NORTH SYDNEY LGA

FLOOD STUDY

FINAL REPORT

FEBRUARY 2017

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Project North Sydney LGA Flood Study	**Project Number** 114035
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NORTH SYDNEY LGA FLOOD STUDY

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LIST OF ABBREVIATIONS

1D	One (1) Dimensional
2D	Two (2) Dimensional
AEP	Annual Exceedance Probability
ALS	Airborne Laser Scanning
ARI	Average Recurrence Interval
AR&R	Australian Rainfall & Runoff
DEM	Digital Elevation Model
E/Y	Exceedances per Year
FPA	Flood Planning Area
FRMP	Floodplain Risk Management Process
FRMS and P	Floodplain Risk Management Study and Plan
IFD	Intensity-Frequency-Duration
LGA	Local Government Area
LiDAR	Airborne Light Detection and Ranging Survey
m AHD	Meters Australian Height Datum
NSC	North Sydney Council
RL	Reduced Level
SES	State Emergency Service
SWC	Sydney Water Corporation
TIN	Triangular Irregular Network

FOREWORD

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

Under the Policy, the management of flood liable land remains the responsibility of Local Government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities. By engaging in the NSW Government's floodplain risk management process Councils are provided with indemnity under Section 733 of the Local Government Act (NSW Government, 1993).

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

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

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

3. Floodplain Risk Management Plan
- Involves formal adoption by Council of a plan of recommended measures to mitigate or eliminate the flood risk to life and property.

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

This report pertains to Stage 1 of the above process and seeks to present flood modelling work, which outlines the nature and extent of the flood risk to the community in North Sydney's Local Government Area (LGA).

The Floodplain Development Manual (NSW Government, 2005) and Australian Rainfall and Runoff (AR&R) (Engineers Australia, 1987) define the industry standards for the undertaking of this work.

EXECUTIVE SUMMARY

THE PROCESS

The Flood Study is the first step in the NSW State Government's overall Floodplain Risk Management Program. The overall program consists of these steps:

- Flood Study – Defines design flood affectation;
- Flood Risk Management Study and Draft Plan – examines flood risk and looks to manage flood risk via actual works, improved planning by Council in future and via emergency response augmentation; and finally
- Plan Implementation.

The terms used within this Flood Study are in accordance with the NSW Government's Floodplain Development Manual (2005), from which:

- A 'Floodplain' is defined as an "area of land that is subject to inundation by floods up to and including the Probable Maximum Flood (PMF) event";
- 'Local overland flooding' is defined as "inundation by local runoff rather than … from a stream, river, estuary, lake or dam";
- 'Mainstream flooding' is defined as "inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam";

STUDY AREA

WMAwater have been appointed by North Sydney Council (Council) to carry out the North Sydney Council LGA Overland Flow Flood Study (the Study). The study investigates local overland flooding and mainstream flooding to determine the nature and extent of the flood hazard.

The study area is Council's 10.9 km^2 LGA and this area contains 18 separate catchments. These 18 catchments have been consolidated into 4 hydraulic modelling extents; herein identified as North, South, East and West.

Modelling has been carried out for all catchments. All main flow elements are represented including creek flow, overland flow and pit/pipe flow interactions.

OBJECTIVES

The aim of the study is to assist Council in satisfying their responsibilities for managing flood risk within the LGA. The study will achieve this by doing the following:

- Create hydrologic and hydraulic models that accurately describe flooding mechanisms in the LGA. These models will suit definition of design flood behaviour and also be suitable for later application in the subsequent management study;
- Describe design flood affectation via tabulated data and maps of flood depth/extent inclusive of digital data for handover to Council; and finally
- Provide a sound basis for the subsequent Floodplain Risk Management Study and Plan.

This report specifically presents the following work:

- Data collection information;
- Community consultation results;
- Methodology description for hydrologic (DRAINS) and hydraulic (TUFLOW) modelling;
- Model verification results;
- Modelling results of design events;
- Modelling results of sensitivity events; and
- Hotspot discussion.

FLOODING HISTORY

Various sources have been examined to gauge the flooding history in the study area, including Council's flood database, Sydney Water's flood database, newspaper articles and community consultation. From this, anecdotal evidence of past flooding was found. However these sources whilst describing flooding had occurred, often did not record the depth or level of flooding (that is required to be of use for calibration).

The anecdotal evidence from these flood databases and newspapers reported instances of flooding as having occurred in 1984, 1986, 1991 and 2010. These dates corresponded to high intensity rainfall recorded by rainfall gauges in the vicinity of the study area.

HYDROLOGIC AND HYDRAULIC MODELLING

The statistical analysis of rainfall to determine design rainfall magnitudes and probabilities was sourced from the Bureau of Meteorology. From this, design flood behaviour has been modelled for various design rainfall events, from the 20% AEP event to the Probable Maximum Flood. Various design rainfall durations have been examined and critical durations identified.

Various hotspots have been identified during the course of the work and detailed information for these is provided in Section 9 of the report. SES information on road crossing inundation has been provided as has specific SES mapping required defining risk precincts.

MOVING FORWARD

The study defines design flood levels for the entire North Sydney LGA of 10.9 km^2, which can be incorporated into Council's database to inform planning and infrastructure decisions. Development Applications (DA) can also benefit from the study, with the flood levels able to be used to inform the floor level of proposed development to ensure that the proposed buildings will not suffer damage (or pose a risk to life) in the future due to flooding. Optionally, where larger scale development is proposed for flood affected properties, it could be possible for developers to utilise the models developed as part of the flood study, reported upon herein. This provides a service to developers as they should be able to avoid the cost of individual model establishment. It also serves Council and its residents as it ensures a base standard for modelling work carried out for flood affected development. The model should be used to ensure that proposed development does not exacerbate flood levels on properties other than those proposed for development.

After the Flood Study has been adopted, Council may apply for funding from the NSW Government to initiate a Floodplain Risk Management Study and Plan (FRMS&P) that are the next two stages of the NSW Floodplain Risk Management Program.

1. INTRODUCTION

1.1. Study Area

The North Sydney Council Local Government Area (LGA) is located on the north shore of Sydney Harbour. The area includes the suburbs of Cammeray, Cremorne, Cremorne Point, Crows Nest, Kirribilli, Kurraba Point, Lavender Bay, McMahons Point, Milsons Point, Neutral Bay, North Sydney and Waverton (shown on Figure 1).

The study area is highly urbanised, with approximately 58% of the LGA zoned for residential use, 9% for commercial and industrial use, 19% for open space areas (including parks, recreational areas and environmental conservation) and 14% for special purpose. Special purpose areas within the study area include schools, classified infrastructure and Commonwealth Government land such as Kirribilli House.

Major infrastructure, such as arterial roads and railway tracks, traverse the study area. The Warringah Expressway bisects the study area from north to south; Military Road is aligned east to west (up to the Warringah Expressway); and the Pacific Highway diverges from the Warringah Expressway in a north-westerly direction. The railway track that services the North Shore line is located south-west of the Pacific Highway and includes the Milsons Point, North Sydney, Waverton and Wollstonecraft Stations.

The study area is bisected by a ridge that reaches elevations up to 100 m AHD and is aligned east to west. Military Road and part of the Pacific Highway form part of the ridge. The north area of the LGA drains north into Long Bay within Middle Harbour, and the south area drains into Sydney Harbour. The study area is relatively steep with an average slope median of 10%.

1.2. Objectives

The primary objective of this Flood Study is to define design flood behaviour for the 20%, 10%, 5% and 1% AEP design storms and the Probable Maximum Flood (PMF) and to:

- prepare suitable models of the catchment and floodplain for use in a subsequent Floodplain Risk Management Study;
- provide results for design flood behaviour in terms of design flood levels, depths, velocities, flows and flood extents within the study area;
- prepare maps of provisional hydraulic categories and provisional hazard categories;
- prepare preliminary emergency response classifications for communities as per SES guidelines; and
- assess the sensitivity of flood behaviour to potential climate change effects such as increases in rainfall intensities and sea level rise.

A glossary of flood related terms is provided in Appendix A. Abbreviations are described following the Table of Contents.

2. AVAILABLE DATA

2.1. Introduction

Data collection and review is a fundamental step in the Study as per the NSW Floodplain Risk Management Program (FRMP). The purpose of the data collection and review stage is to ensure that the best possible use is made of existing resources.

Data of interest to a Flood Study consists of previous reports or observations of catchment flood behaviour, asset and GIS data and hydrological data such as rainfall and stream gauging records. A wide variety of data has been collected and reviewed for this Study. The following sections present the data collected and discuss what use may be best made of this information for the Study.

The two main purposes of data collection are to establish suitable data for model build and to find suitable data for verifying the accuracy of the models prior to them being applied to design flood estimation.

2.2. Model Build Data

As per Sections 3, 4 and 5 the models to be built are hydrologic and hydraulic models. Inputs required are:

- Topographic data to facilitate discretisation of catchments and sub-catchments;
- Detailed topographic data for the hydraulic model extent;
- Data describing drainage assets such as pits, pipes, culverts, open channels and the like;
- Aerial photos so that solid buildings which can obstruct and displace flow can be digitised for inclusion in hydraulic routing;
- GIS layers which describe various land uses as this data forms an excellent starting point for maps of “roughness” which inform hydraulic routing calculations; and
- Design rainfall data for a range of flood events from the more common (20% AEP event) to the rarest (the PMF or Probable Maximum Flood).

The following sections detail the data collected for the model build work. Data for verifying purposes then follows in Section 2.3.

2.2.1. Topographic Data

The Study’s main goal is computer modelling of flooding and this is achieved primarily through use of a 2D hydraulic model. Such models in turn rely entirely on quality topographical data that describes ground elevations and hence heavily influences flow routing calculations.

Bulk topographical data came from Airborne Light Detection and Ranging (LiDAR) survey (also known as Aerial Laser Scanning (ALS)) of the catchment and its immediate surroundings, with this data being provided by Council. Metadata indicates that the survey was collected in 2008.

ALS datasets typically have accuracy in the order of:

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

The accuracy of the ALS data can be negatively influenced by the presence of open water or vegetation (tree or shrub canopy) at the time of the survey.

Using the ALS data a Triangular Irregular Network (TIN) was generated by WMAwater. This TIN was sampled at a regular spacing of 1 m by 1 m to create a Digital Elevation Model (DEM) [shown on Figure 2], which formed the basis of the two-dimensional hydraulic modelling for the study .

Note that the base topographical data described above is amended prior to use in modelling, with this work further described in Section 3.2 and 5 of the report.

2.2.2. General GIS Data

A variety of digital data is required in order to carry out the study. Most datasets are useful for analysis and presentation purposes. Examples of such data provided by Council include:

- High resolution aerial photographs;
- Local Environmental Plan (LEP) layers;
- Cadastre; and
- Road extents and street names.

2.2.3. Sydney Water Stormwater Studies

Sydney Water has prepared various reports that investigate the capacity performance of the SWC owned infrastructure. The reports were:

- Brook Street (SWC 19) Capacity Assessment Report – May 2002;
- Careening Cove SWC 25 Capacity Assessment Report – August 2002;
- Euroka Creek SWC 40 Capacity Assessment Report – May 2002;
- Neutral Bay (SWC 71) Capacity Assessment Report – August 2002; and
- Willoughby Creek (SWC 96) Capacity Assessment Report – June 2002.

The catchments that did not have SWC reports (as listed in Table 8) did not have SWC owned infrastructure within the catchment area.

The drainage data used for the SWC studies included the SWC trunk drainage system only and the analysis was undertaken using a spread sheet analysis based on:

- Rational Method for inflows;
- Approximate capacities of pipes based on grade and area;
- Approximation of channel capacities using Manning’s “n” formula; and the
- Hydraulic Grade Line method.

The SWC Capacity Assessment reports have been used in the present study for informing the SWC owned pit and pipe location, invert and dimensions, as well as for model verification (discussed in Section 6).

Table 1: Previous Reports Summary (see Figure 2)

Catchment	Sydney Water Stormwater Assets Capacity Assessment
Catchment A - Anderson Park	Yes
Catchment B - Brook Street	Yes
Catchment C - Christie Street	No
Catchment F - Fall Street	Yes
Catchment G - Gore Cove	No
Catchment H - Hayes Street	No
Catchment J - Jeffreys Street Wharf	No
Catchment K - Kirribilli	No
Catchment L - Long Bay	No
Catchment M - Milsons Park	Yes
Catchment P - Smoothey Park	No
Catchment R - Rocklands Road	No
Catchment S - Shell Cove	No
Catchment V - Vernon Street	No
Catchment W - Walker Street	No
Catchment X - Mosman Bay	No
Catchment Y - Young Street	No
Catchment Z - Berrys Bay	Yes

2.2.4. Pit and Pipe Data

Pit and pipe data is instrumental to the model works reported upon herein. Pit and pipe assets in the LGA are owned by Council, Sydney Water Corporation (SWC), Roads and Maritime Services (RMS) and the Sydney Train Authority. SWC owned assets are the trunk drainage elements that consist of larger pipes and open channels (discussed in Section 2.2.5). The RMS owned assets convey flow beneath major roads such as the Warringah Expressway. Likewise, the Sydney Trains owned assets convey flow beneath the Railway line. Council pipes tend to discharge flow into the SWC owned trunk system or directly into Sydney Harbour.

SWC provided the location, invert and dimensions of SWC owned drainage assets in electronic format for this study. Another important source of details on SWC assets are the Drainage Capacity Assessment reports that have previously been carried out and are discussed in Section 2.2.3.

Council provided the dimensions for the majority of pipes owned by Council, RMS and Sydney Trains within the LGA. Where dimensions were unavailable, site visits were carried out by WMAwater and in some cases dimensions of upstream and downstream pipes were used to establish sizes by interpolation.

The inverts of pits provided by Council were based upon either ground survey (undertaken in February 2009) or ALS with a known depth to invert or an assumed depth to invert. The small inaccuracies associated with estimating pipe slope based on an assumed depth below ground surface has an insignificant impact on peak flood level estimation.

Note all pipe elements smaller than 450 mm have not been included in modelling work and instead these are implicitly assumed to be blocked in all modelled events.

2.2.5. Open Channel Data

A key feature of the study area is the lack of mainstream flooding (i.e. flooding associated with a watercourse such as a canal, creek, channel or other). That said, there are some sections of open channel in the LGA, though these tend to be at the downstream end of the catchment.

The SWC capacity assessment reports (described in Section 2.2.3) were used to define the cross-sectional geometry of the open channels within the LGA. Site inspections and in some cases measurements undertaken by WMAwater supplemented this data.

Photo 1: Open Channel in North Sydney

2.2.6. Design Rainfall Data

To determine the design flood behaviour (including design flood depths, design flood levels and design flood velocities) within the catchment, it is necessary to obtain the design rainfall data. Design rainfall is statistical rainfall that has a certain probability of occurring; often identified as an Average Recurrence Interval (ARI) or Annual Exceedance Probability (AEP).

The design rainfall intensity-frequency-duration (IFD) data was obtained from the Bureau of Meteorology's online design rainfall tool. The input parameters for these calculations are sourced from AR&R (1987). It is noteworthy that the ongoing revision of AR&R will lead to new design rainfalls being published within the next 12 months.

Table 2: Rainfall IFD data at the centre of the North Sydney Council LGA

DURATION	Design Rainfall Intensity (mm/hr)						
	1 yr ARI	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI
5 minutes	97.1	125	159	179	205	240	266
6 minutes	91	117	149	168	193	225	250
10 minutes	74.5	95.9	123	139	160	188	209
20 minutes	54.4	70.5	91.9	105	121	143	159
30 minutes	44.3	57.5	75.6	86.3	100	119	133
1 hour	30.1	39.2	52	59.6	69.5	82.7	92.8
2 hours	19.8	25.8	34.3	39.4	46	54.8	61.5
3 hours	15.4	20.1	26.6	30.6	35.6	42.4	47.6
6 hours	10	13	17.2	19.6	22.9	27.2	30.4
12 hours	6.49	8.43	11.1	12.7	14.8	17.6	19.7
24 hours	4.23	5.49	7.28	8.35	9.73	11.6	13
48 hours	2.69	3.51	4.69	5.4	6.31	7.52	8.46
72 hours	2	2.62	3.51	4.04	4.73	5.64	6.35

The Probable Maximum Precipitation (PMP) estimates were derived according to Bureau of Meteorology guidelines, namely the *Generalised Short Duration Method* (BoM, 2003). The estimates obtained are summarised in Table 3.

Table 3: PMP Design Rainfall Intensity (mm/hr)

Duration	Design Rainfall Intensity (mm/hr)
30 minutes	470
1 hour	345
2 hours	260
3 hours	210
6 hours	140

Note that the PMF is primarily used to examine emergency flood response issues particularly with reference to vulnerable or sensitive citizens, such as hospitals and child care facilities.

2.2.7. Site Visits

Site visits to the study area are often carried out through the course of the flood study to gain an understanding of catchment details for the purpose of informing the model establishment process.

WMAwater conducted site visits on Wednesday 18th June 2014, Monday 1st September 2014, and Thursday 22nd January 2015. A selection of photographs taken during the June 2014 site visit are shown on Figure 3. Hydraulic structures such as the Warringah Expressway, bridges traversing the open channel, and Gross Pollutant Traps (GPT) were the primary focus of the first site visit. The second site visit focused on supplementing drainage network data. The third site visit was focused on inspecting the four specific hotspots presented in Section 7.

2.3. Model Calibration/Validation Data

The model calibration/validation process is further described in Section 6 of this report. This section describes data collected in order to aid model calibration/validation. Such data includes observed event rainfall and flood marks (where available) as well as known flooding "hotspots", reports from community on flooding locations and newspaper articles on previous flood events which identify flood affected locations.

2.3.1. Historical Rainfall Data

Historic rainfall data is of interest as it allows us to identify occasions when extreme events occurred and to perhaps correlate these with observations of flooding coming from the community or newspapers etc.

2.3.1.1. Rainfall Stations

There are a number of rainfall stations within a 20 km radius of the LGA. This includes daily read stations, continuous pluviometer stations and operational stations.

The daily read stations record total rainfall for the 24 hours to 9am of the day being recorded. Hence the rainfall received for the period between 9:00 am on 3 February 2008 until 9:00 am on 4 February 2008 would be recorded on the 4 February 2008.

The continuous pluviometer stations record rainfall in sub-daily increments (typically every 5 or 6 minutes). These records were used to create detailed rainfall hyetographs, which typically form a model input for historical events against which the model is calibrated.

Table 4 and Table 5 present a summary of the official rainfall gauges (sourced from the Bureau of Meteorology) located close to or within the catchment areas. The location of the rainfall stations are shown on Figure 4.

Table 4: Daily rainfall stations within 10km of the centre of the North Sydney LGA

Distance From Centre of LGA (km)	Station Number	Station name	Date Opened	Date Closed	Years
0.94	66061	Sydney Nth Bowling Club	1950 Apr	1974 Nov	20.9
0.94	66067	Wollstonecraft	1915 Jan	1975 May	59.1
1.24	66166	Cremorne Grasmere Road	1963 Jan	1989 Apr	26
1.45	66075	Waverton Bowling Club	1955 Dec	1999 Aug	33.1
2.03	66151	Primrose Park (Folly Point)	1912 Aug	1918 Aug	6
2.03	66130	Northbridge (Sailors Bay)	1935 Jan	1980 Apr	45.3
2.97	66062	Sydney (Observatory Hill)	1858 Jul	2014 May	155.9
3.02	66178	Birchgrove School	1904 May	1910 Jun	5.9
3.42	66042	Mosman (Bapaume Road)	1895 Sep	2005 Mar	108.6
3.51	66184	Mosman Council	1984 Sep	2007 Aug	19.1
3.58	66006	Sydney Botanic Gardens	1885 Jan	2014 Apr	115.8
3.66	66041	Mosman Water Supply	1904 Jun	1966 Dec	38.9
3.9	66012	Chatswood Water Supply	1894 Jan	1970 Aug	55.9
3.9	66039	Middlecove	1943 Feb	1953 Jul	10.5
3.9	66094	Willoughby	1908 Jan	1925 Jun	17.5
4.01	66010	Chatswood Council Depot	1897 Jan	1993 Dec	58.9
4.36	66175	Schnapper Island	1932 Mar	1939 Jul	5.7
4.36	66149	Glebe Point Syd. Water Supply	1907 Jun	1914 Apr	6.9
4.59	66011	Chatswood Bowling Club	1951 Jul	2014 May	24
5.1	66131	Riverview Observatory	1905 Feb	2014 May	93.9
5.34	66002	Balgowlah (Ethel Street)	1940 Jun	1989 Dec	49.3
5.46	66139	Paddington	1970 Jan	1976 Dec	5.6
5.51	66015	Crown St. Reservoir	1882 Feb	1960 Dec	63.8
5.63	66145	Seaforth Castle Circuit	1968 Oct	1993 Dec	25.1
5.92	66080	Castle Cove (Rosebridge Ave)	1958 Oct	2014 May	55.7
6.15	66056	Roseville Bowling Club	1914 Jun	1979 Aug	60.6
6.39	66163	Watsons Bay (Hmas Watson)	1968 Oct	1997 Nov	21.8
6.43	66099	Manly (Fairlight)	1926 Sep	1936 Aug	10
6.72	66095	Sydney (Hornby Light)	1843 Jan	1918 Dec	14.2
7.02	66089	Manly North Bowling Club	1961 Dec	1987 Nov	21.3
7.06	66108	Hunters Hill St Josephs Colleg	1916 Jan	1923 Dec	8
7.11	66153	Manly Vale (Manly Dam)	1906 Jun	2006 Oct	56.1
7.16	66160	Centennial Park	1900 Jun	2014 May	111.3
7.33	66097	Ranwick Bunnerong Rd	1904 Jan	1924 Apr	16.3
7.35	66033	Alexandria (Henderson Road)	1962 May	1999 Jun	1.3
7.51	66005	Bondi Bowling Club	1939 Jul	1982 Oct	42
7.58	66209	Dover Heights (Portland St)	2007 Apr	2014 May	7.2
7.61	66068	Vaucluse	1934 Mar	1975 Jul	39.9
7.63	66034	Abbotsford (Blackwall Point Rd)	2004 Jan	2014 Apr	10.3
8.01	66081	North Ryde Stroud Street	1960 Aug	1977 Dec	17.4
8.07	66032	Lindfield West	1950 Apr	1992 Jun	40.7

8.14	66066	Waverley Shire Council	1936 Jan	1964 Oct	28.8
8.24	66138	Manly (North Head)	1968 Dec	1997 Jul	20.9
8.43	66065	Watsons Bay (Vaucluse)	1914 Jan	1952 Mar	33.2
8.47	66105	Killara	1909 Jul	1922 Jan	12.3
8.47	66031	Killara (Metro)	1948 Oct	1951 Jun	2.8
8.63	66112	Bondi	1887 Jan	1924 Dec	36.3
8.65	66071	Gladesville Champion Rd	1997 Mar	2000 Sep	3.4
8.76	66111	Craydon	1879 Feb	1921 Aug	13.6
8.77	66088	Manly North	1959 Sep	1975 Jul	15.9
8.8	66102	Meadow Bank	1903 Jan	1916 Dec	13.3
8.83	66052	Randwick Bowling Club	1888 Jan	2014 May	106.3
8.94	66000	Ashfield Bowling Club	1894 Jan	2012 Mar	104.7
8.98	66187	Tamarama (Carlisle St)	1991 Jul	1999 Mar	6.8
9.08	66035	Manly Town Hall	1914 May	1963 Nov	49.3
9.08	66213	North Ryde Golf Club	2011 Aug	2014 May	2.8
9.17	66017	Five Dock (Barnwell Park Golf Course)	1938 Jan	2003 Nov	64
9.17	66021	Alexandria (Erskineville)	1948 Aug	1973 Nov	23.3
9.29	66048	Concord (Brays Rd)	2000 Apr	2014 May	14.2
9.55	66118	Frenchs Forest Fitspatrick Ave	1964 Jun	1982 Aug	18.3
9.63	66182	Frenchs Forest (Frenchs Forest Rd)	1957 Feb	2014 May	56.7
9.78	66189	West Pymble (Wyuna Road)	1992 Apr	2011 May	17.9
9.94	66120	Gordon Golf Club	1906 Aug	2013 Aug	100.2
9.96	66101	Marrickville (Fernbank)	1889 Jan	1913 Dec	25

Table 5: Pluviometer rainfall stations within 20km of the centre of the North Sydney LGA

Distance From Centre of LGA (km)	Station Number	Station name	Date Opened	Date Closed	Years
2.7	66022	Fort Denison	1994 Aug	1997 Sep	0.3
2.97	66062	Sydney (Observatory Hill)	1993 Sep	2014 May	20.8
11.89	66194	Canterbury Racecourse Aws	1996 Jan	2014 May	18.3
12.71	66212	Sydney Olympic Park AWS (Archery Centre)	2011 Aug	2014 May	2.8
12.87	66037	Sydney Airport Amo	1995 Apr	2014 May	19.2
13.52	66195	Sydney Olympic Park (Sydney Olympic Pk A	1996 Jan	2011 Aug	15.7
15.92	66059	Terrey Hills Aws	2004 Sep	2014 May	9.8

2.3.1.2. Analysis of Daily Read Data

An analysis of the daily records for the nearest rainfall stations was undertaken to identify and place past storm events in some context. The Sydney (Observatory Hill) station is located to the south of the LGA study area and the Mosman (Bapaume Road) station is located to the north-east of the LGA study area. Both stations have periods of record greater than 100 years; however the Mosman station was decommissioned in 2005.

