulated design event provided in **Appendix A**.

6.3.6 *Preliminary Hydraulic Categorisation*

There are no prescriptive methods for determining what parts of the floodplain constitute floodways, flood storages and flood fringes. Descriptions of these terms within the Floodplain Development Manual (NSW Government, 2005) are essentially qualitative in nature. Of particular difficulty is the fact that a definition of flood behaviour and associated impacts is likely to vary from one floodplain to another depending on the circumstances and nature of flooding within the catchment.

The hydraulic categories as defined in the Floodplain Development Manual are:

- **Floodway -** Areas that convey a significant portion of the flow. These are areas that, even if partially blocked, would cause a significant increase in flood levels or a significant redistribution of flood flows, which may adversely affect other areas.
- **Flood Storage -** Areas that are important in the temporary storage of the floodwater during the passage of the flood. If the area is substantially removed by levees or fill it will result in elevated water levels and/or elevated discharges. Flood Storage areas, if completely blocked would cause peak flood levels to increase by 0.1m and/or would cause the peak discharge to increase by more than 10%.
- **Flood Fringe -** Remaining area of flood prone land, after Floodway and Flood Storage areas have been defined. Blockage or filling of this area will not have any significant effect on the flood pattern or flood levels.

A number of approaches were considered when attempting to define flood impact categories across study catchments. Approaches to define hydraulic categories that were considered for this assessment included partitioning the floodplain based on:

- Peak flood velocity (m/s);
- Peak flood depth (m);
- Peak velocity * depth (sometimes referred to as discharge per unit width (m^2/s));
- Cumulative volume conveyed during the flood event (m^3); and
- Combinations of the above.

The definition of flood impact categories that was considered to best fit the application within study catchments was ultimately provided by Council and was based on a combination of velocity*depth, velocity and depth parameters. The adopted hydraulic categorisation is defined in **Table 6-11**.

The hydraulic category maps for the 1% AEP and PMF events are included in **Appendix A**. It is also noted that mapping associated with the flood hydraulic categories may be amended in the future, at a local or property scale, subject to appropriate analysis that demonstrates no additional impacts (e.g. if it is to change from floodway to flood storage).

Table 6-11: Hydraulic Categories

Floodway	Defined using the following criteria: Velocity * Depth > 0.3 OR Velocity > 0.5 m/s	Areas and flowpaths where a significant proportion of floodwaters are conveyed (including all bank-to-bank creek sections).
Flood Storage	Defined where Depth > 0.3 metres	Areas where floodwaters accumulate before being conveyed downstream. These areas are important for detention and attenuation of flood peaks.
Flood Fringe	Defined where Depth < 0.3 metres	Areas that are low-velocity backwaters within the floodplain. Filling of these areas generally has little consequence to overall flood behaviour.

7 Model Sensitivity

Sensitivity analysis of model parameters is a required step in the development of a hydraulic model to understand the model's dependence upon model assumptions. Sensitivity analysis can also be undertaken to help understand proposed changes to modelling guidelines and the impacts of climate change. This section documents the sensitivity analysis undertaken as part of this Study.

7.1 Structure Blockages

The percentage of blockage applied to structures (culverts, bridges etc) in the hydraulic model can be a key parameter determining flood extent and levels. To test the sensitivity of the hydraulic model results to structure blockage, two test runs were undertaken applying 50% and 100% blockage to key culvert locations in the Urban Catchments hydraulic model for a 100 year return period event.

Plate 7-1 below is a 'difference map' showing the increase in flood level (m) under a 100% blockage scenario (and 50% blocked scenario in brackets) compared to a baseline scenario with no structure blockage.

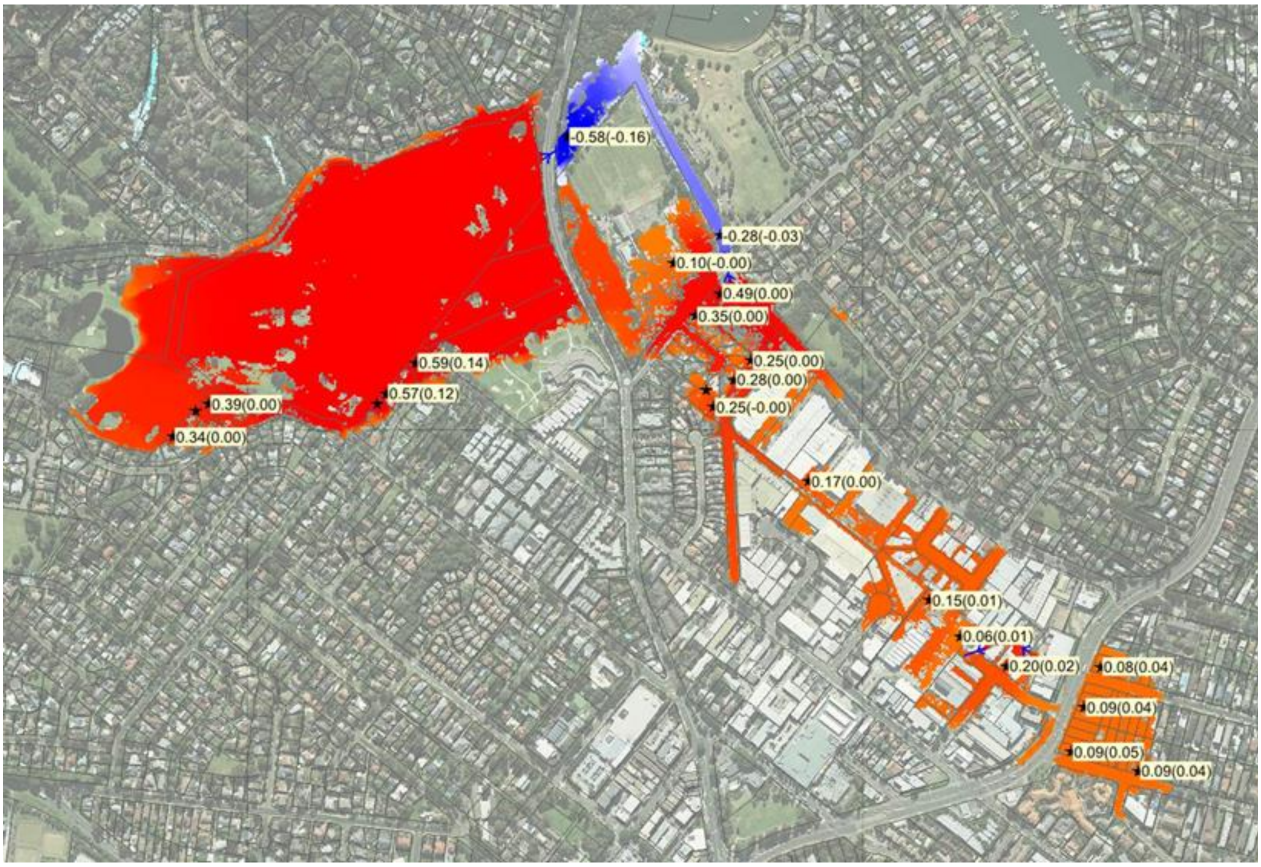


Plate 7-1: Structure Blockage Sensitivity Analysis Difference Map

Plate 7-1 shows that the 100% blocked scenario increases flood levels on properties by up to 600mm; and the 50% blocked scenario increases flood levels on properties by less than 150mm.

The 100% blocked scenario is provided here for context regarding the sensitivity of blockages (i.e. a worst-case scenario). Increases of up to 150mm (seen in the 50% blocked scenario) could be considered to be a relatively insignificant impact on results as applied freeboard for flood planning levels is generally 500mm.

7.1.1 Design Run Recommendations

The following recommendations are made for the design runs based on sensitivity testing of blockage factors:

- Blockages of 20 – 50% could be used in the design runs for culverts without significantly influencing flood levels.
- The main culverts in the design models should have the following level of blockages applied:
- **30%** blockages for all culverts where open channels enter pipes.
- **50%** blockages for the open drain to culverted sections in the Mona Vale industrial complex. The potential for debris to collect in the open drains in this area is considered high.
- **0%** Blockages to inlet pits in the drainage system. The reason to exclude blockages from the drainage network is that:
    - Only the trunk drainage system is being modelled for this study (i.e. pipes greater than 450mm in diameter)
    - The methodology for modelling the drainage network is to assume that the system is not inlet constrained. This assumption is to correctly identify existing pipe capacities in the trunk drainage system (rather than constraints associated with the surface inlets).

7.2 Design Rainfall

Engineers Australia are currently undertaking an extensive revision of the 1987 AR&R guidelines. As part of this process, new IFDs are being generated from a longer and more extensive data set than were used in the 1987 IFDs. The revised IFD data set remains under review and is not yet intended for implementation in design flood studies. A comparison to the new IFD data set is provided here for context of the likely changes to the study when the new AR&R projects are finalised.

Plate 7-2 plots the 1987 IFD and the revised 2013 IFD data sets using the centroid of the McCarrs Creek catchment.