Table 6: Largest daily rainfalls recorded in the vicinity of North Sydney Council LGA

Sydney : Observatory Hill (66062)			
Jul 1858 – to date			
Rank	Date	Number of days accumulated	Rainfall (mm)
1	6/08/1986	1	328
2	28/03/1942	1	281
3	3/02/1990	1	244
4	9/11/1984	1	235
5	25/02/1873	1	226
6	28/05/1889	1	212
7	11/03/1975	1	198
8	7/07/1931	1	198
9	10/02/1956	1	192
10	6/02/1878	1	191
11	29/04/1860	1	191
12	17/01/1988	1	191
13	9/02/1992	1	190
14	1/05/1955	1	188
15	13/01/1911	1	180
16	8/01/1973	1	169
17	3/04/1861	1	168
18	12/01/1918	1	166
19	9/03/1913	1	166
20	11/04/1998	1	165

Mosman : Bapaume Road (66042)			
Sept 1895 – Mar 2005			
Rank	Date	Number of days accumulated	Rainfall (mm)
1	6/08/1986	1	319
2	11/03/1975	1	297
3	3/02/1990	1	251
4	28/03/1942	1	237
5	17/01/1988	1	222
6	14/11/1969	1	215
7	24/01/1976	6	198
8	10/02/1956	1	191
9	24/03/1984	1	189
10	9/02/1992	1	179
11	10/01/1949	1	173
12	1/05/1955	1	168
13	7/07/1931	1	168
14	9/05/1925	1	163
15	8/05/1953	1	163
16	4/02/1990	1	161
17	30/04/1988	1	159
18	5/02/2002	1	159
19	8/08/1998	1	156
20	8/01/1973	1	154

The results indicate that the largest daily rainfall events in recent times (~ last 30 years) occurred in 1984, 1986 and 1990. The 1984 event is known to have caused flooding across Sydney.

High daily rainfall totals will not necessarily result in widespread flooding of the catchment, particularly if the rainfall is fairly evenly distributed throughout the day. This can be attributed to flooding within the catchment typically resulting from intense rainfall over sub-daily durations.

2.3.1.3. Analysis of Pluviometer Data

Continuous pluviometer records provide a more detailed description of temporal variations in rainfall. As such, the Sydney Observatory Hill (066062) pluviometer station was analysed.

Table 7 shows the rainfall intensities at the Sydney Observatory Hill (066062) pluviometer assessed for the 30 minute, 1 hour and 2 hour storm burst durations and compared to frequencies derived from AR&R 1987 (shown in Table 2). These durations were selected for analysis based upon the previous reports; within which it was found that the 25 minute storm duration was critical in the majority of previous studies. The critical storm duration range was from 15 minutes to 90 minutes across the previous studies.

Of the significant rainfall events that have occurred in recent years, the 1984 event produced the highest peak burst intensity, estimated to be in the order of a 100 year ARI (or 1% AEP) event.

The 1984 event was also shown to have a longer duration storm burst than the other two events.

Figure 5 A to C show histograms of the 3 identified events. Figure 6 A to C show events IFD data versus ARR design estimates.

Table 7: Peak Burst Intensities of Significant Rainfall Events at the Observatory Hill (066062) Pluviometer

Rainfall Event		30 min	60 min	120 min
8th November 1984	Intensity (mm/hr)	149	111	77
	ARI Estimate	> 100yr ARI	> 100yr ARI	> 100yr ARI
26th January 1991	Intensity (mm/hr)	119	60	32
	ARI Estimate	50yr – 100yr ARI	10yr – 20yr ARI	2yr – 5yr ARI
12th February 2010	Intensity (mm/hr)	80	58	32
	ARI Estimate	5yr – 10yr ARI	5yr – 10yr ARI	2yr – 5yr ARI

2.3.1.4. Various Studies

A number of previous reports are available for select catchments within the North Sydney Council LGA. These are summarised in Table 8 and discussed in the following section.

Catchment Studies have been carried out by Council or by consultants on Council's behalf. The method used in the studies varies as does the reason for the study being carried out. Almost all of them however do focus on drainage assets and tend not to utilise models which route overland flows. Whilst the Sydney Water studies are an excellent source of data on the drainage system (specifically trunk elements of it) the Catchment Studies are an excellent source of information on which areas have suffered flooding issues over time. Most, if not all, of them appear to have been carried out in response to reported flooding/drainage issues.

Flooding problem areas identified in the Catchment Studies are mapped in Figure 33 to Figure 36 and utilised for model verification purposes. See Section 6 for a discussion of this.

Table 8: Previous Reports Summary – Various Reports (see Figure 2)

Catchment	Catchment Study
Catchment A - Anderson Park	Yes
Catchment B - Brook Street	Yes
Catchment C - Christie Street	No
Catchment F - Fall Street	Yes
Catchment G - Gore Cove	No
Catchment H - Hayes Street	Yes
Catchment J - Jeffreys Street Wharf	No
Catchment K - Kirribilli	No
Catchment L - Long Bay	Yes
Catchment M - Milsons Park	Yes
Catchment P - Smoothey Park	Yes
Catchment R - Rocklands Road	Yes
Catchment S - Shell Cove	Yes
Catchment V - Vernon Street	No
Catchment W - Walker Street	Yes
Catchment X - Mosman Bay	No
Catchment Y - Young Street	Yes
Catchment Z - Berrys Bay	Yes

Anderson Park Catchment Management Study 1996 (Reference 2)

North Sydney Council undertook this study in 1996. The primary aim of this study was to investigate the performance of the existing stormwater drainage system and ascertain where future upgrades and extensions would be necessary.

An ILSAX hydrologic model was established for this study. The design storm events analysed were the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI events for the critical duration of 25 minutes.

The Council's Stormwater Drainage Design Guidelines Document (Reference 12) was used to assess the capacity of the existing drainage system. The criteria were:

- 1 in 50 year ARI where urban neighbourhood development is directly affected and severe damage to property or loss of life could occur;
- 1 in 20 year ARI for major shopping centres and major road crossings;
- 1 in 10 year ARI for neighbourhood shopping centres, industrial and service trade areas;
- 1 in 5 year ARI for urban neighbourhood development;
- 1 in 0.25 to 1 in 5 year ARI for trunk drainage facilities in 'open space' situations. The return frequency adopted will depend on adjoining land use, scour potential, public nuisance and safety etc.

Nineteen locations within the study area were identified as having insufficient capacity. These were:

- Ben Boyd Road;
- Bent Street;
- Bray Street;
- Colindia Street;
- Doris Street (Pipeline Below Glen Ormiston Complex);
- Forsyth Park;
- Holdsworth Road;
- Kurraba Road (Corner of Clarke Road);
- McLaren Street;
- Merlin Street;
- Montpelier Street (Corner of Premier);
- Corner of Montpelier Street and Spruson Street;
- Neutral Street;
- Premier Street;
- Ridge Street;
- Spruson Street (near Holdsworth Road);
- Walker Street (corner of Hampden Street);
- Winter Avenue; and
- Wyagdon Street.

Brook Street Drainage Stormwater Investigation 1990 (Reference 3)

This study was undertaken by Willing and Partners Consulting Engineers on behalf of North Sydney Council in 1990. The scope of this study included field inspection of the stormwater drainage system, and hydrologic and hydraulic analyses. The purpose was to assess the performance of the existing stormwater drainage system and identify where future upgrades and extensions would be necessary.

The hydrologic model used for the study was ILSAX. The design storm events analysed were the 5 year, 10 year, 20 year and 100 year ARI events for the critical duration of 25 minutes. The hydraulic modelling was undertaken using EXTRAN-XP for the 5 year, 20 year and 100 year ARI events.

The study found that "…the most severely affected areas are along the trunk drain upstream of the Warringah Expressway at Hume Lane, Willoughby Road, Chandos Street, Wheatleigh Street and Brook Street." (Willing and Partners, 1990)

Drainage Catchment Management Studies – Crows Nest Road, Ryries Parade, Smoothey Park and Waverton Park Catchment 1998 (Reference 4)

This study was undertaken by Patterson Britton and Partners (PBP) Pty. Ltd. on behalf of North Sydney Council in 1998. The scope of this study included field inspection of the stormwater drainage system, and hydrologic and hydraulic analyses. The purpose was to assess the performance of the existing stormwater drainage system and identify where future upgrades and extensions would be necessary.

The 1998 study investigated four catchments within the North Sydney Council LGA. Since that time, some of the catchments studied in 1998 have been renamed and amalgamated. The corresponding catchment names are presented in Table 9.

Table 9: Catchment Names – 1998 study compared to current study

Catchment Name In 1998 Study	Catchment Name In Current Study
Catchment L – Ryries Parade	Catchment L – Long Bay
Catchment U – Waverton Park	Catchment Z – Berrys Bay
Catchment X – Crows Nest Road	Catchment Z – Berrys Bay
Catchment Z – Smoothey Park	Catchment P – Smoothey Park

The DRAINS software was used for the hydrologic modelling and the hydraulic modelling. The design storm events analysed were the 5 year, 10 year, 20 year and 100 year ARI events for the critical duration of 15 minutes.

The Council's Stormwater Drainage Design Guidelines Document (Reference 12) was used to assess the capacity of the existing drainage system. The criteria were as defined in the Anderson Park Catchment Management Study 1996.

The report classified insufficiencies within the stormwater drainage system as resulting from either water surcharging from pits, freeboard to the top of the inlet pits less than 0.15m, and pit inlet capacity restricting flow into the pipe network.

Within the Crows Nest Road catchment, areas were identified where localised drainage/flooding issues were identified as follows:

- Overland flow (through private property) from the McHatton Street sag;
- Overland flow (through private property) from the Carr Street sag;
- Open channel between sag in Bay Road and 1200mm railway drainage culvert; and
- Sag in Woolcott.

Falls Street Drainage Study 1995 (Reference 5)

This study was undertaken by RUST PPK Pty Ltd. on behalf of North Sydney Council in 1995. The scope of this study included field inspection of the stormwater drainage system, and hydrologic and hydraulic analyses. The purpose was to assess the performance of the existing stormwater drainage system and identify where future upgrades and extensions would be necessary.

The hydrologic model used for the study was ILSAX. The design storm events analysed were the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI events for the critical duration of 90 minutes.

The Council's Stormwater Drainage Design Guidelines Document (Reference 12) was used to assess the capacity of the existing drainage system. The criteria were as defined in the Anderson Park Catchment Management Study 1996.

The ILSAX model was used to estimate the stormwater drainage augmentations required to meet Council's guidelines. This uses the "pipline full, but not under pressure" assumption. Further analysis on the existing drainage network with the augmentation work recommended was undertaken for the 20 year and 100 year ARI events in the hydraulic model, which was the HYLINE model.

Hayes Street Catchment Study 1992 (Reference 6)

This study was undertaken by Willing and Partners Consulting Engineers on behalf of North Sydney Council in 1992. The scope of this study included field inspection of the stormwater drainage system, and hydrologic and hydraulic analyses. The purpose was to assess the performance of the existing stormwater drainage system and identify where future upgrades and extensions would be necessary.

An ILSAX hydrologic model was established for this study. The design storm events analysed were the 5 year, 10 year, 20 year and 100 year ARI events for the critical duration of 25 minutes.

The hydraulic analysis consisted of a hydraulic grade line analysis (in spreadsheet form using LOTUS 123) using the Colebrook-White equation to estimate head-losses in the pipe and the Missouri charts to calculate head-losses in the pits.

During the field inspection, residents reported flooding issues at the following locations:

- Reaches H010050 to H010020 where overland flow floods the lower level of an apartment building on Kurraba Road;
- Reach H010121 on Aubin Street where overland flow enters a town house;
- Along Raymond Road and Harrieette Street where overland flow overtops driveway access laybacks and enter properties; and
- Reaches H060060 to H060020 where flow passes through private property.

Milsons Park Catchment Management Study 1997 (Reference 7)

North Sydney Council undertook this study in 1997. The primary aim of this study was to investigate the performance of the existing stormwater drainage system and ascertain where future upgrades and extensions would be necessary.

An ILSAX hydrologic model was established for this study. The design storm events analysed were the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI events for the critical duration of 25 minutes.

The Council's Stormwater Drainage Design Guidelines Document (Reference 12) was used to assess the capacity of the existing drainage system. The criteria were as defined in the Anderson Park Catchment Management Study 1996.

Ten locations within the study area were identified as having insufficient capacity. These were:

- Angelo Street;
- Berry Street;
- Clark Road and McDougal Street;
- Doohat Avenue;
- High Street;
- McLaren Street;
- Miller Street;
- Mount Street;
- Pacific Highway; and
- Walker Street.

Rocklands Road Catchment Study 1991 (Reference 8)

This study was undertaken by Willing and Partners Consulting Engineers on behalf of North Sydney Council in 1991. The scope of this study included field inspection of the stormwater drainage system, and hydrologic and hydraulic analyses. The purpose was to assess the performance of the existing stormwater drainage system and identify where future upgrades and extensions would be necessary.

The hydrologic analysis for the study used the ILSAX model. The design storm events analysed were the 10 year, 20 year, 50 year and 100 year ARI events for the critical duration of 20 minutes.

The hydraulic analysis consisted of a hydraulic grade line analysis (in spreadsheet form) using the Colebrook-White equation to estimate head-losses in the pipe and the Missouri charts to calculate head-losses in the pits.

Shellcove Catchment Management Study 1996 (Reference 9)

North Sydney Council undertook this study in 1996. The primary aim of this study was to investigate the performance of the existing stormwater drainage system and ascertain where future upgrades and extensions would be necessary.

An ILSAX hydrologic model was established for this study. The design storm events analysed were the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI events for the critical duration of 25 minutes.

The Council's Stormwater Drainage Design Guidelines Document (Reference 12) was used to assess the capacity of the existing drainage system. The criteria were as defined in the Anderson Park Catchment Management Study 1996.

Eleven locations within the study area were identified as having insufficient capacity. These were:

- Bannerman Street;
- Barry Street;
- Bennett Street;
- Bogota Avenue;
- Burroway Street (corner or Bertha Street);
- Burroway Street (corner of Shell Cove Road);
- Guthrie Avenue;
- Harrison Street;
- Murdoch Street;
- Wycombe Road; and
- Yeo Street.

Walker Street Drainage Study 1995 (Reference 10)

This study was undertaken by RUST PPK Pty Ltd. on behalf of North Sydney Council in 1995. The scope of this study included field inspection of the stormwater drainage system, and hydrologic and hydraulic analyses. The purpose was to assess the performance of the existing stormwater drainage system and identify where future upgrades and extensions would be necessary.

The hydrologic model used for the study was ILSAX. The design storm events analysed were the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI events for the critical duration of 25 minutes.

The Council's Stormwater Drainage Design Guidelines Document (Reference 12) was used to assess the capacity of the existing drainage system. The criteria were as defined in the Anderson Park Catchment Management Study 1996.

The ILSAX model was used to estimate the stormwater drainage augmentations required to meet Council's guidelines. This uses the "pipline full, but not under pressure" assumption. Further analysis on the existing drainage network with the augmentation work recommended was undertaken for the 20 year and 100 year ARI events in the hydraulic model, which was the HYLINE model.

Young Street Catchment Study 1996 (Reference 11)

North Sydney Council undertook this study in 1996. The primary aim of this study was to investigate the performance of the existing stormwater drainage system and ascertain where future upgrades and extensions would be necessary.

An ILSAX hydrologic model was established for this study. The design storm events analysed were the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI events for the critical duration of 25 minutes.

The Council's Stormwater Drainage Design Guidelines Document (Reference 12) was used to assess the capacity of the existing drainage system. The criteria were as defined in the Anderson Park Catchment Management Study 1996.

Nine locations within the study area were identified as having insufficient capacity. These were:

- Belgrave Street;
- Benelong Road;
- Brightmore Street;
- Grasmere Road (corner Young Street);
- Grosvenor Street (corner of Young);
- Illilwa Street (corner Illilwa Lane);
- Reynolds Street (corner Illiliwa Lane);
- Sutherland Street (corner of Young); and
- Winnie Street.

2.3.2. Historical Flood Level Data

2.3.2.1. Council Flood Database

Council provided a database of flood and/or stormwater drainage complaints received from residents. The database spanned the period April 2001 up to November 2014. In this period, 4,214 individual complaints were registered. These complaints were submitted from 1,829 properties, with 1,075 lodging a single complaint and 754 lodging multiple complaints.

The issues lodged predominantly related to stormwater drainage, seepage and drainage infrastructure blocked by tree roots etc.

A comprehensive review of the complaints by Council Officers found that more of them are relevant to flooding but more relate to specific property drainage issues.

2.3.2.2. Sydney Water Flood Database

All records of flooding provided by SWC are summarised in Table 10.

Table 10: Historical Flood Levels – SWC

Date Flooded	Catchment	Location	Property Inundated	Flood Level (m AHD)	Floor Level (m AHD)	Comments
8 November 1984	Catchment B - Brook Street	Wheatleigh Street, Naremburn	Yes	N/A	N/A	Garages flooded – floor level below soffit level of stormwater. 1 in 100 year event
22 February 1986	Catchment B - Brook Street	Wheatleigh Street, Naremburn	Yes	66.39	65.94	Garages flooded – floor level below soffit level of stormwater

2.3.2.3. Newspaper Articles

Newspaper articles were sourced from the public records of the State Library of New South Wales. The editorials were the North Shore Times (articles dated 14 November, 1984; 26 January, 1991; 30 January, 1991; 12 February, 2010; 17 February 2010) and The Mosman Daily (articles dated 13 November, 1984; 15 November 1984).

The articles focused on the storm damages incurred due to a number of causes. The most commonly reported form of damage was due to landslides and retaining walls collapsing. A flood level was estimated for the 1984 event on Rangers Avenue, Mosman. The location of this flood level estimate was approximately 200 m east and outside the study area, although it was part of the Mosman Bay Catchment (Catchment X), of which the North Sydney study area accounts for the upstream portion. There were also reports that the northbound railway line was cut at Waverton in the 1984 event, although it was not known if this was due to electrical faults or flood waters.

The areas identified as having been affected by storm damage within the study area were Bent Street, Merlin Street, Edward Street, Peel Street, Kirribilli Avenue, Cabramatta Road, Berry Street, Lavender Crescent, Woolcott Street, among others. These are shown on Table 11. Relevant historical data is summarised in the table below.

Table 11: Historical Flood Data – Newspaper Articles

Location	Comment
Bent St and Merlin St	Retaining wall collapses due to landslide on Bent St and Merlin St in Neutral Bay
Edward St, Peel St, Kirribilli Ave and Cabramatta Rd	Retaining wall collapses
Berry St	Monte Saint Angelo School retaining wall collapses onto Berry St
Lavender Cres	A road beside Watt Park in Lavender Bay was damaged by the storm. The road collapsed due to water washing away soil from around/underneath the road
Woolcott St	Landslip in Woolcott St (Waverton), where a stairway collapsed
Rangers Avenue	Flood waters 1 m deep, located 200 m east of study area. The rainfall event is approximately 1% AEP event
St Leonards Post Office	St Leonards post office flooded
Kirribilli Ex Servicemen's Club	Kirribilli Ex Servicemen's Club had water on the flat roof seeping through the ceiling
Corner of High St and Hipwood St	Collapse of retaining wall on the corner of High St and Hipwood St, Kirribilli where two telephone booths were slipping into a hole

2.3.2.4. Community Consultation

A community consultation process was undertaken. This included distribution of an information sheet and a questionnaire. The information sheet described the role of the Flood Study in the Floodplain Risk Management Process and the questionnaire requested information pertaining to the community's experience of flooding within the catchment. The distribution assumed a targeted approach whereby properties located within 5 m of stormwater drainage infrastructure were contacted, amounting to approximately 28,000 properties receiving community consultation material.

The response rate was on average 4% across the catchment. This gave ~ 1,100 responses. Approximately 10% of these respondents indicated that they had been affected by flooding/stormwater issues in the past. Furthermore, 24 respondents had their house or property isolated due to flooding and 38 respondents have performed flood mitigation work or emergency work on their property as a result of flooding in the past.

Figure 7A presents some statistics based on the questionnaires received.

Figure 7B maps responses to the questionnaire. Note different colours are shown for those properties previously impacted by flooding and those where flood related mitigation works have been carried out. The map indicates a wide distribution of respondents and those properties indicated as being impacted by flooding in the past tend to correlate with modelled flow paths.

Overall the 4% response rate is relatively low, however this was predictable given that flooding in North Sydney is, for the vast majority of residents, limited in extent and duration and will occur only in extreme rainfall events. The small catchment and high slope nature of the area means that in most locations flood events can come and go within minutes, thus removing opportunities for residents to witness events.

3. STUDY METHODOLOGY

Overall the study methodology focuses on building a best practice modelling system that converts applied rainfall into flow and then into levels, depth and velocities of flow. This entails building both hydrology and hydraulics models. The hydrology model converts applied rainfall into flow, taking into account losses due to infiltration and depression storage. The hydraulic model then utilises a detailed representation of study area topography to route applied flow and the result of this is mapped flood extent, depth and levels to Australian Height Datum (AHD).

A summary of the Flood Study process is shown in Diagram 1. The urbanised nature of the study area with its mix of pervious and impervious surfaces, and existing piped and overland flow drainage systems, has created a complex hydrologic and hydraulic flow regime.

The estimation of flood behaviour in a catchment is undertaken as a two-stage process, consisting of:

1. hydrologic modelling to convert rainfall estimates to overland flow and stream runoff; and
2. hydraulic modelling to estimate overland flow distributions, flood levels and velocities.

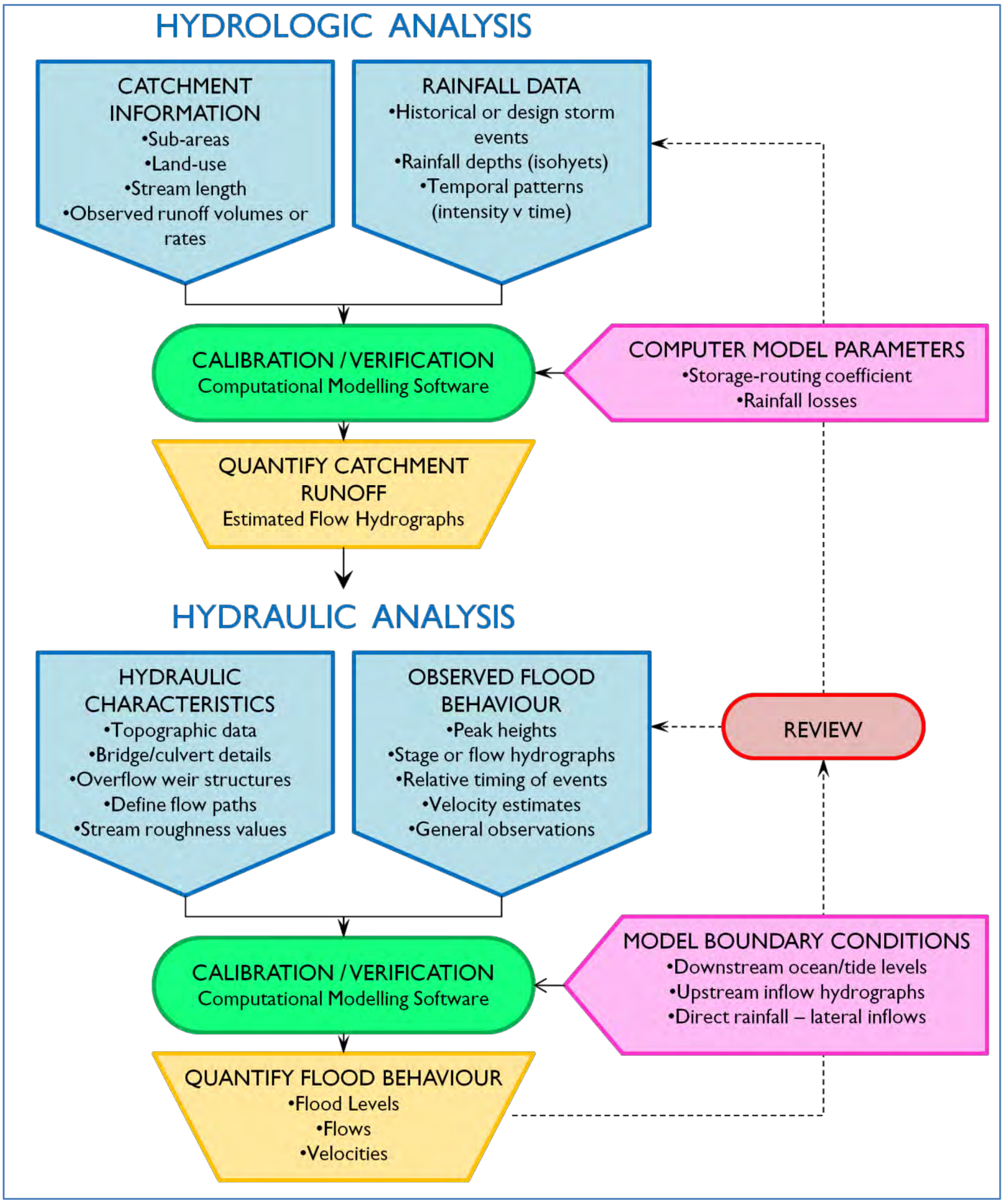
The hydrologic model, DRAINS, was built and used to create flow boundary conditions for input into a two-dimensional unsteady flow hydraulic model, i.e. TUFLOW. Parameters were informed by site conditions as well as by previous model build work in other areas of the Sydney Metropolitan area.

Good historical flood data facilitates calibration of the models and increases confidence in the estimates. The calibration process involves modifying the initial model parameter values to produce modelled results that concur with observed data. Validation is undertaken to ensure that the calibration model parameter values are acceptable in other storm events with no additional alteration of values. Recorded rainfall and stream-flow data are required for calibration of the hydrologic model, while historic records of flood levels, velocities and inundation extents can be used for the calibration of hydraulic model parameters. In the absence of such data, model verification is the only option and a detailed sensitivity analysis of the different model input parameters constitutes current best practice.

There are no stream-flow records in the catchment, so the use of a flood frequency approach for the estimation of design floods or independent calibration of the hydrologic model was not possible.

The broad approach adopted for this study was to use a widely utilised and well-regarded hydrologic model to conceptually model the rainfall concentration phase (including runoff from roof drainage systems, gutters, etc.). The hydrologic model used design rainfall patterns specified in AR&R (1987) and the runoff hydrographs were then used in a hydraulic model to estimate flood depths, velocities and hazard in the study area.

Diagram 1: Flood Study Process



The sub-catchments in the hydrologic model were kept small (on average approximately 1 ha) such that the overland flow behaviour for the study was generally defined by the hydraulic model. This joint modelling approach was verified as possible with this work detailed in Section 6.

3.1. Hydrologic Model

DRAINS is a hydrologic/hydraulic model that can simulate the full storm hydrograph and is capable of describing the flow behaviour of a catchment and pipe system for real storm events, as well as statistically based design storms. It is designed for analysing urban or partly urban catchments where artificial drainage elements have been installed.

The DRAINS model is broadly characterised by the following features:

- the hydrological component is based on the theory applied in the ILSAX model which has seen wide usage and acceptance in Australia;
- its application of the hydraulic grade line method for hydraulic analysis throughout the drainage system; and
- the graphical display of network connections and results.

DRAINS generates a full hydrograph of surface flows arriving at each pit and routes these through the pipe network or overland, combining them where appropriate. Consequently, it avoids the "partial area" problems of the Rational Method and additionally it can model detention basins (unsteady flow rather than steady state).

Runoff hydrographs for each sub-catchment area are calculated using the time area method and the conveyance of flow through the drainage system is then modelled using the Hydraulic Grade Line method. Application of the Hydraulic Grade Line method is recommended for the design of pipe systems in AR&R (1987). The method allows pipes to operate under pressure or to "surcharge", meaning that water rises within pits, but does not necessarily overflow out onto streets. This provides improved prediction of hydraulic behaviour, consistency in design, and greater freedom in selecting pipe slopes. It requires more complicated design procedures, since pipe capacity is influenced by upstream and downstream conditions.

DRAINS cannot however adequately account for an elevated downstream tailwater level which would drown out the lower reaches of a drainage system (it can if the upstream pit is above the tailwater level but not if it is below). For this reason flooding within reaches affected by elevated water levels is more accurately assessed using the TUFLOW model.

It should be noted that DRAINS is not a true unsteady flow model and therefore does not account for the attenuation effects of routing through temporary floodplain storage (down streets or in yards). As such the use of DRAINS within the study is limited to some minor upstream routing and development of hydrological inputs into the downstream TUFLOW model.

3.2. Hydraulic Model

The availability of high quality LIDAR/ALS data means that the study area is suitable for two-dimensional (2D) hydraulic modelling. Various 2D software packages are available and the TUFLOW package was adopted as it is widely used in Australia and WMAwater have extensive experience with the model.

The TUFLOW modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water flow equations in two dimensions. The TUFLOW software has been widely used for a range of similar projects. The model is capable of dynamically simulating complex overland flow regimes. It is especially applicable to the hydraulic analysis of flooding in urban areas which is typically characterised by short duration events and a combination of supercritical and subcritical flow behaviour.

The study area consists of a wide range of land-uses, with residential, commercial and open space areas. For this catchment, the study objectives require accurate representation of the sub-surface and overland flow system including kerbs and gutters and defined drainage controls.

For the hydraulic analysis of complex overland flow paths (such as the present study area where overland flow occurs between and around buildings), an integrated 1D/2D model such as TUFLOW provides several key advantages when compared to a 1D only model. For example, a 2D approach can:

- provide localised detail of any topographic and/or structural features that may influence flood behaviour,
- better facilitate the identification of the potential overland flow paths and flood problem areas,
- dynamically model the interaction between hydraulic structures such as culverts and complex overland flowpaths; and
- inherently represent the available floodplain storage within the 2D model geometry.

Importantly, a 2D hydraulic model can better define the spatial variations in flood behaviour across the study area. Information such as flow velocity, flood levels and hydraulic hazard can be readily mapped across the model extent. This information can then be easily integrated into a GIS based environment enabling the outcomes to be readily incorporated into Council's planning activities. The model developed for the present study provides a flexible modelling platform to properly assess the impacts of any overland flow management strategies within the floodplain (as part of the ongoing floodplain management process).

In TUFLOW the ground topography is represented as a uniformly-spaced grid with a ground elevation and a Manning's "n" roughness value assigned to each grid cell. The grid cell size is determined as a balance between the model result definition required and the computer run time (which is largely determined by the total number of grid cells).

4. HYDROLOGIC MODEL

The hydrologic model converts applied rainfall depth (distributed over time) into a rainfall excess and then into a flow. This flow is then applied to the hydraulic model for routing. The following section details the method utilised in the build of the hydrologic model. This includes detailed hydrologic modelling of the urban extent within the study area and coarser modelling of upstream areas that lie outside the LGA but then subsequently flow into the study area.

4.1. Sub-catchment Definition

Sub-catchment definition was undertaken for local catchment flows as well as those areas external to the study area, which nevertheless flow into the catchment under consideration. The local catchment flow area was the North Sydney LGA study area, in which sub-catchments were delineated to a finer resolution. Out-of-study-area flows that entered or bordered the LGA were located to the north-west (Catchment B–V), south-west (Catchment C-G-P) and north-east (Catchment L). These sub-catchments were delineated to a courser resolution as they were outside of the study area (see Figure 8 for sub-catchment schematisation). The out-of-catchment flow area to the south-east was not delineated as the study area did not receive flows from this area; rather the study area discharged flow into this area.

The study area represented by the DRAINS model is 1,050 ha in size. This area has been represented by a total of 1231 sub-catchments giving an average sub-catchment size of approximately 1 ha.

The upstream catchment area located to the north-west of the study area (that discharges east into Tunks Park) was represented by the WBNM model and is 6.2 km^2 in size. This area has been represented by a total of 24 sub-catchments giving an average sub-catchment size of approximately 0.26 km^2. A coarser representation of this upstream area that lies outside the study area is appropriate as detailed hydraulic modelling is not carried out for such areas.

Overall the sub-catchment delineation ensures that where hydraulic controls exist, they are accounted for and able to be appropriately incorporated into hydraulic routing.

4.2. DRAINS

The DRAINS model is well known and widely used in NSW for urban hydrology and hydraulics modelling. In this study only the hydrologic component of DRAINS is used. The study area is broken into high resolution sub-catchments and these are modelled in DRAINS. The following sections provide details on DRAINS parameters utilised in the modelling.

4.2.1. Impervious Surface Area

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete surfaces occur significantly faster than from vegetated surfaces. This results in a faster concentration of flow within the downstream area of the catchment, and increased peak flow in some situations. It is therefore necessary to estimate the proportion of the catchment area that is covered by such surfaces.

DRAINS categorises these surface areas as either:

- paved areas (impervious areas directly connected to the drainage system),
- supplementary areas (impervious areas not directly connected to the drainage system, instead connected to the drainage system via the pervious areas), and
- grassed areas (pervious areas).

Within the LGA, a uniform 5% was adopted as a supplementary area across the catchment. The remaining 95% was attributed to impervious (or paved areas) and pervious surface areas, as estimated for each individual sub-catchment. This was undertaken by determining the proportion of the sub-catchment area allocated to a land-use category and the estimated impervious percentage of each land-use category, summarised in Table 12.

Table 12: Impervious Surface Area

Land-use Category	Impervious Percentage
Residential Property	50% Impervious
Commercial Property	95% Impervious
Vacant Land	0% Impervious
Vegetation (such as public parks)	0% Impervious
Roadway	100% Impervious

Photo 2 and Photo 3 show representative areas that were inspected in the process of deriving the impervious percentages presented in Table 12.

Photo 2: Impervious Percentage – Residential Area Example

Photo 3: Impervious Percentage – Commercial Area Example

4.2.2. Rainfall Losses

Methods for modelling the proportion of rainfall that is “lost” to infiltration are outlined in AR&R (1987). The methods are of varying degrees of complexity, with the more complex options only suitable if sufficient data are available. The method most typically used for design flood estimation is to apply an initial and continuing loss to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

Rainfall losses from a paved or impervious area are considered to consist of an initial loss (an amount sufficient to wet the pavement and fill minor surface depressions). Losses from grassed areas are comprised of an initial loss and a continuing loss. The continuing loss is calculated from an infiltration equation curve incorporated into the model and is based on the selected representative soil type and antecedent moisture condition. The catchment soil was assumed to have a slow infiltration rate and the antecedent moisture condition was considered to be rather wet. This produces conservative flow estimates, which are appropriate given that flood risk is being assessed.

The adopted parameters are summarised in Table 13. These are consistent with the parameters adopted in similar studies in the Sydney Metropolitan area.

Table 13: Adopted DRAINS parameters – Rainfall Losses

RAINFALL LOSSES	
Paved Area Depression Storage (Initial Loss)	1.0 mm
Grassed Area Depression Storage (Initial Loss)	5.0 mm
SOIL TYPE	**3**
Slow infiltration rates. This parameter, in conjunction with the AMC, determines the continuing loss	
ANTECEDENT MOISTURE CONDITONS (AMC)	**3**
Description	Rather wet
Total Rainfall in 5 Days Preceding the Storm	12.5 to 25 mm

4.3. WBNM Parameters

WBNM is a well-known and widely used NSW based hydrologic model for the conversion of rainfall into flow. It incorporates both a loss model and routing calculations. A key feature of WBNM is that it has been developed empirically using stream gauges located in South East Australia. As such there is less reliance on calibration of model parameters to achieve reasonable results, although where possible calibration and validation of model parameters is preferable.

For the study the WBNM hydrologic runoff-routing model was used to determine hydraulic model inflows from catchment areas upstream of the study area. These areas, as they are not modelled in the hydraulic model, can be described more coarsely than areas in the study area.

The model input parameters for each sub-catchment are:

- a lag factor (termed C), which can be used to accelerate or delay the runoff response to rainfall;
- a stream-flow routing factor, which can speed up or slow down concentrated flows occurring through each catchment;
- rainfall initial and continuing losses to represent infiltration and filling of depression storage; and
- the percentage of catchment area with a pervious/impervious surface.

4.3.1. Lag Parameter

The lag parameter affects the time taken for rainfall over a sub-catchment to collect and concentrate into runoff flows at the bottom of the sub-catchment. Lag times for runoff depend on several physical catchment characteristics, including area, shape and steepness (among other factors) for natural catchments. Experimental data for natural catchments in Australia has demonstrated that the dominant factor affecting lag is catchment area, with other characteristics showing strong correlation with area such that there is a strong case for catchment lag to be determined on area alone.

Since the relationship includes the effect of catchment area and flood magnitude, a similar value of the Lag Parameter (C) should apply to a wide range of catchment and flood sizes (Boyd et al, 2007). Experimental derivation of the Lag Parameter for 129 storms on 10 catchments in eastern NSW found that a value of 1.68 gave a good fit to all the data. A value of 1.7 was adopted for design flood modelling in this study, in agreement with the NSW data.

4.3.2. Stream-flow Routing Parameter

WBNM provides the option to route flows to the bottom of a sub-catchment via nonlinear routing, time-delay routing and Muskingum routing. This routing is required to estimate the attenuation and timing of flows from sub-catchments in the steep upper catchment areas that are not included in the hydraulic model extent. The nonlinear method was adopted for this study. For this method, Boyd et al (2007) recommends values of 1.0 for natural channels, which was adopted for sub-catchments in the upstream area.

4.3.3. Impervious Surface Area

The upstream sub-catchment areas adopted the same impervious surface area parameters as were applied in the local sub-catchment areas, described in Section 4.2.1 and Table 12.

4.3.4. Rainfall Losses

Table 6.2 of AR&R (1987) recommends that for catchments east of the dividing range in New South Wales, an initial loss of 10 mm to 35 mm is appropriate, with a continuing loss of 2.5 mm/hr. For this study, the initial loss of 10 mm and the continuing loss of 2.5 mm/hr were adopted, as it is a typical value used in similar studies. Using the lower bound of recommended values has been found to address issues with low total runoff volumes for the AR&R design burst rainfall patterns. The design storms do not contain antecedent rainfall, whereas real storm bursts are often preceded by a period of lower intensity rainfall, which would wet the catchment and reduce infiltration during the peak storm burst.

5. HYDRAULIC MODEL

In a flood study the hydraulic model is used to route applied flow. Applied flow comes from the hydrologic model described in the previous section. The hydraulic models routes applied flow on the basis of gravity and "roughness" (for channel and overland flow) and on the basis of pipe capacity where it enters the sub-surface drainage system. Equations of flow are incorporated into the hydraulic model in order to achieve this.

A key input to the hydraulic model is the data which describes the study area topography and this data is described in the section below (the background to this data was also discussed in Section 2). Other important data includes the pit/pipe database and aerial photographs and LEP mapping which together are used to describe different areas of "roughness".

5.1. Model Topography

Given the objectives and requirements of the study and the availability of ALS data, a 2D overland flow hydraulic model is the most suitable model to effectively assess flood behaviour.

The model uses a regularly spaced computational grid, with a cell size of 2 m by 2 m. This resolution was adopted as it provides an appropriate balance between providing sufficient detail for roads and overland flow paths, while still resulting in workable computational run-times. The model grid was established by sampling from a 1 m by 1 m DEM. This DEM was generated from a triangulation of filtered ground points from the LiDAR dataset, discussed in Section 2.2.1. This DEM is shown in Figure 2.

5.2. Boundary Locations

5.2.1. Inflows

For local sub-catchments, local runoff hydrographs were extracted from the DRAINS hydrologic model (see Section 4). These were applied to the downstream end of the sub-catchments within the 2D domain of the TUFLOW hydraulic model. The local inflow locations typically corresponded with inlet pits on the roadway as this is where most rainfall runoff is directed.

Flows originating from outside of the study area were routed in the WBNM hydrologic model and applied at the hydraulic model boundary, located where Flat Rock Drive traverses the flow path.

5.2.2. Downstream Boundary

The downstream boundary was located in Sydney Harbour to the south and Middle Harbour to the north of the study area.

5.3. Roughness Co-efficient

"Roughness" in a modelling context refers to the way different areas in a catchment impact on ease of flow. For example a road is smooth, a grassed area less so and an area planted with shrubs less so again.

The TUFLOW model used for this study utilises the Manning's formulation to determine the energy loss from friction and other sources. The roughness coefficient, '*n*', is an empirically derived parameter which represents the retarding force applied to flowing water by the channel bed or ground surface. In practice, in computational modelling of real systems, this parameter often also incorporates other sources of energy loss such as turbulence and flow expansion/contraction from non-uniform cross sections.

Inspection of the aerial photography was used to classify various land-uses categories, such as urban areas and vegetated areas. From this, spatially varying roughness values were applied to the model, based upon these differing categories. The roughness values adopted for the hydraulic model are shown in Table 14 and Figure 10.

The values are consistent with typical values in the literature (Chow, 1959 and Henderson, 1966), industry guidelines (*AR&R Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains Report*, Engineers Australia, 2012) and previous experience with modelling similar catchment conditions. The sensitivity of model results to changes in the roughness values are discussed in Section 8.3.2.

Table 14: Manning's 'n' Values

Surface Type	Manning's 'n' Value
Concrete-lined pipes	0.015
Roads and paved surfaces	0.025
Urban areas – general overland areas, gardens, roadside verges, low density residential lots etc	0.05
Light density vegetation (short grass or sparse vegetation)	0.04
Medium density vegetation	0.07
High density vegetation	0.10
Waterways, such as Lakes, Estuaries and Ocean areas	0.03
Default	0.05

5.4. Hydraulic Structures

5.4.1. Buildings

Buildings and other significant features likely to act as flow obstructions were incorporated into the model network based on building footprints, defined using aerial photography. These types of features were modelled as impermeable obstructions to the floodwaters.

5.4.2. Fencing and Obstructions

Smaller localised obstructions within or bordering private property, such as fences, were not explicitly represented within the hydraulic model, due to the relative impermanence of these features. The cumulative effects of these features on flow behaviour were assumed to be addressed partially by the adopted roughness parameters.

5.4.3. Bridges

Key hydraulic structures were included in the hydraulic model. Culverts and bridges were modelled as 1D features within the 1D open channels, with the purpose of maintaining continuity within the model. All other bridges that contribute to the conveyance of flow were modelled in the 2D domain using a TUFLOW feature specifically designed for this purpose, whereby the energy losses and blockage caused by any piers and the deck can be applied directly to the grid cells.

The modelling parameter values for the culverts and bridges were based on the geometrical properties of the structures, which were obtained from previous reports, details (including measurements) taken during site inspections, and previous experience modelling similar structures.

5.4.4. Sub-surface Drainage Network

Figure 9 shows the location and extent of drainage lines within the study catchment that have been included in the TUFLOW model. The drainage system defined in the model comprises 2,520 pits and junctions, and 2,334 pipe sections.

5.4.5. Blockage Assumptions

Blockage of hydraulic structures can occur with the transportation of a number of materials by flood waters. This includes vegetation, garbage bins, building materials and cars; however there is great disparity in materials that may be mobilised within a catchment.

Debris availability and mobility can be influenced by factors such as channel shear stress, height of floodwaters, severity of winds, storm duration and seasonal factors relating to vegetation. The channel shear stress and height of floodwaters that influence the initial dislodgment of blockage materials are also related to the average exceedance probability (AEP) of the event. Storm duration is another influencing factor, with the mobilisation of blockage materials generally increasing with increasing storm duration (Barthelmess and Rigby 2009, cited in Engineers Australia 2013).

The potential effects of blockage include:

- decreased conveyance of flood waters through the blocked hydraulic structure or drainage system;
- variation in peak flood levels;
- variation in flood extent due to flows diverting into adjoining flow paths; and

- overtopping of hydraulic structures.

Existing practices and guidance on the application of blockage can be found in:

- the Queensland Urban Drainage Manual (Department of Natural Resources and Water, 2008);
- AR&R Revision Project 11 Blockage of Hydraulic Structures (Engineers Australia, 2013); and
- the policies of various local authorities and infrastructure agencies.

The guidelines proposed by the AR&R Revision Project 11 utilise generic blockage factors presented in Table 15.

Table 15: Suggested ‘Design’ and ‘Severe’ Blockage Conditions for Various Structures (Engineers Australia, 2013)

Type of structure		Blockage conditions	
		Design blockage	**Severe blockage**
Sag Kerb Inlet	Kerb slot inlet only Grated inlet only Combined inlets	0/20% 0/50% [1]	100% (all cases)
On-grade kerb inlets	Kerb slot inlet only Grated inlet only (longitudinal bars) Grated inlet only (transverse bars) Combined inlets	0/20% 0/40% 0/50% [2]	100% (all cases)
Field (drop) inlets	Flush mounted Elevated (pill box) horizontal grate Dome screen	0/80% 0/50% 0/50%	100% (all cases)
Pipe inlets and waterway culverts	*Inlet height < 3m and width < 5m* Inlet Chamber	0/20% [3]	100% [4]
	Inlet height > 3m and width > 5m Inlet Chamber	0/10% [3]	25% [3]
	Culverts and pipe inlets with effective debris control features	As above	As above
	Screened pipe and culvert inlets	0/50%	100%
Bridges	Clear opening height < 3 m Clear opening height > 3 m Central piers	[5] 0% [7]	100% [6] [7]
Solid handrails and traffic barriers associated with bridges and culverts		100%	100%
Fencing across overland flow paths		[8]	100%
Screened stormwater outlets		100%	100%

Current modelling has been undertaken assuming no blockage of pipes, culverts and bridges greater than 450 mm in diameter. Pipes less than 450 mm in diameter were conservatively assumed to be completely blocked. The sensitivity of model results to changes in the blockage values (for pipes > 450 mm in diameter) are discussed in Section 8.3.

6. MODEL VERIFICATION

6.1. Introduction

Typically model accuracy (the ability of the computer model to be representative of the physical system it has been built to emulate) is established via a calibration/validation process.

In a flood study specifically, model calibration is the process whereby observed rainfall data (and water levels where applicable) are applied to the model and its ability to replicate observed flood behaviour is assessed. Note model parameters can be adjusted iteratively within reasonable ranges in order to improve model fit to observed data.

Validation is similar except that observed input data is applied blind to a model and its ability to replicate observed behaviour is assessed without recourse to parameter adjustment.

The model calibration and validation paradigm is best practice however frequently there is a lack of data/events to support the process.

In the absence of suitable data to facilitate model calibration/validation verification of the model is carried out. Model verification describes any and all efforts or comparisons carried out in order to increase confidence in the built modelling system.

The following section on model verification methods describes various verification data that has been collected and how this compares to preliminary model estimates.

The following work has been carried out in order to establish the veracity of the modelling work reported upon herein:

- Comparison of model flow estimates for the 1% AEP event with known ranges from other Sydney Metropolitan Studies using identical modelling techniques and in some cases calibrated and validated models;
- Comparing known flood locations in the study area (based on historic events, newspapers and previous catchment studies) with model results for preliminary design events; and
- Comparing Sydney Water and Council flood complaint databases with mapped inundation.

A further aspect of model verification is having experienced Council staff review modelling work for known flooding “hotspots”. This work is presented in Section 7.

6.1.1. Design Unit Flow Rate Estimates

In the absence of other data, model design flow estimates can be compared to those from a calibrated/validated catchment subject to comparable design rainfall estimates and composed from similar drainage elements.

WMAwater have previously undertaken a detailed flood study for Hawthorne Canal in the inner west of Sydney. The centroid of the Hawthorne Canal catchment is located six kilometres south and west of the North Sydney LGA centroid. In comparing 1% AEP design rainfall estimates a discrepancy of 2.3% is noted. Both catchments are highly developed with major/minor drainage systems, as well as sophisticated intra-lot drainage works. North Sydney tends to have higher slopes than the Hawthorne Canal catchment.