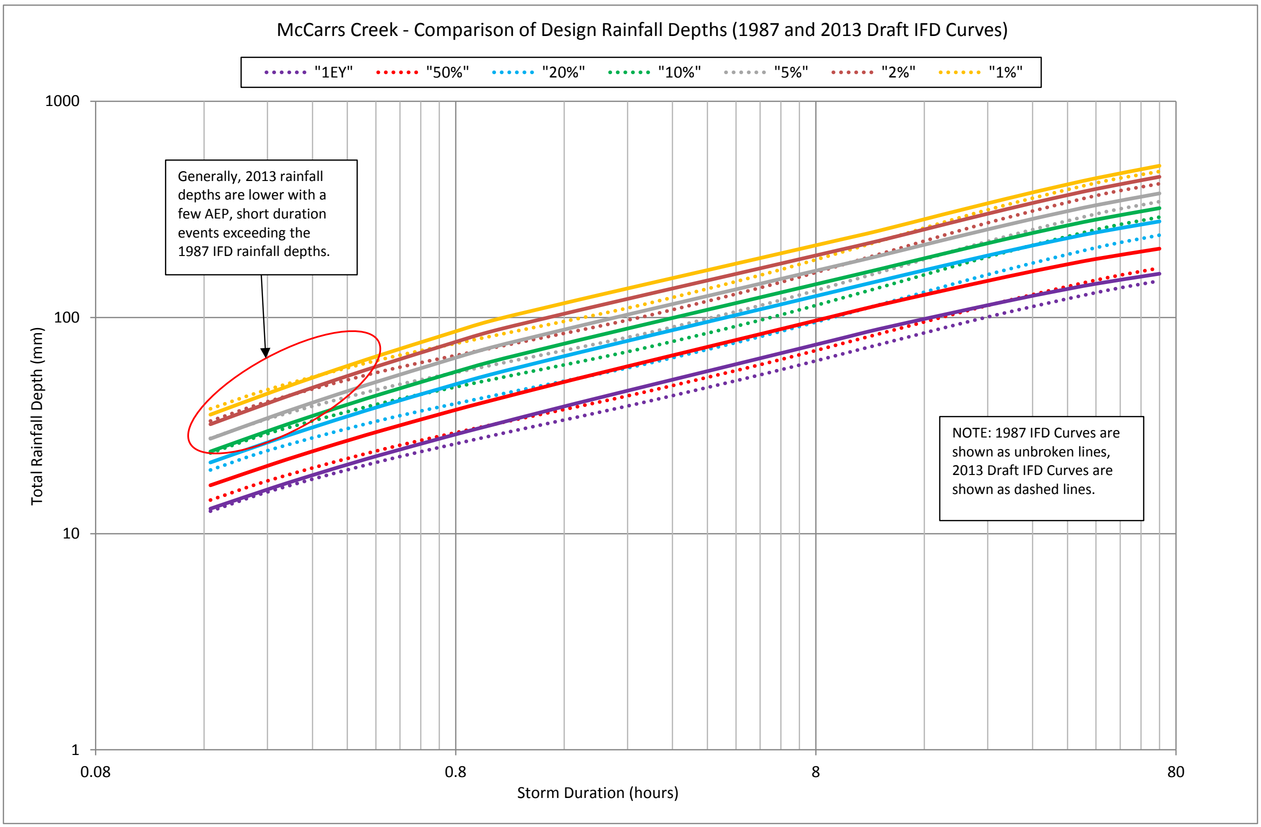


Plate 7-2: Comparison of Design Rainfall Depths at the McCarrs Creek Catchment

Plate 7-2 shows that for the McCarrs Creek Catchment, the 2013 draft IFD curves have generally lower design rainfall depths, averaging 13% less than the 1987 IFD curves. However, shorter duration events (<10mins) with a low Annual Exceedance Probability (AEP < 5%) are shown to have between 0 – 7% greater rainfall depths with the revised IFD curves. It is considered unnecessary to run a hydraulic model sensitivity of these events as the storm durations are too short for the size and nature of each of the catchments to be the critical duration storm.

8 Climate Change

8.1 Latest International Research on Climate Change Impacts

8.1.1 Annual Average Rainfall

Rainfall is the single most important climate variable for flood risk estimation. Several modelling studies are available to estimate changes in rainfall based on simulations from climate models. In mid-latitude and subtropical dry regions, mean precipitation is likely to decrease (IPCC 2013). Pittwater Council's Climate Change Risk Assessment (2012) also concluded that annual average rainfall is likely to decrease across the LGA.

Climate in Australia has a high natural variability, owing largely to the strong influence of the El Niño Southern Oscillation (ENSO), and especially rainfall is highly variable over Australia (Reisinger et al. 2014). Increases in precipitation have been found in north-western Australia since the 1950s, and declines in autumn/winter precipitation in south-western Australia since the 1970s and in south-east Australia since the 1990s (Reisinger et al. 2014). Apart from overall rainfall changes, the frequency of conditions suitable for thunderstorm occurrence has not been increasing in Australia, according to one study (Allen and Karoly 2013). For Australia, the evidence for past changes in extreme rainfall events (95th and 99th percentile) is mixed or insignificant (Reisinger et al. 2014), with for the east coast region, significant declines in total rainfall and extremes over the period 1950-2005 (Gallant et al, 2007).

8.1.2 Extreme Rainfall Events

With regard to heavy precipitation events, there are more land areas where heavy precipitation events have increased in frequency than areas where these have decreased (IPCC 2013). Extreme precipitation events will very likely become more intense and more frequent over mid-latitude land areas by the year 2100 (IPCC 2013). Pittwater Council's Climate Change Risk Assessment (2012) also concluded that rainfall event intensity will increase across the LGA.