Historic rainfall events occurred in the Hawthorne Canal catchment in the years 1986, 1993 and 2012. Suitable observed data was available to facilitate calibration/validation of the model system used. The calibrated/validated model produced 1% AEP design flow estimates in the order of 0.5 $m^3/s/ha$.

Note that in the various urban catchments WMAwater have modelled in detail within the Sydney Metropolitan area, the 1% AEP design flows have tended to range from 0.4 $m^3/s/ha$ to 0.6 $m^3/s/ha$.

For each of the four quadrants that comprise the study area, three locations have been selected (shown on Figure 12A) and unit flow rates calculated. These have then been examined in light of expectations as per above estimates from Hawthorne Canal and shown in Table 16.

Table 16: Verification – 1% AEP Event Unit Flow Rate

Area (ha)	**Peak Discharge (m^3/s)**	**Unit Flow Rates ($m^3/s/ha$)**
North Catchment		
7.5	4.1	0.55
18.5	8.5	0.46
39.5	16.9	0.43
South Catchment		
5.7	3.3	0.58
4.2	2.2	0.52
1.9	1.3	0.64
East Catchment		
14.8	7.5	0.50
5.1	2.8	0.54
3.8	2.1	0.56
West Catchment		
6.3	3.6	0.49
13.3	6.7	0.50
13.4	6.9	0.52

Average is 0.52 $m^3/s/ha$ and standard deviation is 0.06 $m^3/s/ha$.

Generally, larger upstream catchment areas produced lower unit flow rates due to the cumulative effects of flow attenuation, with flow through private property (perpendicular to the roadway), parks that act as detention basins and other obstructions. This relationship between unit rate flow and upstream catchment area is shown on Figure 12B.

Overall the results are well in range and hence are indicative of a good model build and good design flow estimates.

6.1.2. Historic Information

Various forms of information are available to identify flood prone areas within the study area for subsequent checking against preliminary model results. Suitable information sources identified through the course of the study are as follows:

- Newspapers – The North Shore Times and The Mosman Daily (Gazette) were examined at the State Library and locations where flooding was identified noted (further information provided in Section 2.3.2.3);
- Council Records – Council maintains a database of flooding/stormwater related complaints and these have been examined for locations suitable for use in model verification work; and
- Community Consultation – refer to Section 2.3.2.4 for complete details; in summary, the exercise involved a mailout of ~ 28,000 questionnaires, with ~ 1,100 replies received and ~100 residents indicating knowledge or experience of a flooding issue. These results have been mapped and are the main source of historical data used in model verification work (against historic events at least).

A form of model verification is then to compare those locations identified via information sources noted above with model predictions. This work is presented in Figure 13.

Generally, what is observed is a good correlation between modelled flood behaviour observations. The match is in Table 17 presented below.

Table 17: Historic Flooding Verification

	Year of Recorded Flooding	**1% AEP Event Affectation**
Anderson Park	1984	Yes
Baden Rd	1991, 2010	No
Bank St	1984, 1991, 2010	Yes
Bardsley Gdns	1984	Yes
Ben Boyd Rd	2010	Yes
Byrnes Av	1984	Yes
Clark Rd	1984	Yes
Corner Euroka St and Woolcott St	1984	Yes
Fall St	1984	Yes
Gerard St	1991	Yes
Gillies St	2010	Yes
Hampden St	1984, 1991, 2010	Yes
Hazelbank Rd	2010	Yes
High St	1984	Yes
Hipwood St	1984	Yes
Illiliwa St	2010	Yes
Morton St	2010	Yes
Mount St	1984	Yes
Queens Av	2010	No
Thrupp St	1991	Yes

	Year of Recorded Flooding	1% AEP Event Affectation
Tunks Park	1984	Yes
Walker St	2010	Yes
Westleigh La	1984	Yes
Woolcott St	1984	Yes

6.1.3. Catchment Studies

The catchment studies summarised in Section 2.3.1.4 identified a total of 62 locations where some sort of flooding issue exists (or existed). Precise locations could not always be identified however generally the reports identified specific locations that could be mapped.

These locations have been used to interrogate the preliminary 1% AEP results. This information is presented in Table 18 below. Of 62 locations only 1 was found not to be subject to any flooding. Examining the map of the points it can be clearly seen that the majority of them lie on flow paths which have been identified by the hydraulic model.

Table 18: Flooding Issue Locations Identified Through Previous Studies

Location	Report Section	Flood Depth (m)
Ben Boyd Road	2.5.2	0.01
Bent Street	2.5.2	0.54
Bray Street	2.5.2	0.86
Colindia Street	2.5.2	Not Flooded
Doris Street	2.5.2	0.22
Forsyth Park	2.5.2	4.29
Holdsworth Road	2.5.2	0.56
Kurraba Road	2.5.2/2.5.6	0.07
McLaren Street	2.5.2/2.5.7	0.29
Merlin Street	2.5.2	0.19
Montpelier Street (Cnr Premier)	2.5.2	0.04
Cnr Montpelier and Spruson	2.5.2	0.02
Neutral Street	2.5.2	Not Flooded
Premier Street	2.5.2	0.22
Ridge Street	2.5.2	Not Flooded
Spruson Street (near Holdsworth Road)	2.5.2	0.03
Walker Street (corner of Hampden Street)	2.5.2	Not Flooded
Winter Avenue	2.5.2	2.42
Wyagdon Street	2.5.2	0.01
Willoughby Road	2.5.3	0.40
Hume Lane	2.5.3	1.98
Chandos Street	2.5.3	0.17
Wheatlegh Street	2.5.3	0.75
Brook Street	2.5.3	1.94
McHatton Street	2.5.4	0.03
Carr Street	2.5.4	0.21
Bay Road	2.5.4	0.74
Woolcott Street	2.5.4	0.36
Aubin Street	2.5.6	0.37

Location	Report Section	Flood Depth (m)
Raymond Road	2.5.6	0.21
Harriette Street	2.5.6	0.03
Walker Street	2.5.7	0.26
Angelo Street	2.5.7	0.15
Berry Street	2.5.7	0.18
Clark Road and McDougal Street	2.5.7	1.63
Doohat Avenue	2.5.7	Not Flooded
High Street	2.5.7	0.40
Miller Street	2.5.7	0.39
Mount St	2.5.7	0.34
Pacific Highway	2.5.7	0.17
Bannerman Street	2.5.9	0.54
Barry Street	2.5.9	1.29
Bennett Street	2.5.9	0.20
Bogota Avenue	2.5.9	1.56
Burroway Street (corner Bertha)	2.5.9	0.44
Burroway Street (corner of Shell Cove Road)	2.5.9	0.21
Guthrie Avenue	2.5.9	0.26
Harrison Street	2.5.9	0.23
Murdoch Street	2.5.9	0.22
Wycombe Road	2.5.9	0.19
Yeo Street	2.5.9	1.06
Belgrave Street	2.5.11	0.62
Benelong Road	2.5.11	0.32
Brightmore Street	2.5.11	0.58
Grasmere Road (corner Young St)	2.5.11	0.24
Grosvenor Street (corner of young)	2.5.11	0.32
Illiliwa Street	2.5.11	0.55
Reynolds Street	2.5.11	0.18
Sutherland Street	2.5.11	0.48
Winnie Street	2.5.11	0.29

The good correlation of these locations with preliminary design model results is supportive of the modelling work. It indicates that the model is representing flooding where it should which further enhances confidence in modelling work.

6.2. Summary

Three methods have been used to verify the modelling work undertaken for the 10.9 km^2 study area.

A comparison of 12 location's flow rates (on a per unit area basis) with expected 1% AEP unit rate flows was undertaken. This found a good match with the average result being 0.52 $m^3/s/ha$. Taking into account a standard deviation of 0.06 $m^3/s/ha$ the result is well within the expected range of 0.4 to 0.6 $m^3/s/ha$ and at the upper end. This makes sense given the location (Sydney's Lower North Shore) and steep slopes with a mean of 10%.

Locations identified as prone to some flooding via analysis of Community Consultation results, Council and Sydney Water records and newspapers were identified and mapped. These were then compared to the 1% AEP model results. This work established that the model does indeed find flooding at these locations.

Further, previous studies commissioned by Council were used to identify 62 locations which had previously been the focus of drainage assessments and hence presumably have some flooding issue. 61 of these 62 locations were found to be flood prone via modelling and it is probable that for those locations with no flooding that either the issue was more stormwater related, and hence occurs at a scale that is below the threshold for modelling in this study (i.e. is not flooding), or that given the approximate location provided in the report a mismatch has occurred.

Overall verification work (which was used iteratively in order to improve model build over the course of the project) can be said to establish that the model accurately reflects observed flood behaviour.

In Section 7, modelling results for four hot spots are examined in some detail. It is hoped that Council review of these may further reinforce the representativeness of the modelling work to date or provide feedback which can be used to enhance overall model accuracy.

7. DESIGN EVENT MODELLING

7.1. Overview

There are two basic approaches to determining design flood levels, namely:

- *flood frequency analysis* – based upon a statistical analysis of the flood events, and
- *rainfall and runoff routing* – design rainfalls are processed by hydrologic and hydraulic computer models to produce estimates of design flood behaviour.

The *flood frequency* approach requires a reasonably complete homogenous record of flood levels and flows over a number of decades to give satisfactory results. No such records were available within this catchment. For this reason a *rainfall and runoff routing* approach using DRAINS model results was adopted for this study to derive inflow hydrographs for input to the TUFLOW hydraulic model, which determines design flood levels, flows and velocities. This approach reflects current engineering practice and is consistent with the quality and quantity of available data.

7.2. Critical Duration

To determine the critical storm duration for various parts of the catchment, modelling of the 1% AEP event was undertaken for a range of design storm durations from 15 minutes to 6 hours, using temporal patterns from AR&R (1987). An envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

Additionally, the critical storm duration was determined for the PMF event for a range of storm durations, ranging from 15 minutes to 6 hours. Similarly, an envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

In the north model, it was found that a combination of the 25 minute, 1 hour, 1.5 hour and 2 hour storm durations were critical. The 25 minute design storm duration was typically critical in areas of shallow depths less than 0.15 m. The 2 hour design storm duration was critical through Flat Rock Creek to Tunks Park and Brightmore Reserve, which have either a large upstream catchment (from outside the study area) or were a flood storage area, respectively. The 1 hour and 1.5 hour storm durations occurred alternatively along the flow path areas. The peak flood level difference between the 1 hour and the 1.5 hour storm duration was found to be negligible, within ± 0.03 m. The 1 hour storm duration was adopted as the critical duration in the 1% AEP event for the north model.

For the PMF event in the north model, it was found that a combination of the 15 minute, 1 hour and 2 hour storm durations were critical. The 2 hour design storm duration was critical along the flow path from downstream of the Warringah Freeway to Grafton Street and the upper portion of Primrose Park. The 1 hour storm duration was critical in the area upstream of Warringah Freeway to halfway along Lytton Street, as well as the vegetated channel from Flat Rock Drive to Strathallen Avenue and through Tunks Park. The 15 minute design storm duration was critical across the remained of the north model; including the flow path from Brook Street to Strathallen Avenue, the flow path from Military Road to Brightmore Reserve via Young Street and the flow path from Miliary Road to Brightmore Reserve via Benelong Road. The peak flood level difference between the 1 hour and the 2 hour storm durations was found to be within ± 0.2 m; and each of the aforementioned durations compared to the 15 minute storm duration resulted in peak flood level differences up to ± 0.6 m. For these reasons it was considered appropriate to adopt an envelope of the 15 minute, 1 hour and 2 hour design storm durations for the PMF event in the north model.

In the south model, it was found that a combination of the 25 minute, 1.5 hour and 2 hour design storm durations were critical for the 1% AEP event. The 25 minute design storm duration was typically critical in areas of shallow depths less than 0.15 m. The 1.5 hour and 2 hour storm durations occurred alternatively along the flow path areas. The peak flood level difference between the 1.5 hour and the 2 hour storm duration was found to be negligible, within ± 0.02 m. The 2 hour storm duration was adopted as the critical duration in the 1% AEP event for the south model.

For the PMF event in the south model, it was found that a combination of the 15 minute, 30 minute and 1 hour design storm durations were critical. The 1 hour design storm duration was critical from Warringah Freeway to Sydney Harbour, via Milson Park and the open channel. The 30 minute storm duration was critical along the Warringah Freeway upstream of the Milson Park flow path. Across the majority of the western model the 15 minute storm duration was critical, including the open channel flow path through Anderson Park. The peak flood level difference showed the 1 hour event was higher than the 30 minute event by up to 0.06 m whereas the converse was marginal; therefore the 30 minute storm duration was discarded for adoption. The peak flood level difference between the 15 minute and the 1 hour storm duration was found to be within ± 0.2 m. For these reasons it was considered appropriate to adopt an envelope of the 15 minute and 1 hour design storm durations for the PMF event in the south model.

In the east model, it was found that a combination of the 25 minute and 1 hour design storm durations were critical for the 1% AEP event. The 25 minute design storm duration was critical across a greater portion of the area; comprising of shallow depths less than 0.15 m and the flow paths in the upstream areas of the east model. The 1 hour design storm duration was critical in the flow path downstream of Bertha Road. However, the peak flood level difference between the two durations was within ± 0.02 m across 99% of the area affected by the two durations. Therefore, it was considered appropriate to adopt the 25 minute design storm for events up to and including the 1% AEP event in the east model. For the PMF event, it was found that the 15 minute storm duration was critical across the east model.

In the west model, it was found that a combination of the 25 minute, 1 hour and 2 hour design storm durations were critical for the 1% AEP event. The 25 minute design storm duration was typically critical in areas of shallow depths less than 0.15 m. The 1 hour and 2 hour storm durations were critical in the flow path areas; with the 1 hour critical in the upstream portion of the model and the 2 hour critical in the downstream portion of the model. The 1 hour storm duration was adopted as the critical duration in the 1% AEP event for the west model.

For the PMF event in the west model, it was found that a combination of the 15 minute, 30 minute and 2 hour design storm durations were critical. The 2 hour design storm duration was critical in moderate to high depth flood accumulation areas located upstream of railway embankments that obstructed flow paths. The 30 minute design storm duration was critical along the primary flow path from Newlands Reserve to Balls Head Bay. Across the majority of the west model the 15 minute storm duration was critical, including primary flow paths discharging into Gore Cove and Berrys Bay. The peak flood level difference between these three durations was generally within 0.1 m to 0.2 m in the primary flow path areas. For these reasons it was considered appropriate to adopt an envelope of the 15 minute, 30 minute and 2 hour design storm durations for the PMF event in the west model.

Based on this outcome, it was considered appropriate to adopt the critical durations shown in Table 19.

Table 19: Design Rainfall Event – Critical Duration

Model	1% AEP Critical Duration	PMF Critical Duration
North	1 hour	15 minute 1 hour 2 hour
South	2 hour	15 minute 1 hour
East	25 minute	15 minute
West	1 hour	15 minute 30 minute 2 hour

7.3. Downstream Boundary Conditions

In addition to runoff from the catchment, downstream areas can also be influenced by high water levels within Sydney Harbour and Middle Harbour. Consideration must therefore also be given to accounting for the joint probability to coincident flooding from both catchment runoff and backwater effects.

A full joint probability analysis to consider the interaction of these two mechanisms is beyond the scope of the present study. It is accepted practice to estimate design flood levels in these situations using a 'peak envelope' approach that adopts the highest of the predicted levels from the two mechanisms. The constant water level applied to the downstream boundary for each design rainfall event is summarised in Table 20.

Table 20: Design Rainfall Event and Downstream Boundary Conditions

Design Event (AEP)	Rainfall Event	Ocean Level
20% AEP	20% AEP Rainfall	HHWS Ocean Level 1.13 m AHD
10% AEP	10% AEP Rainfall	HHWS Ocean Level 1.13 m AHD
5% AEP	5% AEP Rainfall	HHWS Ocean Level 1.13 m AHD
2% AEP	2% AEP Rainfall	5% AEP Ocean Level 1.40 m AHD
1% AEP (Enveloped)	1% AEP Rainfall	5% AEP Ocean Level 1.40 m AHD
	5% AEP Rainfall	1% AEP Ocean Level 1.45 m AHD
PMF	Probable Maximum Precipitation	1% AEP Ocean Level 1.45 m AHD

7.4. Analysis

7.4.1. Road Access

Current revisions being undertaken on Australian Rainfall and Runoff discuss appropriate safety criteria for vehicles (Engineers Australia, 2011). The criteria proposed, as of February 2011, are presented in Table 21.

Table 21: Draft interim criteria for stationary vehicular stability (Engineers Australia, 2011)

Class of vehicle	Limiting still water depth	Limiting high velocity flow depth (velocity >= 3 m/s)	Limiting Velocity	Equation of stability *
Small passenger	0.3	0.1	3.0	$DV \leq 0.3$
Large passenger	0.4	0.15	3.0	$DV \leq 0.45$
Large 4WD	0.5	0.2	3.0	$DV \leq 0.6$

** DV refers to the multiplication of depth and velocity*

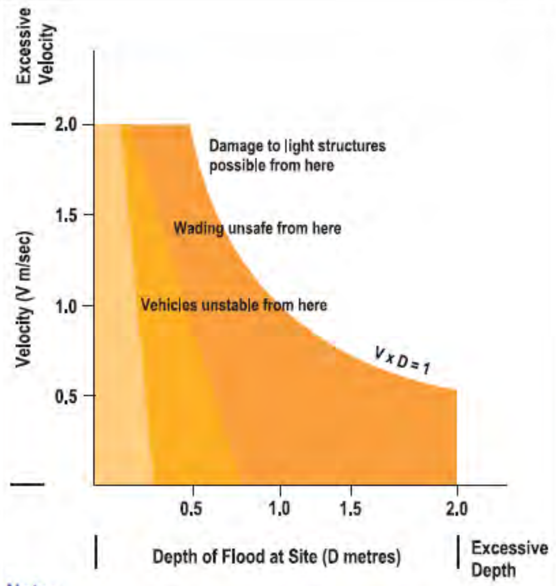
The application of these criteria allows an assessment of the trafficability of key roads within the catchment to be undertaken.

It should be noted that the critical storm duration used for the design events is based upon the storm duration that produces the maximum flood level. This storm duration may not be the same as the storm duration that would produce the longest time of inundation for the road crossings. It is therefore possible for the roads to be cut for longer periods than estimated or possibly for multiple storm peaks to cut the road at separate times.

7.4.2. Provisional Hydraulic Hazard

Hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual, the relevant section of which is shown in Diagram 2. For the purposes of this report, the transition zone presented in Diagram 2 (L2) was considered to be high hazard.

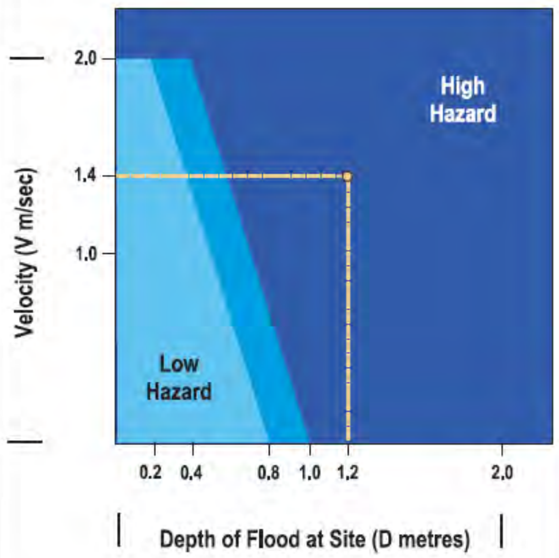
Diagram 2: (L1) Velocity and Depth Relationship; (L2) Provisional Hydraulic Hazard Categories (NSW State Government, 2005)



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**

7.4.3. Provisional Hydraulic Categorisation

The hydraulic categories, namely floodway, flood storage and flood fringe, are described in the Floodplain Development Manual (NSW State Government, 2005). However, there is no technical definition of hydraulic categorisation that would be suitable for all catchments, and different approaches are used by different consultants and authorities, based on the specific features of the study area.

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

- Floodway is defined as areas where:
  - the peak value of velocity multiplied by depth (V x D) > 0.25 m^2/s **AND** peak velocity > 0.25 m/s, **OR**
  - peak velocity > 1.0 m/s **AND** peak depth > 0.15 m

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

- Flood Storage comprises areas outside the floodway where peak depth > 0.5 m; and
- Flood Fringe comprises areas outside the Floodway where peak depth < 0.5 m

7.4.4. Preliminary Flood Emergency Response Classification of Communities

The Floodplain Development Manual, 2005 requires flood studies to address the management of continuing flood risk to both existing and future development areas. As continuing flood risk varies across the floodplain so does the type and scale of emergency response problem and therefore the information necessary for effective Emergency Response Planning (ERP). Classification provides an indication of the vulnerability of the community in flood emergency response and identifies the type and scale of information needed by the SES to assist in emergency response planning (ERP).

Criteria for determining flood ERP classifications and an indication of the emergency response required for these classifications are provided in the Floodplain Risk Management Guideline, 2007 (Flood Emergency Response Planning: Classification of Communities). Table 22 summarises the response required for areas of different classification. However, these may vary depending on local flood characteristics and resultant flood behaviour, i.e. in flash flooding or overland flood areas.

Table 22: Response Required for Different Flood ERP Classifications

Classification	Response Required		
	Resupply	Rescue/Medivac	Evacuation
High Flood Island	Yes	Possibly	Possibly
Low Flood Island	No	Yes	Yes
Area with Rising Road Access	No	Possibly	Yes
Area with Overland Escape Routes	No	Possibly	Yes
Low Trapped Perimeter	No	Yes	Yes
High Trapped Perimeter	Yes	Possibly	Possibly
Indirectly Affected Areas	Possibly	Possibly	Possibly

7.5. Results

The results from this study are presented as:

- Peak flood level profiles in Figure 15;
- Flow and level hydrographs in Figure 16;
- Pipe capacity in Figure 17;
- Peak flood depths and level contours in Figure 18 to Figure 23;
- Provisional hydraulic hazard in Figure 24 to Figure 27;
- Provisional hydraulic categorisation in Figure 28 to Figure 31; and
- Preliminary flood emergency response classification of communities in Figure 32.

7.5.1. Peak Flood Depths and Levels

The peak flood depths and levels within the north model are shown in Table 23. Three roads (Palmer Street, Miller Street and Young Street) were found to have depths less than 0.3 m in the 20% AEP event, however the depths on these roads increased to greater than or equal to 0.5 m in the 1% AEP event.

Table 23: North Model – Peak Flood Depths (m) and Levels (m AHD)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Depth (m)						
Palmer Street	0.2	0.3	0.4	0.5	0.6	0.9
Wheatleigh Street	0.7	0.7	0.8	0.8	0.9	1.6
Atchison Street	0.8	0.9	0.9	1.0	1.1	1.7
Primrose Park	0.5	0.5	0.6	0.6	0.6	1.1
Grafton Street	0.5	0.6	0.6	0.7	0.7	1.1
Warringa Road	0.6	0.6	0.7	0.7	0.7	1.3
Anzac Park	1.9	2.3	2.8	3.4	3.8	7.5
Ernest Street	0.3	0.4	0.4	0.4	0.7	4.5
Miller Street	0.2	0.4	0.5	0.6	0.7	1.8
Young Street	0.1	0.1	0.3	0.5	0.5	1.2
Brightmore Reserve	2.5	2.9	3.0	3.1	3.2	3.9
Grasmere Road	0.3	0.4	0.4	0.4	0.5	1.0
Belgrave Street	0.6	0.6	0.6	0.7	0.7	1.1
Brightmore Street	0.5	0.5	0.6	0.6	0.6	1.2
Level (m AHD)						
Palmer Street	59.0	59.1	59.2	59.3	59.4	59.7
Wheatleigh Street	73.3	73.4	73.5	73.5	73.6	74.2
Atchison Street	78.1	78.2	78.3	78.4	78.4	79.1
Primrose Park	2.8	2.8	2.9	2.9	3.0	3.4
Grafton Street	41.8	41.9	41.9	42.0	42.0	42.5
Warringa Road	54.1	54.2	54.2	54.3	54.3	54.8
Anzac Park	65.6	66.0	66.6	67.1	67.5	71.3
Ernest Street	67.1	67.1	67.2	67.2	67.5	71.3
Miller Street	75.3	75.5	75.6	75.7	75.8	76.9
Young Street	9.0	9.1	9.3	9.4	9.4	10.1
Brightmore Reserve	9.0	9.3	9.5	9.6	9.7	10.4
Grasmere Road	55.7	55.7	55.7	55.8	55.8	56.4
Belgrave Street	68.9	69.0	69.0	69.0	69.1	69.5
Brightmore Street	45.4	45.4	45.5	45.5	45.6	46.1

The peak flood depths and levels within the south model are shown in Table 24. Four roads (Military Road, Aubin Street, High Street and Miller Street) were found to have depths greater than or equal to 0.3 m in the 20% AEP event, however only one of these roads had depths increase to greater than or equal to 0.5 m in the 1% AEP event. The former three roads had little variation in flood depth with varying storm magnitudes.