It is important to note that changes in rainfall extremes in Australia have been observed for very small time intervals; i.e. sub-daily (Westra 2011; Westra and Sisson 2011; Jakob et al. 2011). This suggests that with further increasing temperatures, changes in rainfall may also occur at sub-daily time steps. Westra (2011) further notes that the spatial scale at which changes in rainfall occur, have not really been well addressed yet by research, and that while mean rainfall changes are related to circulation changes over larger areas, changes in intense rainfall may be occurring at smaller spatial scales. The same holds true for rainfall type (Westra, 2011); where mid-latitude storm types may move pole ward and become more important for Australia, while tropical cyclones become less frequent and tracks move southward.

Plate 8-1 illustrates according to two different ensemble simulation datasets the extent to which rainfall in December-February may change across Australia by the end of the century, albeit with considerable uncertainties for the Australian east coast. Future patterns of precipitation change according to a high climate change scenario (the RCP8.5 scenario; see left panel in **Plate 8-1** below) indicate that the east coast of NSW may see very small changes in total annual rainfall, while precipitation during December-February is expected to increase significantly (right panel) - (Irving et al 2012).

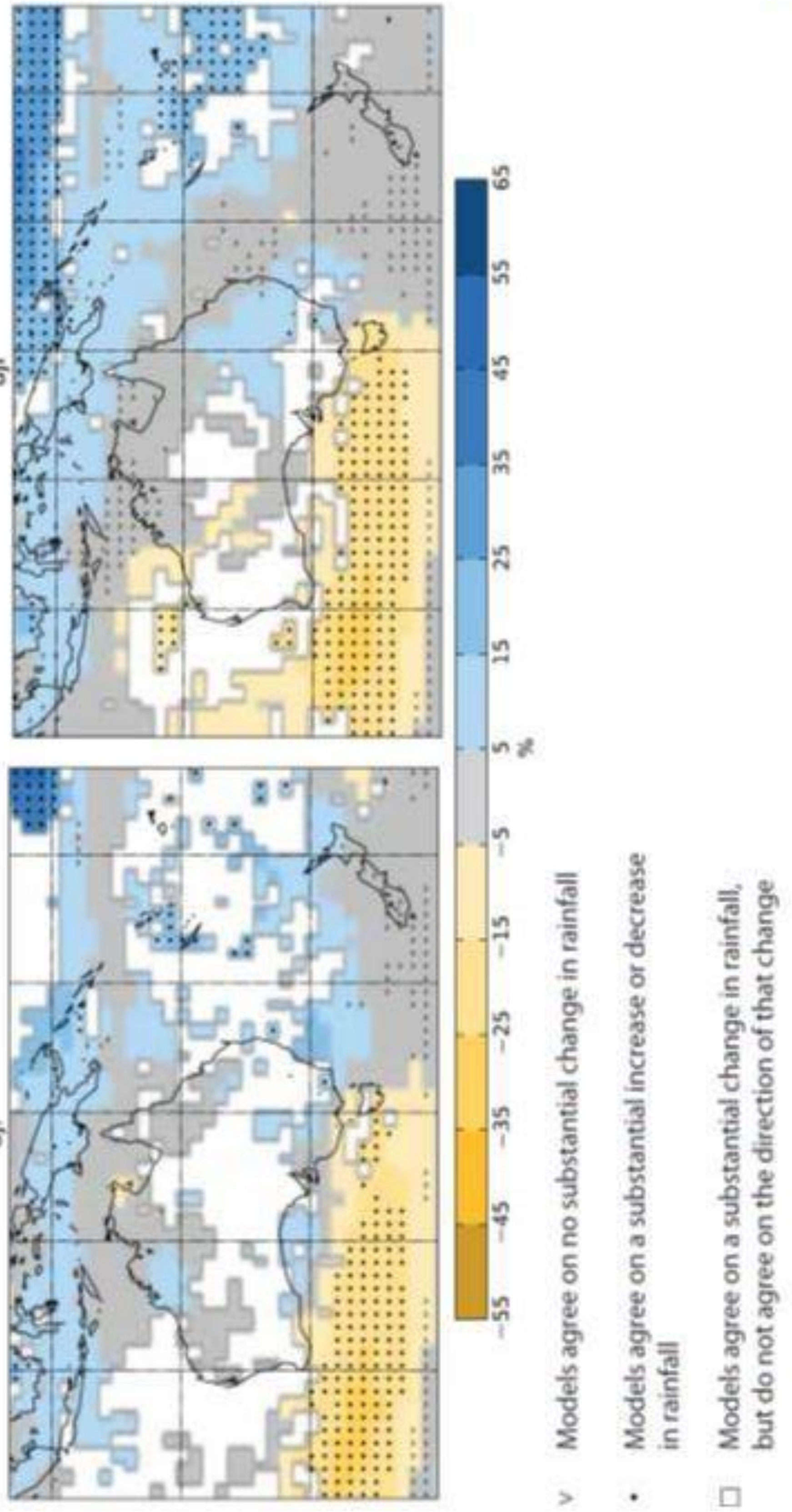


Plate 8-1: Expected Climate Change impacts in Australia based on IPCC Fifth Report

The finding that sub-daily rainfall amounts have increased alongside mixed trends in total rainfall (see above), leads to the implication that despite uncertainties in the change of annual and seasonal mean rainfall, intense rainfall events may occur more frequently in the future.

Projections of future extreme rainfall for Australia show possible increases in heavy rainfall events, substantially contributing to 5-day rainfall total and to annual rainfall totals (Alexander and Arblaster, 2009). Overall, the IPCC concludes that there is medium confidence in changes in current 20 year return period events and in short duration (sub-daily) extremes in most regions of Australia (Reisinger et al., 2014: Table 25-1).

8.1.3 Antecedent Conditions Considerations

There is little research on how climate change may affect antecedent conditions (principally soil moisture and evapotranspiration) that are important for the occurrence of flooding. In southern Australia there are indications that large scale circulation variability related to the Pacific-Decadal Oscillation (IPO) modulates soil moisture, thereby influencing flood occurrence (generally declining) through antecedent conditions, rather than through rainfall (Westra, 2011).

For instance, Micevski et al. (2003) demonstrate that during IPO negative phases, flood risk is substantially increased (up to a factor 2.0 x higher discharge). Flood conditions are expected to increase in the north of Australia, whereas in the south of Australia increasingly drier soil moisture conditions may compensate for changes in rainfall (Reisinger et al., 2014: Box 25-8). There is no research related specifically to Sydney on this topic.

Other processes that are less frequently considered include increased evaporation that could result in drier soil moisture conditions. A rapid change from a dry situation to a highly intense rainfall situation could influence runoff. Equally, drought conditions followed by extreme rainfall can exacerbate the amount of sediment discharged from the catchment. It could be recommended to monitor and assess both processes into the future, to inform modelling.

8.1.4 Sea Level Rise Considerations

The global mean sea-level increased by some 0.19 m between 1901 and 2010. For Australia, the rate of sea-level rise was 1.4 mm per year over the period 1900-2011 (Reisinger et al., 2014; Burgette et al., in press), slightly below the global average rate. Extreme sea-levels in Australia have risen at the same rate as the average sea level rise (Reisinger et al., 2014; Menendez and Woodworth, 2010).

Depending on the assumed emission scenario, global mean sea level is projected to rise by 0.53 to 0.97 m by 2100 (high emissions, RCP8.5) relative to the average of 1986-2005, or between 0.28 and 0.6 m (low emissions, RCP2.6) (IPCC, 2013).

Projected future sea-level rise along the Australian coast is expected to exceed the average historic rate, contributing to the trend in higher extreme sea- levels (Reisinger et al., 2014 IPCC, 2013). Studies suggest that with sea-level rise, the frequency of extreme sea-levels, as well as the number of exposed properties, may increase disproportionally along the Australian south-east coast (Reisinger et al., 2014; McInnes et al., 2011; McInnes et al.,2012), although other studies assume a proportionate increase (Wang et al., 2010).

For Australia, changes in future storms and cyclones are expected to play a minor role in changes in the occurrence of extreme sea-levels, compared to sea-level rise. A study using the CSIRO CCAM model found that the number of tropical cyclones may decrease strongly (by about 50%) by the end of the century (period 2051-2090 compared to 1971-2000), and a southward movement of genesis and decay regions (Abbs, 2012).

Finally, combinations of storm surge levels at the tail-end of the catchments, combined with intense rainfall from storm activity could potentially lead to peak water levels. It could be useful to assess the joint probabilities and intensities of these two processes, as a low-probability and high-impact event in a model.

8.2 NSW Government Policy Development on Climate Change

In NSW the 'Floodplain Development Manual: the management of flood liable land' (NSW Government, 2005), states that a flood study should address the possible implications of climate change on flood behaviour, including sea level rise, altered storm patterns and intensity and increased intensity and frequency of extreme events. The manual states the consequences of climate change on flood levels and behaviour should be analysed as part of a flood study either:

- Qualitatively based upon the broad range of floods being examined up to and including the PMF; or
- Sensitivity analysis in relation to rainfall intensity or downstream water level conditions for key flood events.

In 2007, more specific guidance was provided by the NSW Department of Environment and Climate Change (DECC, now Office of Environment and Heritage, OEH): 'Practical Consideration of Climate Change in Flood Investigations'. The guidelines recommend sensitivity analysis is considered for:

- Sea level rise – for low (0.18 m), medium (0.55 m) and high level impacts (up to 0.91 m); and
- Rainfall Intensities – for 10%, 20% and 30% increase in peak rainfall and storm volume.