Table 24: South Model – Peak Flood Depths (m) and Levels (m AHD)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Depth (m)						
Intersection of Clark Rd, Rawson St and Kurraba Rd	0.2	0.3	0.4	0.5	0.5	1.3
Eaton Street	0.2	0.2	0.3	0.3	0.4	0.9
Military Road	0.3	0.3	0.3	0.3	0.3	0.5
Cnr Hayes St and Lower Wycombe Rd	0.0	0.0	0.1	0.1	0.2	0.9
Aubin St	0.3	0.3	0.3	0.4	0.4	0.8
Phillips St	0.1	0.1	0.2	0.2	0.2	0.7
Hipwood St	0.1	0.1	0.2	0.2	0.2	0.3
High St	0.4	0.4	0.4	0.4	0.4	0.6
Miller St	0.3	0.4	0.4	0.5	0.5	1.0
Level (m AHD)						
Intersection of Clark Rd, Rawson St and Kurraba Rd	3.0	3.1	3.2	3.3	3.3	4.1
Eaton Street	13.4	13.4	13.5	13.5	13.6	14.1
Military Road	88.4	88.4	88.4	88.4	88.4	88.6
Cnr Hayes St and Lower Wycombe Rd	11.7	11.8	11.8	11.8	11.8	12.5
Aubin St	31.3	31.3	31.4	31.4	31.5	31.9
Phillips St	41.3	41.3	41.3	41.3	41.3	41.9
Hipwood St	4.5	4.5	4.5	4.5	4.6	4.7
High St	15.3	15.3	15.3	15.3	15.3	15.5
Miller St	63.3	63.3	63.4	63.4	63.5	64.0

The peak flood depths and levels within the east model are shown in Table 25. Four roads were found to have depths greater than or equal to 0.3 m in the 20% AEP event that increased to greater than or equal to 0.5 m in the 1% AEP event. These were Bogota Avenue, Honda Avenue, Bannerman Street and the corner of Spofforth Street and Reginald Street. The remaining four roads analysed had shallow depths of 0.2 m across all the events up to and including the 1% AEP event.

Table 25: East Model – Peak Flood Depths (m) and Levels (m AHD)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Depth (m)						
Bogota Ave	0.4	0.4	0.5	0.5	0.6	1.4
Honda Ave	0.4	0.4	0.5	0.6	0.7	1.8
Bannerman St	0.5	0.5	0.5	0.5	0.6	0.9
Cnr Burroway St and Shellcove Rd	0.2	0.2	0.2	0.2	0.2	0.3
Bertha St	0.1	0.2	0.2	0.2	0.2	0.4
Bennett St	0.2	0.2	0.2	0.2	0.2	0.5
Harrison St	0.2	0.2	0.2	0.2	0.2	0.3
Cnr Spofforth St and Reginald St	0.3	0.4	0.4	0.4	0.4	0.9
Level (m AHD)						
Bogota Ave	17.9	18.0	18.0	18.1	18.1	18.9
Honda Ave	21.8	21.8	21.9	22.0	22.1	23.2
Bannerman St	34.7	34.8	34.8	34.8	34.8	35.2
Cnr Burroway St and Shellcove Rd	54.4	54.4	54.5	54.5	54.5	54.6
Bertha St	58.8	58.8	58.8	58.8	58.8	59.0
Bennett St	68.2	68.2	68.2	68.2	68.2	68.5
Harrison St	76.5	76.6	76.6	76.6	76.6	76.7
Cnr Spofforth St and Reginald St	57.5	57.6	57.6	57.6	57.6	58.1

The peak flood depths and levels within the west model are shown in Table 26. All the locations analysed were found to have depths greater than or equal to 0.3 m in the 20% AEP event that increased to greater than or equal to 0.5 m in the 1% AEP event.

Table 26: West Model – Peak Flood Depths (m) and Levels (m AHD)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Depth (m)						
Woolcott St	1.2	1.4	1.6	1.7	1.8	3.0
Euroka St	0.4	0.5	0.6	0.7	0.8	2.3
Bank St	0.4	0.4	0.5	0.5	0.5	1.2
Cnr Bay Rd and Crows Nest Rd	0.3	0.4	0.4	0.5	0.5	1.2
Cnr Newlands La and Meadow La	0.5	0.8	1.2	1.7	2.1	5.6
Cnr Carlyle La and Miller La	0.8	1.1	1.5	1.9	2.3	7.7
Russell St	0.5	0.6	0.6	0.6	0.6	1.0
Lithgow St	0.6	0.7	0.7	0.7	0.8	1.1
Level (m AHD)						
Woolcott St	27.0	27.2	27.4	27.5	27.6	28.8
Euroka St	30.1	30.2	30.3	30.4	30.5	32.0
Bank St	45.4	45.4	45.4	45.5	45.5	46.2
Cnr Bay Rd and Crows Nest Rd	41.1	41.1	41.2	41.2	41.2	41.9
Cnr Newlands La and Meadow La	40.7	41.0	41.4	41.9	42.3	45.8
Cnr Carlyle La and Miller La	53.3	53.6	54.0	54.4	54.8	60.3
Russell St	41.9	41.9	41.9	42.0	42.0	42.3
Lithgow St	65.6	65.6	65.7	65.7	65.7	66.1

7.5.2. Peak Flow

The peak flows within the north model are shown in Table 27. The greatest flow observed across the events was located at Grafton Street.

Table 27: North Model – Peak Flows (m^3/s)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Flow (m^3/s)						
Brooke Street	3.2	5.2	7.7	10.2	12.7	71.9
Wheatleigh Street	0.3	0.3	0.3	0.3	0.3	0.2
Atchison Street	2.3	3.2	4.1	5.1	5.9	30.1
Grafton Street	12.7	16.3	20.7	24.8	28.3	109.6
Ernest Street	4.8	7.1	9.3	11.8	14.4	56.8
Anzac Avenue	1.1	1.6	2.3	2.8	3.4	18.1
Young Street	1.2	1.0	6.5	13.1	18.4	142.5
Grasmere Road	5.1	6.9	9.1	11.3	13.4	72.0
Brightmore Street	0.0	0.0	0.0	0.0	9.5	49.9

The peak flows within the south model are shown in Table 28. The greatest flow observed across the events greater than and including the 10% AEP event was located at the intersection of Clark Road, Rawson Street and Kurraba Road. In the 20% AEP event, there is little difference between the flows experienced at the former location and Eaton Street, which is located upstream. This is due to the mainstream flow being contained within the open channel with predominantly local overland flow affecting these roads in the 20% AEP event.

Table 28: South Model – Peak Flows (m^3/s)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Flow (m^3/s)						
Intersection of Clark Rd, Rawson St and Kurraba Rd	1.7	4.1	9.3	13.6	17.9	112.7
Eaton St	2.2	3.6	5.4	7.0	8.7	51.5
Cnr Hayes St and Lower Wycombe Rd	3.6	5.0	6.0	7.9	9.6	43.5
Aubin St	1.6	2.2	3.5	4.7	5.8	35.8
Phillips St	1.2	1.8	2.7	3.2	3.8	23.7
Hipwood Street	1.1	1.4	1.9	2.2	3.0	16.7
Mount St	1.0	1.4	2.0	2.8	3.6	27.2

The peak flows within the east model are shown in Table 29. The greatest flow observed across the events was located at Bannerman Street, which is located downstream of Bennett Street and downstream of Harrison Street.

Table 29: East Model – Peak Flows (m^3/s)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Flow (m^3/s)						
Bannerman St	3.2	4.1	5.3	6.1	7.4	42.2
Bennett St	1.5	1.7	2.2	2.7	3.2	16.9
Harrison St	0.9	1.2	1.6	1.9	2.2	9.9
Spofforth St - Between Brierley St and Florence St	2.6	3.3	4.1	4.6	5.3	26.7
Lower Spofforth Walk - Between Boyle St and Hodgson Ave	2.1	2.4	2.9	3.4	3.9	22.7

The peak flows within the west model are shown in Table 30. The greatest flow observed across the events was located at Russell Street.

Table 30: West Model – Peak Flows (m^3/s)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Flow (m^3/s)						
Woolcott St	0.4	1.5	3.3	4.7	6.2	38.4
Ancrum St	2.5	3.4	4.5	5.4	6.4	31.5
Carr St	2.0	2.5	3.3	4.1	4.8	24.0
Brennan Park	2.6	3.3	4.2	5.1	5.8	27.4
Newlands St	2.1	2.6	3.4	4.3	5.1	25.2
Russell St	13.7	16.5	19.8	23.2	26.1	117.2
Lithgow St	1.7	2.2	2.9	3.5	4.2	18.1

7.5.3. Road Access

A number of roads in the north model are subject to overland flow in the 1% AEP event that exceeded the safety criteria for vehicles (as defined in Section 7.4.1). This is shown in Table 31.

In all events greater than and including the 20% AEP event, Brightmore Street, Cammeray Road / Amherst Street, Ernest Street and Young Street are inaccessible to varying depths and durations. Grafton Street, Park Avenue and Waters Road are inaccessible in events larger than and including the 10% AEP event; Grasmere Road and Miller Street are inaccessible in events larger than and including the 5% AEP event; Cooper Lane is inaccessible in events larger than and including the 2% AEP event; and Falcon Street and West Street are inaccessible in events larger than and including the 1% AEP event.

Table 31: North Model – Road Trafficability (Duration above depth)

Location	Duration (hr) Depth > 0.3m	Duration (hr) Depth > 0.4m	Duration (hr) Depth > 0.5m
20% AEP event			
Belgrave St - Between Young St and Cooper La	0	0	0
Brightmore St	0.73	0.35	0
Cammeray Rd / Amherst St - Between Bellevue St and Grafton St	0.35	0	0
Cooper La - Between Grosvenor La and Belgrave St	0	0	0
Ernest St - Between Warringah Fwy and Miller St	0.68	0.51	0.26
Falcon St - Between Rodborough Ave and Lytton St	0	0	0
Grafton St - Between Cammeray Rd and Fall St	0	0	0
Grasmere Rd - Between Young St and Benelong Rd	0	0	0
Miller St - Between Ernest St and Falcon St	0	0	0
Park Ave - Between Cammeray Rd and Sutherland St	0	0	0
Reynolds St - Between Benelong Rd and Levick St	0	0	0
Waters Rd - Between Belgrave St and Winnie St	0	0	0
West St - Between Hayberry St and Holtermann St	0	0	0
Young St - Between Little Young St and Wonga Rd	2.17	1.32	0.96
10% AEP event			
Belgrave St - Between Young St and Cooper La	0	0	0
Brightmore St	0.79	0.41	0.12
Cammeray Rd / Amherst St - Between Bellevue St and Grafton St	0.47	0.17	0
Cooper La - Between Grosvenor La and Belgrave St	0	0	0
Ernest St - Between Warringah Fwy and Miller St	0.8	0.62	0.41
Falcon St - Between Rodborough Ave and Lytton St	0	0	0
Grafton St - Between Cammeray Rd and Fall St	0.11	0	0
Grasmere Rd - Between Young St and Benelong Rd	0	0	0
Miller St - Between Ernest St and Falcon St	0	0	0
Park Ave - Between Cammeray Rd and Sutherland St	0.14	0	0
Reynolds St - Between Benelong Rd and Levick St	0	0	0
Waters Rd - Between Belgrave St and Winnie St	0.07	0	0
West St - Between Hayberry St and Holtermann St	0	0	0
Young St - Between Little Young St and Wonga Rd	2.2	1.35	0.98
5% AEP event			
Belgrave St - Between Young St and Cooper La	0	0	0
Brightmore St	0.84	0.47	0.2
Cammeray Rd / Amherst St - Between Bellevue St and Grafton St	0.57	0.3	0.07
Cooper La - Between Grosvenor La and Belgrave St	0	0	0
Ernest St - Between Warringah Fwy and Miller St	0.88	0.72	0.54
Falcon St - Between Rodborough Ave and Lytton St	0	0	0
Grafton St - Between Cammeray Rd and Fall St	0.23	0	0
Grasmere Rd - Between Young St and Benelong Rd	0.14	0	0
Miller St - Between Ernest St and Falcon St	0.23	0	0
Park Ave - Between Cammeray Rd and Sutherland St	0.27	0.08	0
Reynolds St - Between Benelong Rd and Levick St	0	0	0
Waters Rd - Between Belgrave St and Winnie St	0.19	0	0
West St - Between Hayberry St and Holtermann St	0	0	0
Young St - Between Little Young St and Wonga Rd	2.23	1.38	1.01
2% AEP event			

Location	Duration (hr) Depth > 0.3m	Duration (hr) Depth > 0.4m	Duration (hr) Depth > 0.5m
Belgrave St - Between Young St and Cooper La	0	0	0
Brightmore St	0.91	0.54	0.29
Cammeray Rd / Amherst St - Between Bellevue St and Grafton St	0.71	0.43	0.2
Cooper La - Between Grosvenor La and Belgrave St	0.1	0	0
Ernest St - Between Warringah Fwy and Miller St	0.99	0.84	0.65
Falcon St - Between Rodborough Ave and Lytton St	0	0	0
Grafton St - Between Cammeray Rd and Fall St	0.36	0.08	0
Grasmere Rd - Between Young St and Benelong Rd	0.23	0	0
Miller St - Between Ernest St and Falcon St	0.38	0.26	0
Park Ave - Between Cammeray Rd and Sutherland St	0.39	0.2	0
Reynolds St - Between Benelong Rd and Levick St	0	0	0
Waters Rd - Between Belgrave St and Winnie St	0.34	0	0
West St - Between Hayberry St and Holtermann St	0	0	0
Young St - Between Little Young St and Wonga Rd	2.28	1.44	1.06
1% AEP event			
Belgrave St - Between Young St and Cooper La	0	0	0
Brightmore St	0.93	0.59	0.36
Cammeray Rd / Amherst St - Between Bellevue St and Grafton St	0.8	0.52	0.29
Cooper La - Between Grosvenor La and Belgrave St	0.16	0	0
Ernest St - Between Warringah Fwy and Miller St	1.08	0.99	0.85
Falcon St - Between Rodborough Ave and Lytton St	0.1	0	0
Grafton St - Between Cammeray Rd and Fall St	0.46	0.17	0
Grasmere Rd - Between Young St and Benelong Rd	0.29	0	0
Miller St - Between Ernest St and Falcon St	0.46	0.35	0.22
Park Ave - Between Cammeray Rd and Sutherland St	0.48	0.28	0.11
Reynolds St - Between Benelong Rd and Levick St	0	0	0
Waters Rd - Between Belgrave St and Winnie St	0.41	0	0
West St - Between Hayberry St and Holtermann St	0.14	0	0
Young St - Between Little Young St and Wonga Rd	2.3	1.45	1.08
PMF event			
Belgrave St - Between Young St and Cooper La	1.39	0.74	0.14
Brightmore St	2.08	1.97	1.81
Cammeray Rd / Amherst St - Between Bellevue St and Grafton St	3.33	2.88	2.21
Cooper La - Between Grosvenor La and Belgrave St	1.63	1.03	0.42
Ernest St - Between Warringah Fwy and Miller St	3.72	3.65	3.55
Falcon St - Between Rodborough Ave and Lytton St	1.61	1.44	1.25
Grafton St - Between Cammeray Rd and Fall St	2.39	2.02	1.83
Grasmere Rd - Between Young St and Benelong Rd	1.76	1.51	1.32
Miller St - Between Ernest St and Falcon St	1.83	1.73	1.61
Park Ave - Between Cammeray Rd and Sutherland St	2.69	2.16	2.01
Reynolds St - Between Benelong Rd and Levick St	0.49	0.1	0
Waters Rd - Between Belgrave St and Winnie St	1.84	0.52	0
West St - Between Hayberry St and Holtermann St	1.58	0.97	0.18
Young St - Between Little Young St and Wonga Rd	3.39	2.6	2.25

A number of roads in the south model are subject to overland flow in the 1% AEP event that exceeded the safety criteria for vehicles (as defined in Section 7.4.1). This is shown in Table 32.

In all events greater than and including the 20% AEP event, Mount Street, Clark Road, Rawson Street and Falcon Street are inaccessible to varying depths and durations. Kurraba Road is inaccessible in events larger than and including the 10% AEP event; Angelo Street is inaccessible in events larger than and including the 5% AEP event; Eaton Street is inaccessible in events larger than and including the 2% AEP event; and Miller Street is inaccessible in events larger than and including the 1% AEP event.

Table 32: South Model – Road Trafficability (Duration above depth)

Location	Duration (hr) Depth > 0.3m	Duration (hr) Depth > 0.4m	Duration (hr) Depth > 0.5m
20% AEP event			
Miller St - Between Pacific Hwy and McLaren St	0	0	0
Pacific Hwy - Between McLaren St and High St	0	0	0
Walker St - Between Pacific Hwy and Berry St	0	0	0
Mount St - Between Pacific Hwy and Arthur St	0.83	0	0
Angelo St - Between Berry St and McLaren St	0	0	0
High St - Between Little Alfred St and Hipwood St	0	0	0
Hipwood St - Between McDougall St and High St	0	0	0
Clark Rd - Between McDougall St and High St	0	0	0
Clark Rd - Between Margaret St and Kurraba Rd	0.48	0.39	0.1
Rawson St - Between Kurraba Rd and Darley St	0.21	0.1	0
Kurraba Rd - Between Neutral St and Holdsworth St	0	0	0
Eaton St - Between Nook La and Montpeller St	0	0	0
Falcon St - Between Military Rd and Merlin St	0.22	0	0
Military Rd - Between Falcon St and Park Av	0	0	0
10% AEP event			
Miller St - Between Pacific Hwy and McLaren St	0	0	0
Pacific Hwy - Between McLaren St and High St	0	0	0
Walker St - Between Pacific Hwy and Berry St	0	0	0
Mount St - Between Pacific Hwy and Arthur St	0.97	0	0
Angelo St - Between Berry St and McLaren St	0	0	0
High St - Between Little Alfred St and Hipwood St	0	0	0
Hipwood St - Between McDougall St and High St	0	0	0
Clark Rd - Between McDougall St and High St	0	0	0
Clark Rd - Between Margaret St and Kurraba Rd	0.58	0.48	0.26
Rawson St - Between Kurraba Rd and Darley St	0.32	0.25	0.13
Kurraba Rd - Between Neutral St and Holdsworth St	0.12	0	0
Eaton St - Between Nook La and Montpeller St	0	0	0
Falcon St - Between Military Rd and Merlin St	0.26	0	0
Military Rd - Between Falcon St and Park Av	0	0	0
5% AEP event			
Miller St - Between Pacific Hwy and McLaren St	0	0	0
Pacific Hwy - Between McLaren St and High St	0	0	0
Walker St - Between Pacific Hwy and Berry St	0	0	0
Mount St - Between Pacific Hwy and Arthur St	1.14	0	0
Angelo St - Between Berry St and McLaren St	0.03	0	0
High St - Between Little Alfred St and Hipwood St	0	0	0
Hipwood St - Between McDougall St and High St	0	0	0
Clark Rd - Between McDougall St and High St	0	0	0
Clark Rd - Between Margaret St and Kurraba Rd	0.71	0.55	0.41
Rawson St - Between Kurraba Rd and Darley St	0.42	0.34	0.25

Location	Duration (hr) Depth > 0.3m	Duration (hr) Depth > 0.4m	Duration (hr) Depth > 0.5m
Kurraba Rd - Between Neutral St and Holdsworth St	0.24	0.1	0
Eaton St - Between Nook La and Montpeller St	0	0	0
Falcon St - Between Military Rd and Merlin St	0.32	0	0
Military Rd - Between Falcon St and Park Av	0	0	0
2% AEP event			
Miller St - Between Pacific Hwy and McLaren St	0	0	0
Pacific Hwy - Between McLaren St and High St	0	0	0
Walker St - Between Pacific Hwy and Berry St	0	0	0
Mount St - Between Pacific Hwy and Arthur St	1.34	0.09	0
Angelo St - Between Berry St and McLaren St	0.09	0	0
High St - Between Little Alfred St and Hipwood St	0	0	0
Hipwood St - Between McDougall St and High St	0	0	0
Clark Rd - Between McDougall St and High St	0	0	0
Clark Rd - Between Margaret St and Kurraba Rd	0.84	0.71	0.49
Rawson St - Between Kurraba Rd and Darley St	0.52	0.42	0.32
Kurraba Rd - Between Neutral St and Holdsworth St	0.31	0.18	0.03
Eaton St - Between Nook La and Montpeller St	0.1	0	0
Falcon St - Between Military Rd and Merlin St	0.4	0	0
Military Rd - Between Falcon St and Park Av	0	0	0
1% AEP event			
Miller St - Between Pacific Hwy and McLaren St	0.05	0	0
Pacific Hwy - Between McLaren St and High St	0	0	0
Walker St - Between Pacific Hwy and Berry St	0	0	0
Mount St - Between Pacific Hwy and Arthur St	1.19	0.12	0
Angelo St - Between Berry St and McLaren St	0.13	0	0
High St - Between Little Alfred St and Hipwood St	0	0	0
Hipwood St - Between McDougall St and High St	0	0	0
Clark Rd - Between McDougall St and High St	0	0	0
Clark Rd - Between Margaret St and Kurraba Rd	0.97	0.8	0.56
Rawson St - Between Kurraba Rd and Darley St	0.62	0.5	0.39
Kurraba Rd - Between Neutral St and Holdsworth St	0.37	0.25	0.14
Eaton St - Between Nook La and Montpeller St	0.16	0	0
Falcon St - Between Military Rd and Merlin St	0.49	0	0
Military Rd - Between Falcon St and Park Av	0	0	0
PMF event			
Miller St - Between Pacific Hwy and McLaren St	0.82	0.69	0.55
Pacific Hwy - Between McLaren St and High St	0.76	0.6	0.12
Walker St - Between Pacific Hwy and Berry St	0.68	0.54	0.25
Mount St - Between Pacific Hwy and Arthur St	1.04	0.84	0.26
Angelo St - Between Berry St and McLaren St	0.84	0.68	0.36
High St - Between Little Alfred St and Hipwood St	0	0	0
Hipwood St - Between McDougall St and High St	1.02	0.88	0.54
Clark Rd - Between McDougall St and High St	1.02	0.89	0.69
Clark Rd - Between Margaret St and Kurraba Rd	1.3	1.25	1.19
Rawson St - Between Kurraba Rd and Darley St	1.2	1.11	1.04
Kurraba Rd - Between Neutral St and Holdsworth St	1.03	0.95	0.88
Eaton St - Between Nook La and Montpeller St	0.88	0.73	0.6
Falcon St - Between Military Rd and Merlin St	0.95	0.64	0.06
Military Rd - Between Falcon St and Park Av	0	0	0

A number of roads in the east model are subject to overland flow in the 1% AEP event that exceeded the safety criteria for vehicles (as defined in Section 7.4.1). This is shown in Table 33.

In all events greater than and including the 20% AEP event, Bogota Avenue, Honda Road and Spofforth Street are inaccessible to varying depths and durations. Bannerman Street is inaccessible in events larger than and including the 2% AEP event. Bennett Street, Bertha Road and Rangers Road are inaccessible in the PMF event.