The NSW Sea Level Rise Policy Statement (2009) provided by NSW DECC, now OEH, updated the best projections of sea level rise along the NSW coast, relative to 1990 mean sea levels, to be 0.4 m by 2050 and 0.9 m by 2100. It was acknowledged that higher rates were possible. The policy statement recommended these sea level rise benchmarks for use in flood risk assessments.

In 2012, the above sea level rise benchmarks were withdrawn by the NSW Government, following widespread concern that the coastal zone implications of their implementation were too onerous. The State Government instructed each Council to determine and implement its own benchmarks. In reality, and without any better science or guidance to follow, most NSW Councils have continued to adopt the 0.4 and 0.9 m sea level rise benchmarks.

Until relatively recently the table below provided the range of climate change scenarios typically modelled in NSW flood studies for each AEP event. Whilst being a comprehensive approach, this led to a significant number of events being run and subsequent modelling effort.

Table 8-1: Typical Matrix of Climate Change Scenarios adopted by NSW Coastal Councils

Scenario	Sea level rise (m)	Rainfall intensity increase
1	0.4	-
2	0.9	-
3	-	10%
4	-	20%
5	-	30%
6	0.4	10%
7	0.4	20%
8	0.4	30%
9	0.9	10%
10	0.9	20%
11	0.9	30%

More recently NSW practice has moved away from running all design events for all climate change cases. Instead it is now usual practice to only run a couple of events, the 1% AEP and a bigger and a smaller event. Consideration is currently being given in NSW and in the interim climate guidelines to putting more emphasis on how the probabilities of different events change rather than running extra climate change IFD runs. In most cases the relatively simple exercise of determining the percentage rainfall increase that would turn a 1% AEP event into a 0.5% (200 year) and 0.2% (500 year) will give a good picture of how changes in rainfall will affect risk management.

8.3 Adopted Tailwater Levels for different Sea Level Rise Scenarios

As outlined in **Section 6.2**, a number of Climate Change Sea Level Rise Boundary conditions were adopted based on a review of recent relevant literature. Different tailwater levels were adopted for either Pittwater estuary or Pacific Ocean (beach) outlet pipes.

Table 8-2: Adopted Tailwater Boundaries (Peak Level in Tidal Cycle)

Event	Pittwater Estuary Peak Level (mAHD)	Ocean Boundary Peak Level (mAHD)
1% AEP Event	1.50	2.60
2050 Climate Change Scenario (+0.33m)	1.83	2.93
2100 Climate Change Scenario (+0.83m)	2.35	3.43

8.4 Climate Change Model Scenarios

A total of six (6) climate change scenarios were simulated as outlined in **Table 8-3** below.

Table 8-3: Adopted Climate Change Scenarios to be modelled for this Study

Scenario	Rainfall	Tailwater
1	1% AEP Event + 10% Rainfall (simulated as 0.5% AEP Rainfall)	Current Conditions
2	1% AEP Event + 30% Rainfall (simulated as 0.2% AEP Rainfall)	Current Conditions
3	1% AEP Event	2050 Conditions
4	1% AEP Event	2100 Conditions
5	1% AEP Event + 30% Rainfall	2100 Conditions
6	PMF Event + 30% Rainfall	2100 Conditions

8.5 Climate Change Assessment Results

Results of the climate change simulations are presented in **Figures A16 – A23,** through a series of results maps and difference maps, highlighting the potential effects of climate change.

9 Development Controls

9.1 Flood Planning Areas

Flood planning areas were calculated for the study area for three main areas, utilising freeboard to the 1% AEP flood results. These include:

- Area 1: Mainstream Flooding – For areas within defined watercourse channels, a 0.5m freeboard was applied and the flood surface was laterally extended until it intersected with the ground surface (i.e. were the planning level would intersect the watercourse overbank).

- Area 2: Major Overland Flow Paths – For overland flow areas greater than 0.3m deep a 5m horizontal buffer was applied to the modelled flood extent as freeboard.

- Area 3: Minor Overland Flow Paths – For overland flow areas less than 0.3m deep no freeboard was applied.

Flood planning areas are presented in **Figure A26**.

9.2 Council's Flood Categories

Council's Flood Control categories were updated during the course of the study to include the following:

- ***Low Flood Risk precinct*** refers to all *flood prone land* (i.e. within the PMF extent) not identified within the High or Medium flood risk precincts.
- ***Medium Flood Risk precinct*** means all *flood prone land* that is (a) within the 1% AEP Flood Planning Area; and (b) is not within the high flood risk precinct.
- ***High Flood Risk precinct*** means all *flood prone land* (a) within the 1% AEP Flood Planning Area; and (b) is either subject to a high hydraulic hazard, within the floodway or subject to significant evacuation difficulties (H5 and or H6 Life Hazard Classification).

Property classification mapping was undertaken utilising the filtered flood results. In addition to the map filtering (outlined in **Section 2.7.1**), properties were tagged where the flood extent encroached on the property boundary by more than 1m. Results of the property classification mapping are presented in **Figure A25**.

Properties at Risk Analysis

A flood information database was produced for all of the study area catchments and contained the following:

- Address, lot and DP number;
- Land use (i.e. residential, commercial etc);
- Typical ground level (assumed from LiDAR);
- Maximum peak flood level and flood depth across the property for:
    - 20% AEP;
    - 10% AEP;
    - 5% AEP;
    - 2% AEP;
    - 1% AEP;
    - 0.5% AEP;
    - 0.2% AEP; and
    - PMF;
- Average flow velocity, flood hazard, flood risk (high or low) and hydraulic flood category for:
    - 1% AEP; and
    - PMF;
- Flood Planning Levels;
- Flood categories under Council's DCP, as discussed in **Section 9.1**; and
- Climate Change Levels.

10 State Emergency Services (SES) Requirements

10.1 Duration of Inundation for Road Crossings

The maximum duration of inundation of flood waters over a number of road crossings was derived from the flood model results. To achieve this, the total duration that flood waters exceeded a cut-off value (0.15m) over the road crown was extracted from the time series results.

A summary of the maximum time of inundation a number of road crossings is shown in **Table 6-8** through **Table 6-10** below. The placement of these locations is shown in **Figure 14** to **16**. The values reported below are the maximum values from an envelope of scenarios as described in **Section 6.3.1**.

Table 10-1: Duration of Road Inundation – Model Area 1 – The 'Rural' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 1 – The Rural Catchments – Duration of Inundation (hours)									
1	West Wirreanda Rd	13.6	13.7	13.9	14.0	14.1	14.2	14.2	7.9
2	East Wirreanda Rd	12.6	12.8	13.0	13.4	13.5	13.7	13.8	7.8
3	Wirreanda Rd North	11.8	11.9	12.2	12.4	12.6	12.7	12.8	7.2
4	Wirreanda Rd North	9.2	9.6	9.8	10.5	10.8	11.1	11.4	6.5
5	McCarrs Creek Rd	10.4	10.8	11.3	11.6	11.9	12.1	12.4	7.5
6	McCarrs Creek Rd	13.7	13.8	13.8	13.9	13.9	13.9	14.0	7.7
7	McCarrs Creek Rd	11.4	11.6	11.6	12.1	12.3	12.5	12.8	7.4
8	McCarrs Creek	7.5	8.2	8.8	9.4	9.8	10.2	10.6	7.2
9	Sophie Avenue	0.0	0.2	0.3	0.5	0.7	1.1	1.7	3.0
10	Chiltern Road	0.0	0.0	0.0	0.4	0.9	1.3	1.6	3.0
11	Glen Cicada Creek	14.4	14.5	14.6	14.6	14.6	14.6	14.7	8.0
12	Cicada Glen Rd	14.4	14.4	14.6	14.7	14.7	14.7	14.7	7.9
13	McCarrs Creek Rd	0.0	0.0	0.0	0.0	0.4	0.8	1.1	2.6

Table 10-2: Duration of Road Inundation – Model Area 2 – The 'Pittwater' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 2 – The Pittwater Catchments – Duraiton of Inundation (hours)									
1	McCarrs Creek Rd	0.0	0.1	0.2	0.3	0.6	0.7	0.9	0.7
2	Barcoola Place	5.5	6.0	6.1	5.9	6.2	6.2	6.3	3.9
3	Gilwinga Drive	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.8
4	McCarrs Creek Rd	0.0	0.1	0.2	0.3	0.4	0.5	0.7	0.8
5	McCarrs Creek Rd	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1
6	Kananook Avenue	1.4	1.4	1.5	1.6	1.7	1.7	1.9	1.5
7	Kananook Avenue	0.0	0.0	0.1	0.2	0.2	0.3	0.3	0.7
8	Pittwater Rd	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1
9	Pittwater Rd	0.1	0.1	0.3	0.4	0.5	0.6	0.8	0.9
10	Clive Crescent	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2
11	Jendi Avenue	0.7	0.8	0.9	1.3	1.4	1.5	1.6	0.9
12	Jendi Avenue	0.0	0.0	0.0	0.1	0.1	0.2	0.2	0.6
13	Loquat Valley Rd	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.7
14	Kookaburra Close	4.8	4.8	4.8	4.7	4.9	4.9	4.9	3.2
15	Pittwater Rd	0.0	0.0	0.0	0.0	0.1	0.1	0.2	0.7
16	Pittwater Rd	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.1
17	Pittwater Rd	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.4
18	Gerroa Avenue	0.1	0.1	0.2	0.2	0.3	0.3	0.5	0.7
19	Pittwater Rd	1.2	1.7	2.1	2.4	2.6	2.7	2.9	2.1
20	Pittwater Rd	0.7	0.8	0.9	1.2	1.3	1.4	1.5	0.9
21	The Esplande	8.5	8.5	8.6	8.6	8.6	8.6	8.7	6.0
22	Rednal Street	1.1	1.2	1.5	1.9	2.0	2.1	2.3	1.0
23	Crescent Rd	0.9	1.0	1.1	1.3	1.4	1.4	1.5	0.8
24	Suncrest Avenue	0.2	0.2	0.4	0.6	0.7	0.7	0.9	0.7
25	Crescent Rd	0.2	0.2	0.3	0.4	0.5	0.7	0.8	0.8
26	Yachtsmans Paradise	2.3	2.4	2.6	2.7	2.8	2.9	2.9	1.2
27	Crescent Rd	0.1	0.1	0.2	0.3	0.3	0.4	0.5	0.7