Table 33: East Model – Road Trafficability (Duration above depth)

Location	Duration (hr) Depth > 0.3m	Duration (hr) Depth > 0.4m	Duration (hr) Depth > 0.5m
20% AEP event			
Bannerman St - Between Murdoch St and Shellcove Rd	0	0	0
Bennett St - Between Murdoch St and Wycombe Rd	0	0	0
Bertha Rd - Between Murdoch St and Burroway St	0	0	0
Bogota Av - Between Murdoch St and Honda Rd	0.28	0	0
Harrison St - Between Rangers Rd and Wycombe Rd	0	0	0
Honda Rd - Between Bogota Av and Shellcove Rd	0.09	0	0
Yeo St - Between Rangers Rd and Barry La	0	0	0
Rangers Rd - Between Spofforth St and Murdoch St	0	0	0
Spofforth St - Between Military Rd and Florence St	0	0	0
Spofforth St - Between Boyle St and Kareela La	0.44	0.13	0
10% AEP event			
Bannerman St - Between Murdoch St and Shellcove Rd	0	0	0
Bennett St - Between Murdoch St and Wycombe Rd	0	0	0
Bertha Rd - Between Murdoch St and Burroway St	0	0	0
Bogota Av - Between Murdoch St and Honda Rd	0.3	0.09	0
Harrison St - Between Rangers Rd and Wycombe Rd	0	0	0
Honda Rd - Between Bogota Av and Shellcove Rd	0.13	0.07	0
Yeo St - Between Rangers Rd and Barry La	0	0	0
Rangers Rd - Between Spofforth St and Murdoch St	0	0	0
Spofforth St - Between Military Rd and Florence St	0	0	0
Spofforth St - Between Boyle St and Kareela La	0.47	0.14	0
5% AEP event			
Bannerman St - Between Murdoch St and Shellcove Rd	0	0	0
Bennett St - Between Murdoch St and Wycombe Rd	0	0	0
Bertha Rd - Between Murdoch St and Burroway St	0	0	0
Bogota Av - Between Murdoch St and Honda Rd	0.33	0.15	0
Harrison St - Between Rangers Rd and Wycombe Rd	0	0	0
Honda Rd - Between Bogota Av and Shellcove Rd	0.18	0.13	0.07
Yeo St - Between Rangers Rd and Barry La	0	0	0
Rangers Rd - Between Spofforth St and Murdoch St	0	0	0
Spofforth St - Between Military Rd and Florence St	0	0	0
Spofforth St - Between Boyle St and Kareela La	0.48	0.17	0
2% AEP event			
Bannerman St - Between Murdoch St and Shellcove Rd	0.04	0	0
Bennett St - Between Murdoch St and Wycombe Rd	0	0	0
Bertha Rd - Between Murdoch St and Burroway St	0	0	0
Bogota Av - Between Murdoch St and Honda Rd	0.38	0.2	0.1
Harrison St - Between Rangers Rd and Wycombe Rd	0	0	0

Location	Duration (hr) Depth > 0.3m	Duration (hr) Depth > 0.4m	Duration (hr) Depth > 0.5m
Honda Rd - Between Bogota Av and Shellcove Rd	0.24	0.18	0.12
Yeo St - Between Rangers Rd and Barry La	0	0	0
Rangers Rd - Between Spofforth St and Murdoch St	0	0	0
Spofforth St - Between Military Rd and Florence St	0.06	0	0
Spofforth St - Between Boyle St and Kareela La	0.52	0.2	0
1% AEP event			
Bannerman St - Between Murdoch St and Shellcove Rd	0.11	0	0
Bennett St - Between Murdoch St and Wycombe Rd	0	0	0
Bertha Rd - Between Murdoch St and Burroway St	0	0	0
Bogota Av - Between Murdoch St and Honda Rd	0.4	0.24	0.14
Harrison St - Between Rangers Rd and Wycombe Rd	0	0	0
Honda Rd - Between Bogota Av and Shellcove Rd	0.27	0.21	0.16
Yeo St - Between Rangers Rd and Barry La	0	0	0
Rangers Rd - Between Spofforth St and Murdoch St	0	0	0
Spofforth St - Between Military Rd and Florence St	0.11	0	0
Spofforth St - Between Boyle St and Kareela La	0.52	0.23	0.08
PMF event			
Bannerman St - Between Murdoch St and Shellcove Rd	0.3	0.23	0.17
Bennett St - Between Murdoch St and Wycombe Rd	0.22	0.14	0
Bertha Rd - Between Murdoch St and Burroway St	0.25	0.06	0
Bogota Av - Between Murdoch St and Honda Rd	0.42	0.36	0.32
Harrison St - Between Rangers Rd and Wycombe Rd	0.02	0	0
Honda Rd - Between Bogota Av and Shellcove Rd	0.35	0.33	0.31
Yeo St - Between Rangers Rd and Barry La	0	0	0
Rangers Rd - Between Spofforth St and Murdoch St	0.19	0.11	0
Spofforth St - Between Military Rd and Florence St	0.29	0.23	0.19
Spofforth St - Between Boyle St and Kareela La	0.47	0.32	0.28

A number of roads in the west model are subject to overland flow that exceeded the safety criteria for vehicles (as defined in Section 7.4.1) in events greater than and including the 20% AEP event. This is shown in Table 34.

Table 34: West Model – Road Trafficability (Duration above depth)

Location	Duration (hr) Depth > 0.3m	Duration (hr) Depth > 0.4m	Duration (hr) Depth > 0.5m
20% AEP event			
Euroka St / Union St - Between Bank St and Euroka La	0.17	0.08	0
Hazelbank Rd - Between Pacific Hwy and Ivy St	0.58	0.27	0
Lithgow St - Between River Rd and Oxley St	1.17	0.75	0.42
Meadow La - Between Shirley Rd and Rocklands Rd	1.08	0.08	0.08
River Rd - Between Eastview St and Russell St	0.76	0.52	0.16
Woolcott St - Between Euroka St and Larkin St	0.42	0.38	0.34
10% AEP event			
Euroka St / Union St - Between Bank St and Euroka La	0.25	0.16	0.08
Hazelbank Rd - Between Pacific Hwy and Ivy St	0.64	0.36	0
Lithgow St - Between River Rd and Oxley St	1.2	0.8	0.49
Meadow La - Between Shirley Rd and Rocklands Rd	1.09	0.29	0.27
River Rd - Between Eastview St and Russell St	0.81	0.6	0.32
Woolcott St - Between Euroka St and Larkin St	0.5	0.46	0.42
5% AEP event			
Euroka St / Union St - Between Bank St and Euroka La	0.33	0.23	0.16
Hazelbank Rd - Between Pacific Hwy and Ivy St	0.7	0.43	0.1
Lithgow St - Between River Rd and Oxley St	1.23	0.85	0.56
Meadow La - Between Shirley Rd and Rocklands Rd	1.1	0.43	0.41
River Rd - Between Eastview St and Russell St	0.85	0.67	0.38
Woolcott St - Between Euroka St and Larkin St	0.59	0.54	0.5
2% AEP event			
Euroka St / Union St - Between Bank St and Euroka La	0.42	0.31	0.23
Hazelbank Rd - Between Pacific Hwy and Ivy St	0.8	0.51	0.16
Lithgow St - Between River Rd and Oxley St	1.28	0.92	0.66
Meadow La - Between Shirley Rd and Rocklands Rd	1.21	0.62	0.6
River Rd - Between Eastview St and Russell St	0.92	0.78	0.47
Woolcott St - Between Euroka St and Larkin St	0.69	0.65	0.61
1% AEP event			
Euroka St / Union St - Between Bank St and Euroka La	0.48	0.37	0.29
Hazelbank Rd - Between Pacific Hwy and Ivy St	0.85	0.56	0.26
Lithgow St - Between River Rd and Oxley St	1.3	0.95	0.72
Meadow La - Between Shirley Rd and Rocklands Rd	1.26	0.73	0.71
River Rd - Between Eastview St and Russell St	0.95	0.81	0.54
Woolcott St - Between Euroka St and Larkin St	0.75	0.71	0.67
PMF event			
Euroka St / Union St - Between Bank St and Euroka La	1.9	1.83	1.74
Hazelbank Rd - Between Pacific Hwy and Ivy St	2.08	1.95	1.67
Lithgow St - Between River Rd and Oxley St	2.48	2.15	1.99
Meadow La - Between Shirley Rd and Rocklands Rd	2.72	2.63	2.62
River Rd - Between Eastview St and Russell St	2.13	2.05	1.95
Woolcott St - Between Euroka St and Larkin St	2.09	2.06	2.04

7.5.4. Provisional Hydraulic Hazard

During the 1% AEP event, a number of high hazard flow paths were identified across the study area.

In the north model, the high hazard areas predominantly occurred through private property, and through parks and reserves. A small section of Cammeray Road and Lytton Street – Lillis Street contained the high hazard flow in the kerb and gutter system; whereas most flow was found to occur either perpendicular to the roadway or between parallel roadways (thereby affecting private property).

In the south model, the high hazard areas mostly coincided with the open channel sections. Additionally, a small flow path of high hazard occurred in the kerb and gutter system along Hayes Street and the private property upstream from the corner of Hayes Street and Manns Avenue towards Phillips Street.

In the east model, the high hazard areas occurred through residential private property and perpendicular to the roadway, with the exception of Bogota Aveune. Along Bogota Avenue, the high hazard flow path occurred along the roadway, within the kerb and gutter system.

In the west model, high hazard flow occurred along the kerb and gutter system of Woolcott Street, Euroka Street, Gasworks Road and Newlands Lane. Additional high hazard flow was found to occur through parks and reserves. The number of properties with high hazard flow affectation was limited.

7.5.5. Provisional Hydraulic Categorisation

During the 1% AEP event, a number of floodway and flood storage areas were identified across the study area.

In the north model, the floodway areas rarely coincided with the kerb and gutter system, predominantly occurring through private properties and through parks, reserves and golf courses. There were some flood storage areas on private property where buildings impeded flow paths. Other flood storage areas were located in parks upstream of obstructions such as Anzac Park and Brightmore Reserve, which are located upstream of Warringah Freeway and Young Street respectively.

In the south model, the floodway occurred along the open channels and through properties located between Ben Boyd Road and Undercliff Street. There were limited areas of flood storage, with small areas on the Warringah Freeway, Hampden Street upstream of Warringah Freeway and the open channel upstream of Kurraba Road.

In the east model, the floodway areas coincided with the principal flow paths and rarely coincided with the kerb and gutter system of the roadways (with the exception of Bogota Avenue); predominately occurring perpendicular to the roadway through residential private property. However, the width of the floodway was narrow, generally less than two or three properties wide. There were limited areas of flood storage and these areas were typically highly localised within individual properties.

In the west model, the floodway areas coincided with the principal flow paths. These flow paths corresponded with the kerb and gutter system along Hazelbank Road and Newlands Lane. In the vicinity of Euroka Street and Bank Street the floodway area traverses private property as well as the roadway. The flood storage areas were localised and typically occurred upstream of obstructions to the flow path, such as upstream of railway embankments.

7.5.6. Preliminary Flood Emergency Response Classification of Communities

The criteria for classification of floodplain communities are generally more applicable to riverine flooding where significant flood warning time is available and emergency response action can be taken prior to the flood. In urban areas like North Sydney, flash flooding from local catchment and overland flow will generally occur as a direct response to intense rainfall without significant warning. For most (if not all) flood affected properties in the catchment, remaining inside the building is likely to present less risk to life than attempting to drive or wade through floodwaters, as flow velocities and depths are likely to be greater in the roadway.

There were a number of areas in the study area that were classified as either Rising Road Access or Low Flood Island, predominantly occurring along major flow paths. Smaller areas of High Flood Island and Low Trapped Perimeter Area were also located within the study area.

8. SENSITIVITY ANALYSIS

8.1. Overview

The following sensitivity analyses were undertaken to establish the variation in design flood levels and flow that may occur if different parameter assumptions were made:

- Routing Lag: The hydrologic routing length values were increased and decreased by 20% for all sub-catchments;
- Manning's "n": The hydraulic roughness values were increased and decreased by 20%;
- Blockage: Sensitivity to blockage of all pipes was assessed for 20% and 50% blockage;
- Climate Change (Rainfall Increase): Sensitivity to rainfall/runoff estimates were assessed by increasing the rainfall intensities by 10%, 20% and 30% as recommended under current guidelines;
- Climate Change (Sea Level Rise): Sea level rise scenarios of 0.4 m and 0.9 m were assessed.

These sensitivity scenarios were undertaken for the 1% AEP rainfall event with the 5% AEP ocean level, with the exception of the sea level rise scenarios that were undertaken on the enveloped 1% AEP event.

8.2. Climate Change Background

Intensive scientific investigation is ongoing to estimate the effects that increasing amounts of greenhouse gases (water vapour, carbon dioxide, methane, nitrous oxide, ozone) are having on the average earth surface temperature. Changes to surface and atmospheric temperatures may affect climate and sea levels. The extent of any permanent climatic or sea level change can only be established with certainty through scientific observations over several decades. Nevertheless, it is prudent to consider the possible range of impacts with regard to flooding and the level of flood protection provided by any mitigation works.

Based on the latest research by the United Nations Intergovernmental Panel on Climate Change, evidence is emerging on the likelihood of climate change and sea level rise as a result of increasing greenhouse gasses. In this regard, the following points can be made:

- greenhouse gas concentrations continue to increase;
- global sea level has risen about 0.1 m to 0.25 m in the past century;
- many uncertainties limit the accuracy to which future climate change and sea level rises can be projected and predicted

8.2.1. Rainfall Increase

The Bureau of Meteorology has indicated that there is no intention at present to revise design rainfalls to take account of the potential climate change, as the implications of temperature changes on extreme rainfall intensities are presently unclear, and there is no certainty that the changes would in fact increase design rainfalls for major flood producing storms. There is some recent literature by CSIRO that suggests extreme rainfalls may increase by up to 30% in parts of NSW (in other places the projected increases are much less or even decrease); however this information is not of sufficient accuracy for use as yet (NSW State Government, 2007).

Any increase in design flood rainfall intensities will increase the frequency, depth and extent of inundation across the catchment. It has also been suggested that the cyclone belt may move further southwards. The possible impacts of this on design rainfalls cannot be ascertained at this time as little is known about the mechanisms that determine the movement of cyclones under existing conditions.

Projected increases to evaporation are also an important consideration because increased evaporation would lead to generally dryer catchment conditions, resulting in lower runoff from rainfall. Mean annual rainfall is projected to decrease, which will also result in generally dryer catchment conditions. The influence of dry catchment conditions on river runoff is observable in climate variability using the Indian Pacific Oscillation (IPO) index (Westra et al, 2009). Although mean daily rainfall intensity is not observed to differ significantly between IPO phases, runoff is significantly reduced during periods with fewer rain days.

The combination of uncertainty about projected changes in rainfall and evaporation makes it extremely difficult to predict with confidence the likely changes to peak flows for large flood events within the Dobroyd Canal catchment under warmer climate scenarios.

In light of this uncertainty, the NSW State Government (2007) advice recommends sensitivity analysis on flood modelling should be undertaken to develop an understanding of the effect of various levels of change in the hydrologic regime on the project at hand. Specifically, it is suggested that increases of 10%, 20% and 30% to rainfall intensity be considered.

8.2.2. Sea Level Rise

The *NSW Sea Level Rise Policy Statement* was released by the NSW Government in October 2009. This Policy Statement was accompanied by the *Derivation of the NSW Government's sea level rise planning benchmarks* (NSW State Government, 2009) which provided technical details on how the sea level rise assessment was undertaken. Additional guidelines were issued by OEH, including the *Flood Risk Management Guide: Incorporating sea level rise benchmarks in flood risk assessments 2010.*

The Policy Statement says:

> *"Over the period 1870-2001, global sea levels rose by 20 cm, with a current global average rate of increase approximately twice the historical average. Sea levels are expected to continue rising throughout the twenty-first century and there is no scientific evidence to suggest that sea levels will stop rising beyond 2100 or that current trends will be reversed… However, the 4th Intergovernmental Panel on Climate Change in 2007 also acknowledged that higher rates of sea level rise are possible"* (NSW State Government, 2009)

In light of this uncertainty, the NSW State Government's advice is subject to periodical review. As of 2012, the NSW State Government withdrew endorsement of sea level rise predictions (which were for projected sea level rise of 0.4 m by 2050 and 0.9 m by 2100) but still require sea level rise to be considered.

8.3. Results

Comparison of peak flood levels are shown in the following tables, with emphasis given to impacts greater than 0.1 m (yellow) and impacts greater than 0.3 m (red).

8.3.1. Routing Lag

Overall peak flood level results were shown to be relatively insensitive to variations in the routing parameter. The increase and decrease to peak flood levels was within ± 0.01 m. The flood level variations were limited to the flow path areas; where decreasing the routing lag increased the peak flood level and the converse occurred when increasing the routing lag.

8.3.2. Manning's Roughness

Peak flood level impacts resulting from increasing and decreasing the Manning's roughness value was limited to the primary flow path areas and flood storage areas. Areas of shallow overland flow displayed little to no impact in peak flood levels.

Conveyance areas had decreasing flood levels with decreasing Manning's value. Flood storage areas had increasing flood levels with decreasing Manning's value. The converse occurred when increasing the Manning's value.

The increase and decrease to peak flood levels was minimal and generally within ± 0.025 m.

8.3.3. Pipe Blockage

Blockage of the stormwater pipes resulted in marginal increases in peak flood levels that were limited to the flow path areas. Blockage of 20% resulted in 0.01 m increases to flood levels. Blockage of 50% resulted in 0.02 m increases to flood levels.

8.3.4. Rainfall Increase

The effect of increasing the design rainfalls by 10%, 20% and 30% has been evaluated for the 1% AEP rainfall event with impacts on peak flood levels observed along all flow paths throughout the study area. Generally, each incremental 10% increase in rainfall resulted in an approximate 0.01 m increase in peak flood levels. The 1% AEP event with a rainfall increase of 30% is approximately equivalent to a 0.2% AEP event under present conditions and an impact on flood levels is to be expected.

8.3.5. Sea Level Rise

Due to the steep terrain of the study area, sea level rise scenarios were found to have a marginal effect on peak flood levels in the study area. In the 2100 scenario (in which sea levels were increased by 0.9 m), the only areas with increases to peak flood levels were Anderson Park, Milsons Park and Reid Park (although the latter was outside the study area). These flood level increases were generally less than 0.4 m.

9. HOTSPOT DISCUSSION

Hotspots were identified by Council and the community consultation responses (refer to Figure 12A for hotspot locations). These are discussed below and shown on Figure 33 to Figure 36. Some detail is provided in the text below and on the figures in the hope that experienced Council engineers can review modelled behaviour and provide some feedback on modelling results.

9.1. Cassins Lane (North Catchment)

West Street and Cassins Lane, North Sydney, have been identified as an area for flood concern. In the 1% AEP flood event, the capacity of the pipes at this location are exceeded, causing flooding due to local overland flow on West Street and Cassins Lane. Flood waters in the 1% AEP event are carried east along Myrtle Street, meeting West Street at a small dip near the intersection. Exacerbating the flood problem, the eastern side of West Street then acts as an obstruction to the natural flow path, causing water to pool, up to depths of approximate 0.2 m. This water disperses before flowing between properties to Cassins Lane and then Miller Street.

Myrtle Street carries a peak overland flow of 0.45 m^3/s and piped flow of 0.20 m^3/s. Approximately 0.62 m^3/s from the north of West Street and 0.18 m^3/s from the south then intersect with flood waters from Myrtle Street. Since West Street's grade is low at 1%, water pools to depths of approximately 0.2 m at the intersection with Myrtle Street. In the 1% AEP flood event, a peak overland flow of 2.0 m^3/s and piped flow of 0.7 m^3/s carries flood water between properties on West Street toward Cassins Lane. The drainage at this hotspot is insufficient resulting in overland flow and pooled water around properties. In the 10% AEP event also, the capacity of the pipes across West Street is reached and this location carries a peak overland flow of 1.1 m^3/s.

9.2. Cooper Lane (North Catchment)

Cooper Lane in Neutral Bay has experienced flooding in the past and is known to Council as a flooding hotspot. This location has an upstream catchment of 3.8 hectares.

In the 1% AEP event, approximately 2.1 m^3/s of overland flow and 1.0 m^3/s of piped flow are carried down Young Street from Military Road. The overland flow divides at Grosvenor Lane with 1.0 m^3/s flowing down Cooper Lane and 1.1 m^3/s flowing down Young Street. Flood waters then meet again at the intersection of Grosvenor Street and Cooper Lane. In the 1% AEP flood event water pools at this intersection to depths of 0.3 m before flowing down Cooper Lane (approximately 1.6 m^3/s).

The capacity of the pipe flowing along the western side of Cooper Lane is reached in the 10% flood event. The pipe on the eastern side of Cooper Lane is 73% full in the 10% AEP event and full in the 1% AEP event. Generally speaking, the resultant flooding along Young Street, Grosvenor Street and Cooper Lane is relatively shallow due to the steep grades of these roads (3.7%, 4.3% and 4.9% grade respectively) although velocities are high.

9.3. High Street – Hipwood Street Junction (South Catchment)

At the intersection of Hipwood and High Street a number of properties are at and/or below the adjacent road level. As water flows down High Street from the north-west, some flow is diverted along Clark Street while the rest either pools in front of properties on the north side of High Street or continues south along Hipwood Street with some pooling around properties. High Street has a gradient of approximately 5% before plateauing at the intersection of Hipwood Street. Leading away from the intersection, Hipwood Street has a gradient of 16%. This sudden change in gradient at Hipwood Street, and just before it with properties being 3 m below High Street, diverts flow through the properties or down Hipwood Street.

In the 1% AEP event a peak discharge of 1.6 m^3/s flows down High Street, some of this being diverted down Clark Road (approximately 0.5 m^3/s). This flow, combined with local catchment flow, increases to 1.5 m^3/s down Hipwood Street and around the properties at the intersection of Hipwood and High Street. Drainage in the area consists of pipes starting at the north side of High Street and continuing down to Hipwood Street.

As Hipwood Street has a high gradient, water does not pool along the road. Water does pool in front of the property at the intersection of High and Hipwood Street up to 0.7 m in a 1% AEP event and 0.6 m in a 10% AEP event. On the north side of High Street, water accumulates in a large area in front of two properties. Pooled water reaches a peak depth of 0.45 m in a 1% AEP event.

Subsequent to the commencement of this study, mitigation works have been undertaken by Council at this location. Future FRMS&P will include this mitigation work in the hydraulic model schematisation.

9.4. Rawson Street – Kurraba Road Junction (South Catchment)

The open channel flows down parallel to Rawson St with a peak flow of 21.0 m^3/s. This is supplemented by pipe flow from the north-west and other minor lateral tributaries. Flow under Kurraba Road is facilitated via culvert which passes only 13.5 m^3/s in the 1% AEP event. Flow in excess of culvert capacity fills the channel with flood waters spilling out onto the intersection and adjoining areas. Generally, this flood surcharge then moves south along Clark Road with much of the flow rejoining the channel downstream of Kurraba Road adjacent to Anderson Park.

The profile shows that Kurraba Road is an important control on upstream flood levels, whilst downstream flood levels are influenced by ocean levels.

10. PUBLIC EXHIBITION

North Sydney Council carried out the public exhibition of the North Sydney LGA Flood Study over the period of the 28th July to the 8th September 2016. The public exhibition period was communicated to the community via the Council website, a media release in the local newspaper, and a letter posted to 5,000 properties within the broader floodplain area. Community information sessions were also held at various locations across the LGA; with 6 information sessions, each approximately 2 hours in duration. The owners of 179 properties attended the community information sessions.

During the public exhibition period a number of enquiries were made and 90 submissions were received. Many of the submissions raised multiple issues. The main issues raised have been identified and described below. Furthermore, all submissions have been replied to individually.

A major concern raised by most respondents (67 submissions) was that their property should not be included in the Flood Planning Area (FPA)). To confirm, this flood study has not identified properties to be included in the FPA; this process will be carried out in the subsequent Floodplain Risk Management Study and Plan. Additional concerns accompanied by this issue include the definition of the broader floodplain area, the definition of flooding (be it overland flooding or mainstream flooding) and a preference for the delineation of the range of risk across different properties (i.e. properties with a lower risk differentiated from properties with a higher risk).

The second most prevalent matter (21 submissions) included the desire for Council to improve the stormwater drainage infrastructure and often indicated that it was their belief that if such measures were taken (or if the existing infrastructure were better maintained) then they would no longer be affected by flooding.

The financial implications of the Flood Study on the resident (particularly by way of increased insurance costs or decreased property values) were a concern raised in 12 of the submissions.

Seven respondents were concerned that the outcome of the Flood Study would be a restriction on future development on their property.

Given all submissions have received written responses and this study is preliminary to further stages of the overall FRMP, the community consultation works is for the moment considered to be complete.

11. CONSIDERATIONS MOVING FORWARD

The Study defines design flood levels for the study area which is the entire LGA of 10.9 km^2. Moving forward the best possible use of this work would include the following steps:

- Initiate a Floodplain Risk Management Study and Draft Plan – the FRMS&P is Stage 2 of the three-stage NSW Floodplain Risk Management Program. It produces outputs suitable for use by Council in its ongoing effort to manage flood risk such as final hazard and hydraulic category maps as well as leading to DCPs relevant to flooding and a finalised set of property for which 149(2) certificates will be required (i.e. the flood planning area);
- Utilise design mapping presented herein within Council's own GIS in order to inform development applications and other Council planning where relevant; and
- Optionally, where development is proposed for flood prone lots, Council could have developers utilise the model developed as part of the flood study reported upon herein. This provides a service to developers as they should be able to avoid the cost of individual model setup. It also serves Council and its residents as it ensures a base standard for modelling work carried out in support of flood affected development. The model should be used to ensure that proposed development does not exacerbate flood levels on lots other than those proposed for development.

12. ACKNOWLEDGEMENTS

WMAwater wish to acknowledge the assistance of North Sydney Council staff in carrying out this study, as well as the NSW Government (Office of Environment and Heritage) and the residents of the North Sydney LGA. This study was jointly funded by North Sydney Council and the NSW Government.

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11. North Sydney Council

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North Sydney Council, December 1993

13. Pilgrim DH (Editor in Chief)

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Institution of Engineers, Australia, 1987.

Figures

FIGURE 1
STUDY AREA

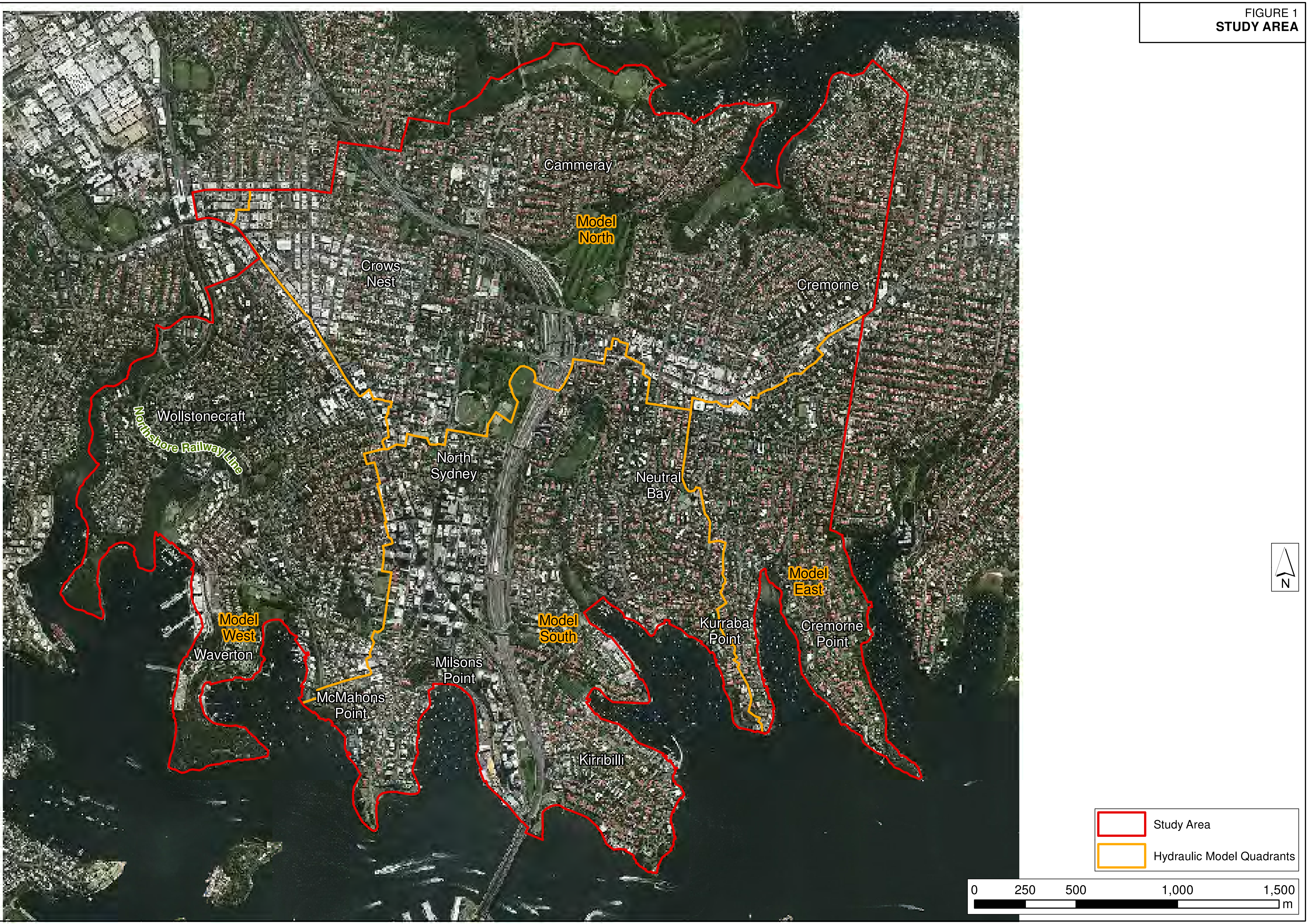

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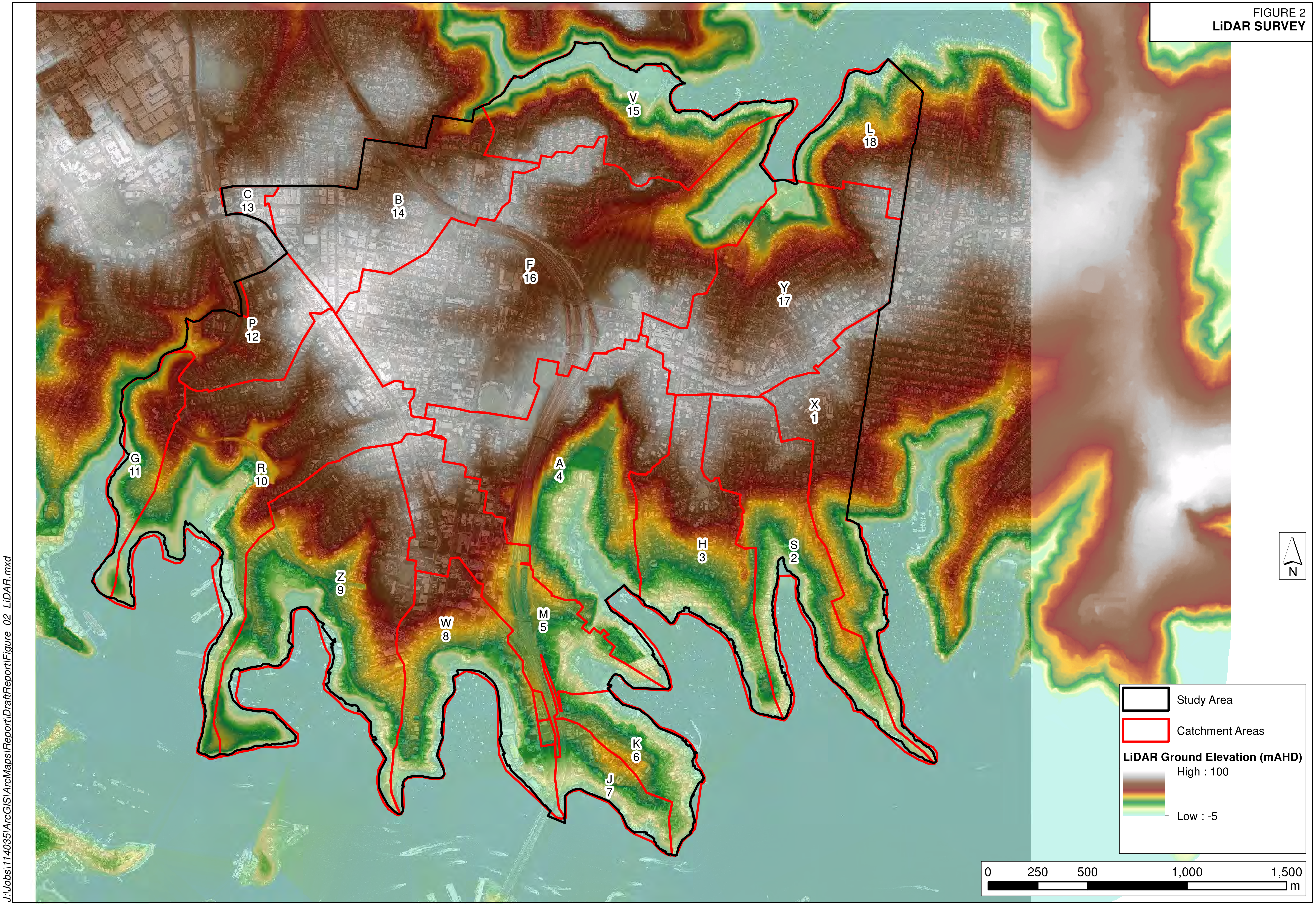
FIGURE 2
LiDAR SURVEY



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PHOTOGRAPHS FROM WMAwater SITE VISIT

Above: Open Channel Downstream of Eaton Street
(18/06/2014)

Above: Darley Street Culvert
(18/06/2014)

Above: CDS unit located at Junction of Rawson Street and Kurraba Road
(18/06/2014)

Above: Kurraba Road Culvert
(18/06/2014)

FIGURE 4
RAINFALL GAUGE LOCATIONS

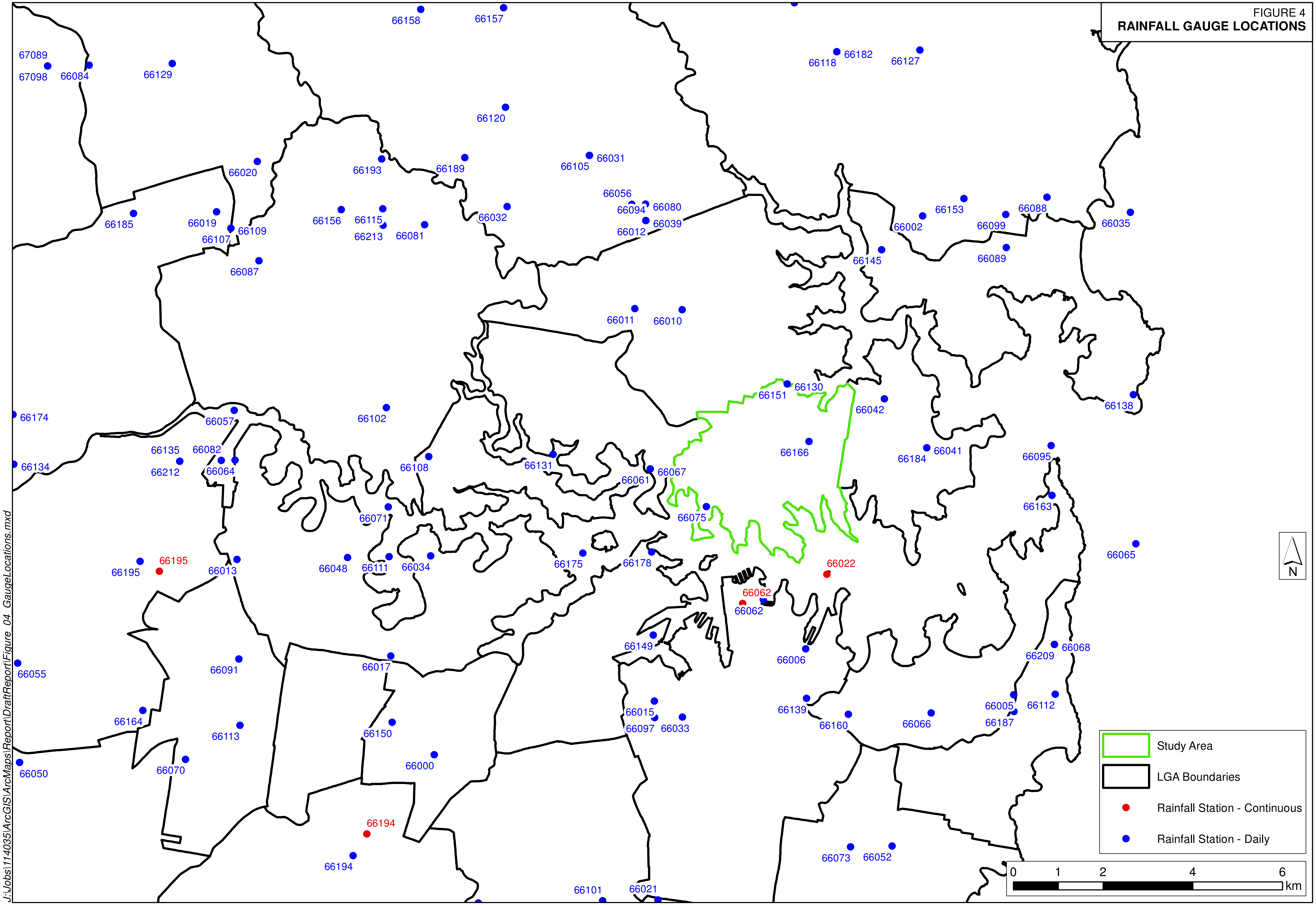

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FIGURE 5A
STAGE HYDROGRAPH
8TH NOVEMEBER 1984 EVENT

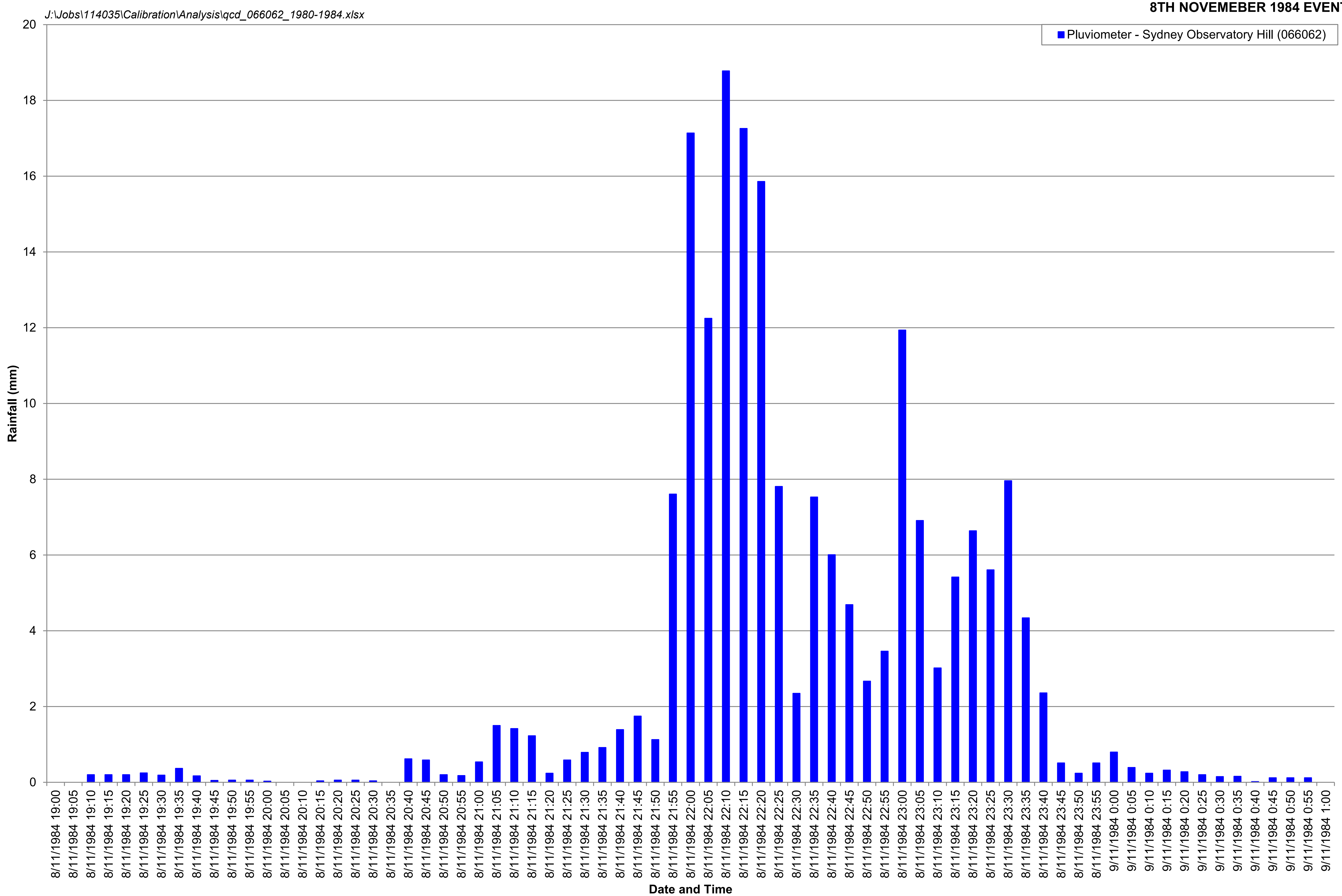

FIGURE 5B
STAGE HYDROGRAPH
26TH JANUARY1991 EVENT

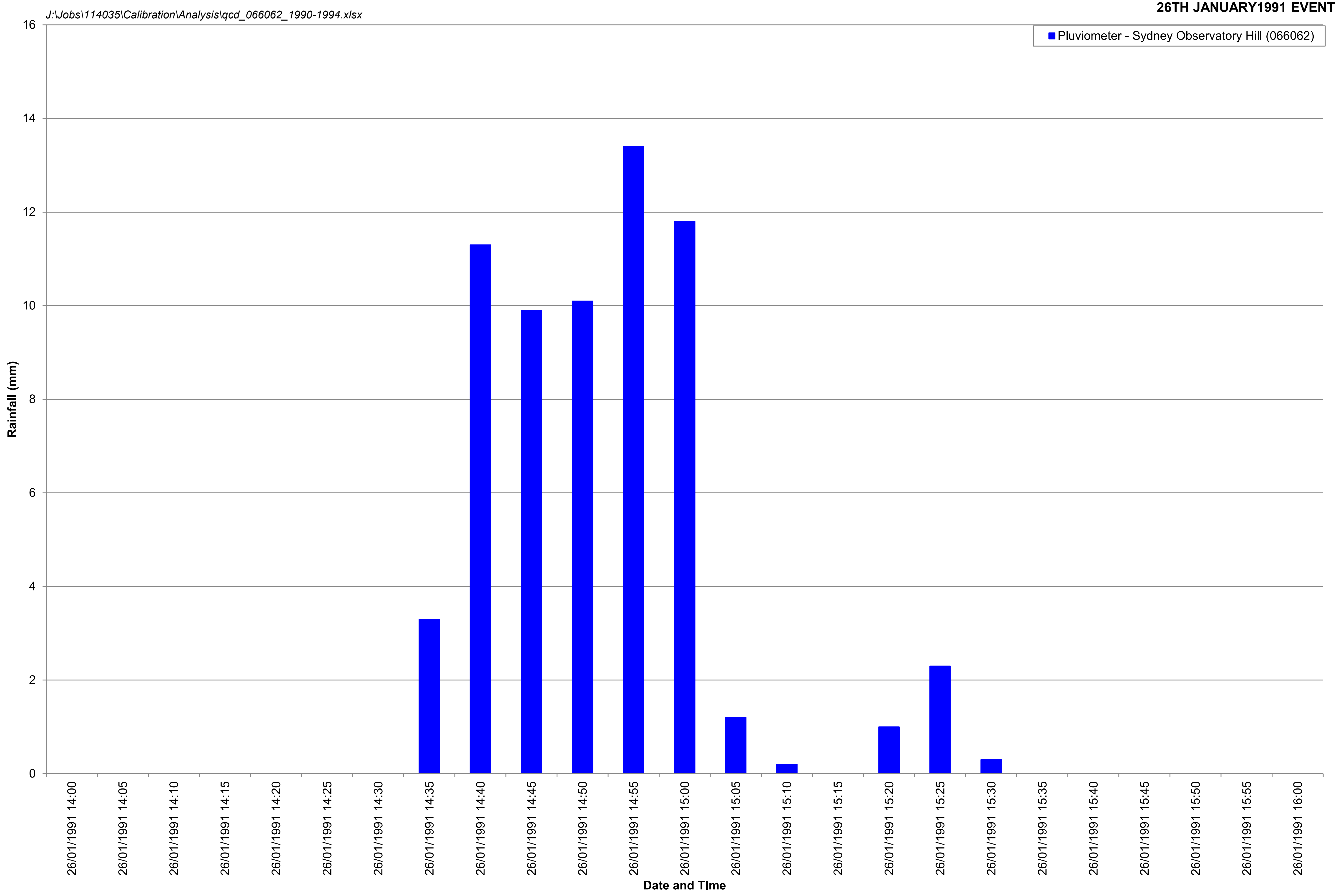

FIGURE 5C

STAGE HYDROGRAPH

12TH FEBRUARY 2010 EVENT

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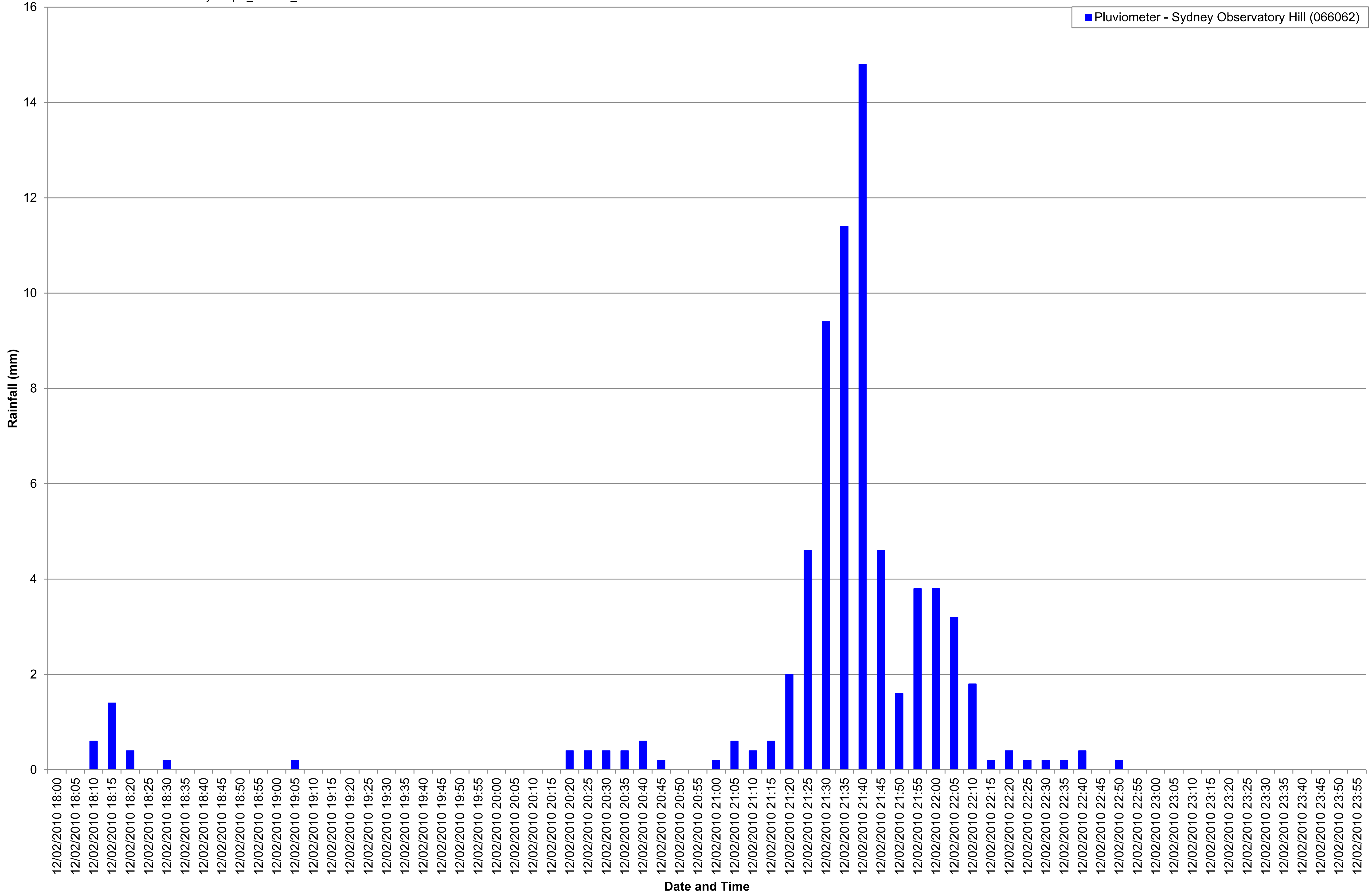