Table 10-3: Duration of Road Inundation – Model Area 3 – The 'Urban' Catchments

ID	Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF Event
Model Area 3 – The Urban Catchments – Duration of Inundation (hours)									
1	Peninsula Gdns Retirement	5.0	5.4	5.9	6.6	7.1	7.4	7.8	3.3
2	Old Samuel Street	2.0	2.3	2.7	3.5	4.1	4.8	5.4	3.4
3	Samuel Street	0.6	0.9	1.1	1.4	1.8	2.1	2.7	3.2
4	Parkland Road	0.7	1.1	1.2	1.6	1.8	2.2	2.8	3.1
5	Cabbage Tree Road	3.7	3.8	4.1	4.9	5.2	5.5	5.9	3.2
6	Annam Road	9.6	9.8	10.2	10.5	10.5	10.6	10.7	5.2
7	Annam Road	0.5	0.5	0.6	0.7	0.7	0.8	0.8	3.0
8	Annam Road	0.1	0.2	0.3	0.4	0.6	0.8	0.9	3.3
9	Cabbage Tree Road	1.2	2.3	3.1	4.0	4.5	5.0	5.6	4.6
10	Bayview Golf Course	6.3	6.7	7.2	7.9	8.3	8.8	9.2	6.6
11	Mona Vale Road	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.4
12	Samuel Street	0.7	0.9	1.1	1.4	1.7	2.0	2.5	3.4
13	Marie Crescent	1.2	1.5	1.9	2.3	2.8	3.2	3.9	3.3
14	Siobhan Place	10.9	11.1	11.3	11.4	11.5	11.6	11.8	7.0
15	Parkland Road	12.0	12.2	12.4	12.7	12.8	12.9	12.9	8.4
16	Waratah Street	0.1	0.1	0.3	0.6	0.7	0.8	1.0	3.2
17	Waratah Street	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.8
18	Cnr Maxwell St & Parkland Road	0.0	0.0	0.0	0.1	0.1	0.1	0.2	1.9
19	Wilmette Place	2.7	3.1	3.4	3.8	4.2	4.5	4.7	3.3
20	Grandview Parade	13.8	13.9	14.0	14.1	14.2	14.2	14.3	9.9
21	Orana Road	0.5	0.6	0.9	1.1	1.3	1.6	1.7	3.3
22	Bassett Street	0.2	0.3	0.5	0.6	0.8	0.9	1.1	3.2
23	Surfview Road	2.4	2.8	3.4	4.1	4.7	5.3	6.3	4.2
24	Seabeach Avenue	0.0	0.0	0.0	0.1	0.1	0.2	0.3	2.9
25	Heath Street	3.3	5.0	4.3	5.0	6.1	6.8	7.4	4.8

26	Polo Avenue	10.7	11.1	11.2	11.5	11.7	11.9	12.1	8.2
27	Tengah Crescent	1.2	3.5	2.0	2.7	3.4	4.1	4.9	3.6
28	Bassett Street	2.4	3.4	4.1	5.0	6.0	6.9	7.4	5.2
29	Mona Street	0.6	1.0	1.5	2.2	3.0	3.9	4.4	3.4
30	Barrenjoey Road	12.4	12.8	13.0	13.4	13.5	13.7	13.8	9.8
31	Barrenjoey Road	12.1	12.2	12.3	13.0	13.3	13.5	13.7	9.8
32	Brinawa Street	6.0	6.5	7.0	8.2	8.7	9.3	10.2	5.6
33	Brinawa Street	0.0	0.0	0.0	0.1	0.1	0.2	0.2	2.3
34	Vineyard Street	6.5	7.0	7.4	8.0	8.2	8.4	8.5	3.4
35	Cnr Mona Vale Rd & Pittwater Rd	0.1	0.2	0.2	0.4	0.4	0.6	0.6	2.4
36	Cnr Rowan St & Pittwater Rd	5.5	5.9	6.4	7.0	7.2	7.5	7.7	3.4
37	Pittwater Road	0.5	0.6	0.8	0.9	1.1	1.3	1.5	3.3
38	Mona Vale Golf Course	7.0	7.4	7.8	8.2	8.3	8.5	8.6	3.4

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Appendix A – Preliminary Flood Depth Maps

List of Figures (Note: All maps have windows A, B and C)

Appendix B – Community Consultation Information

07 July 2017

Appendix C – Design Hydrographs

07 July 2017