FIGURE 6A
BURST INTENSITIES AND FREQUENCIES
8TH NOVEMBER 1984 EVENT

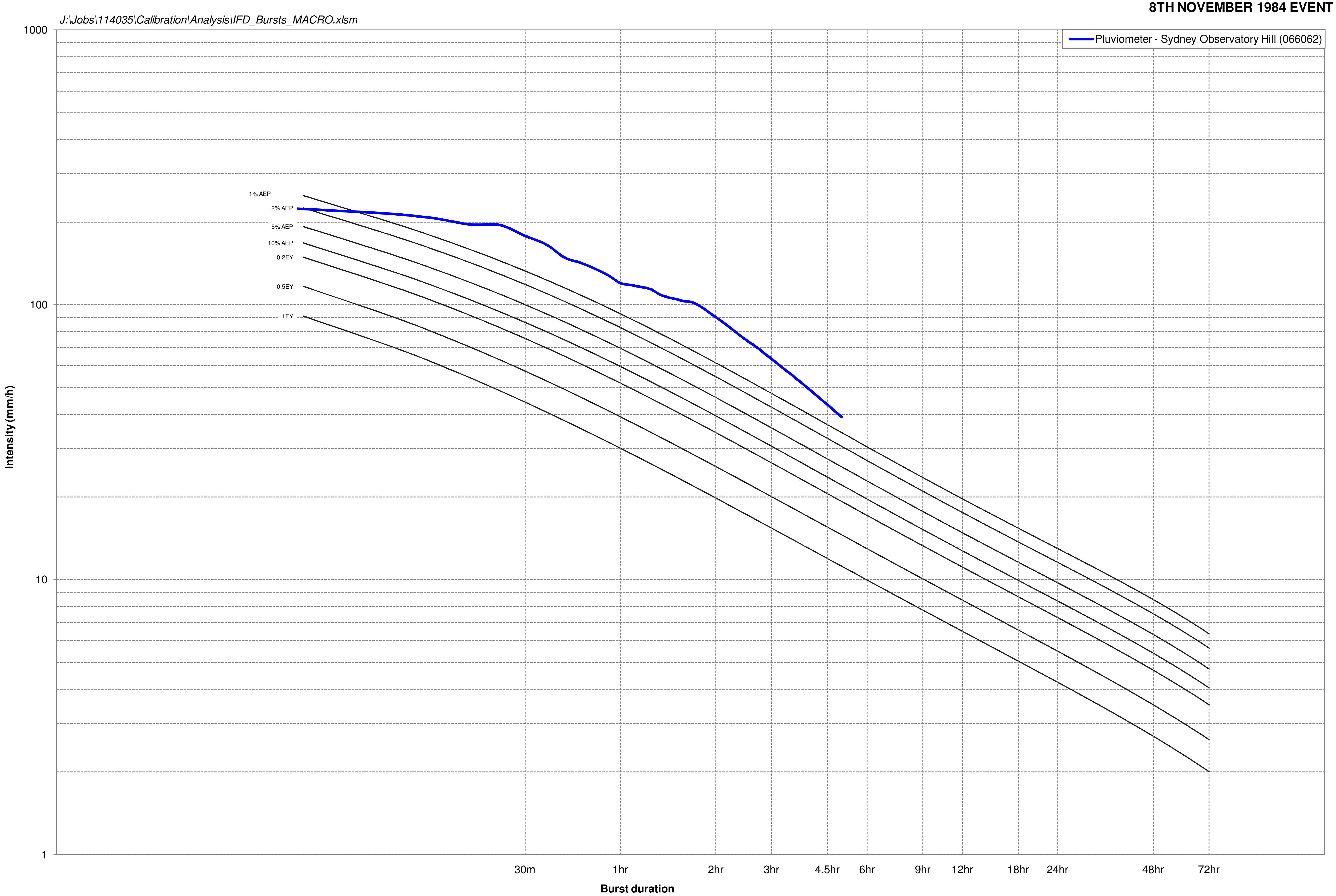

FIGURE 6B
BURST INTENSITIES AND FREQUENCIES
26TH JANUARY 1991 EVENT

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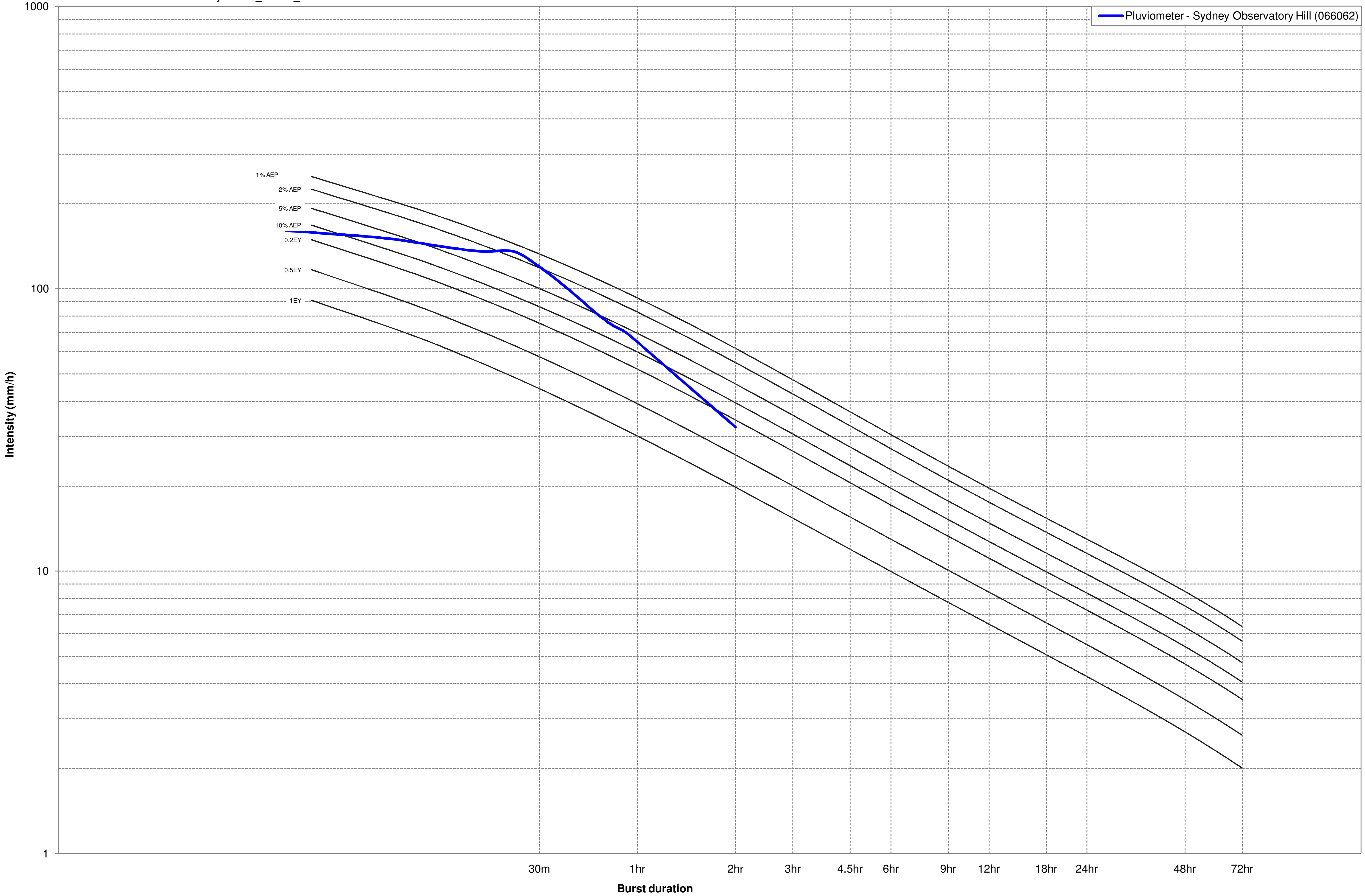

BURST INTENSITIES AND FREQUENCIES
12TH FEBRUARY 2010 EVENT

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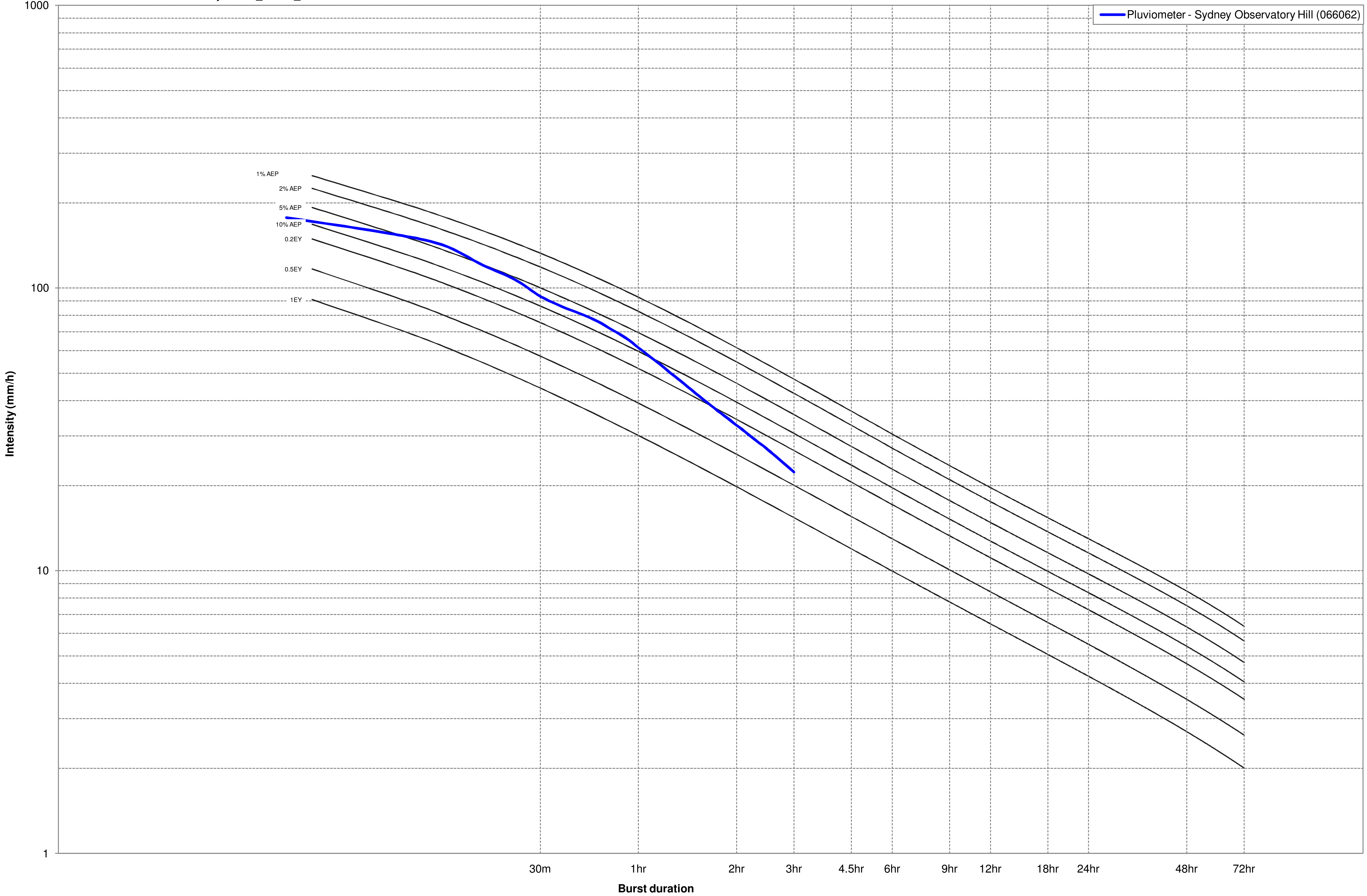
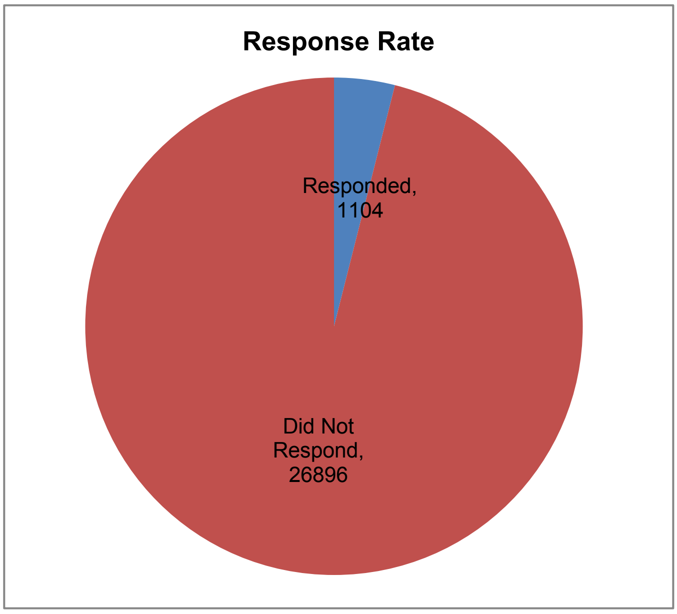
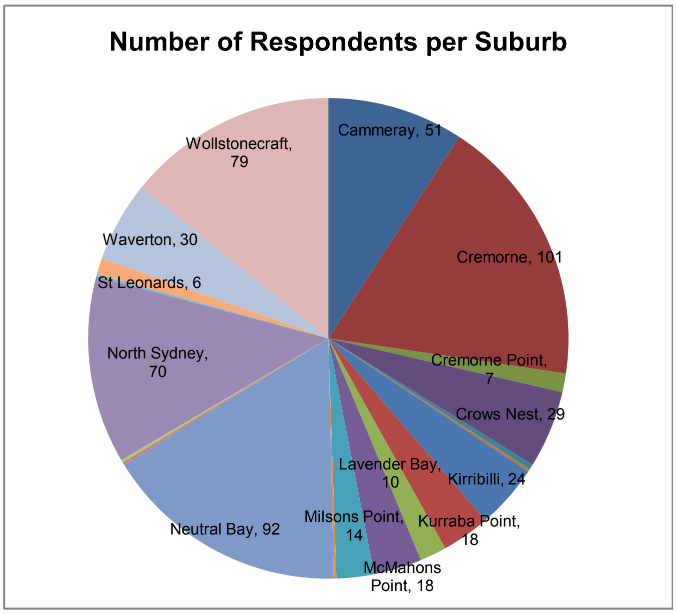
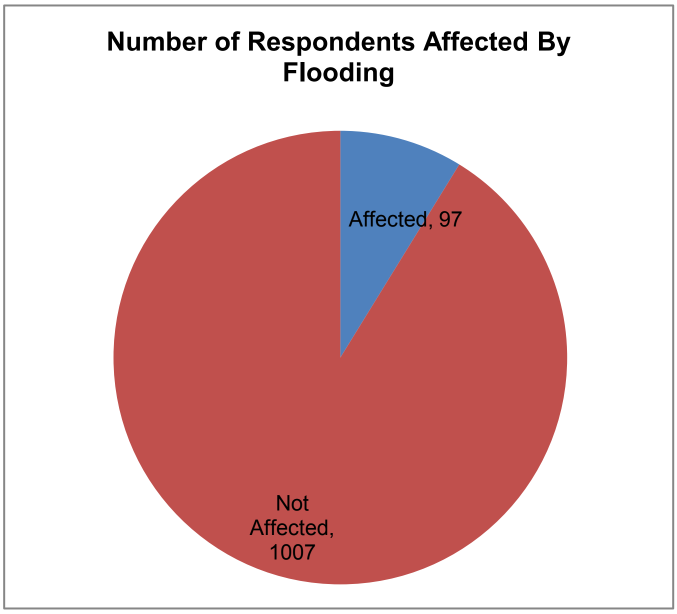
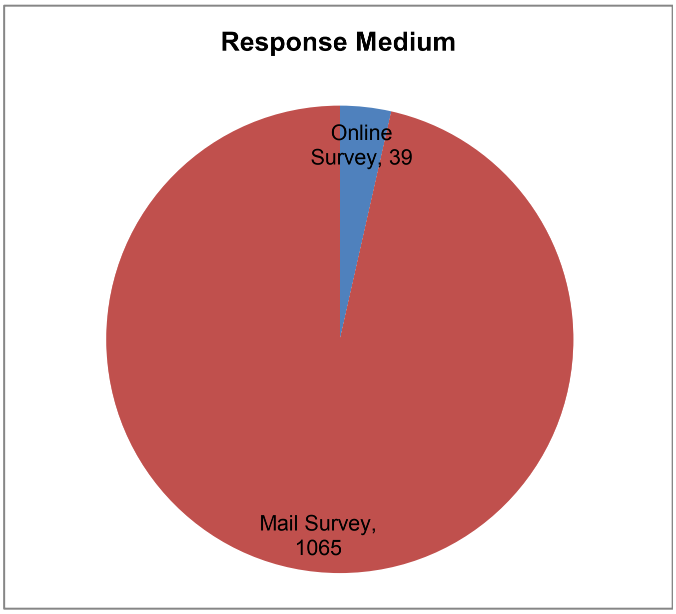


FIGURE 7A
COMMUNITY CONSULTATION RESPONSE ANALYSIS









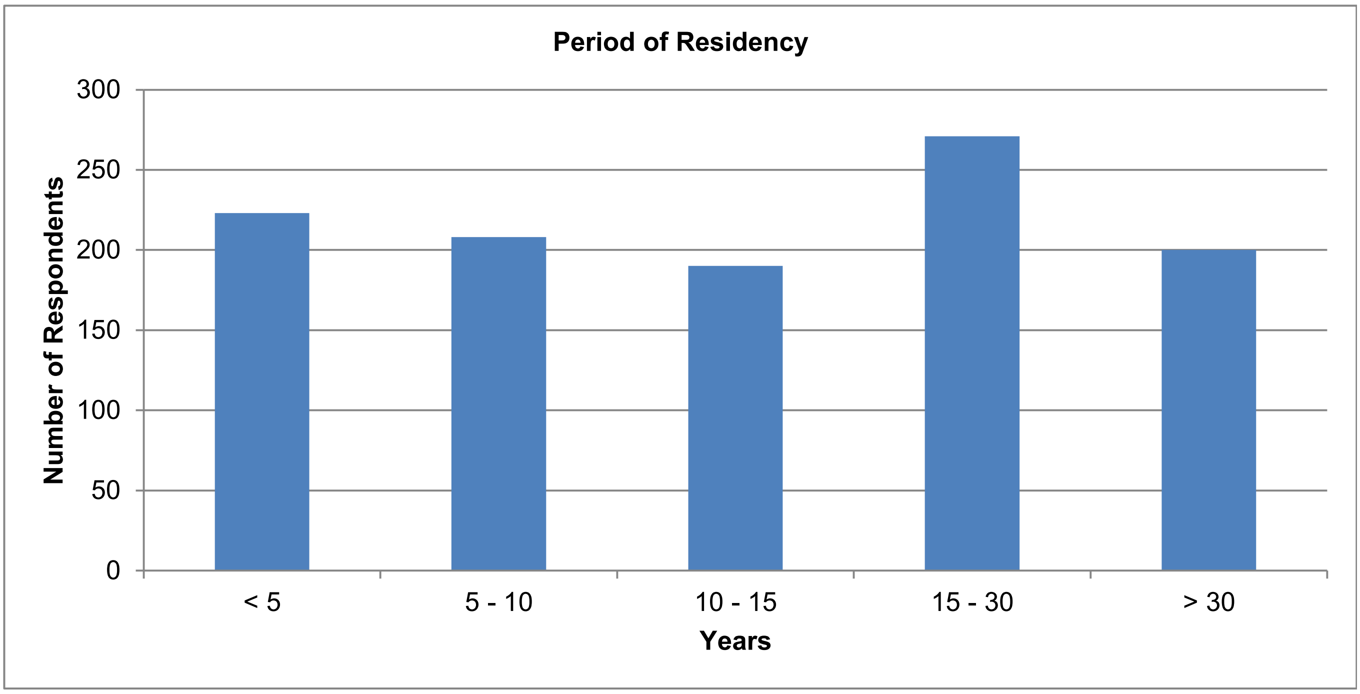


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FIGURE 07B
COMMUNITY CONSULTATION RESPONSE LOCATION

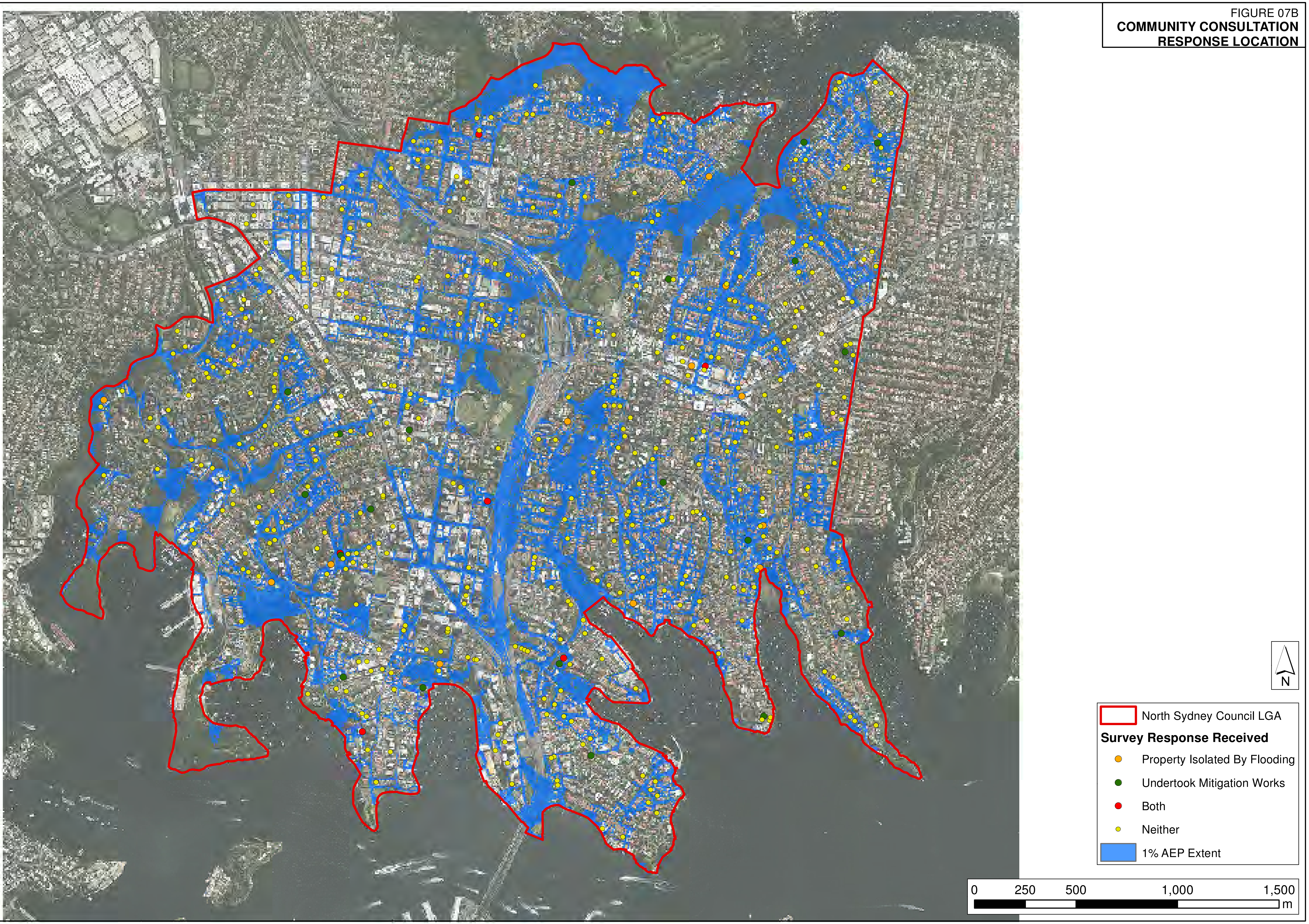

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FIGURE 8
HYDROLOGIC MODEL SCHEMATISATION

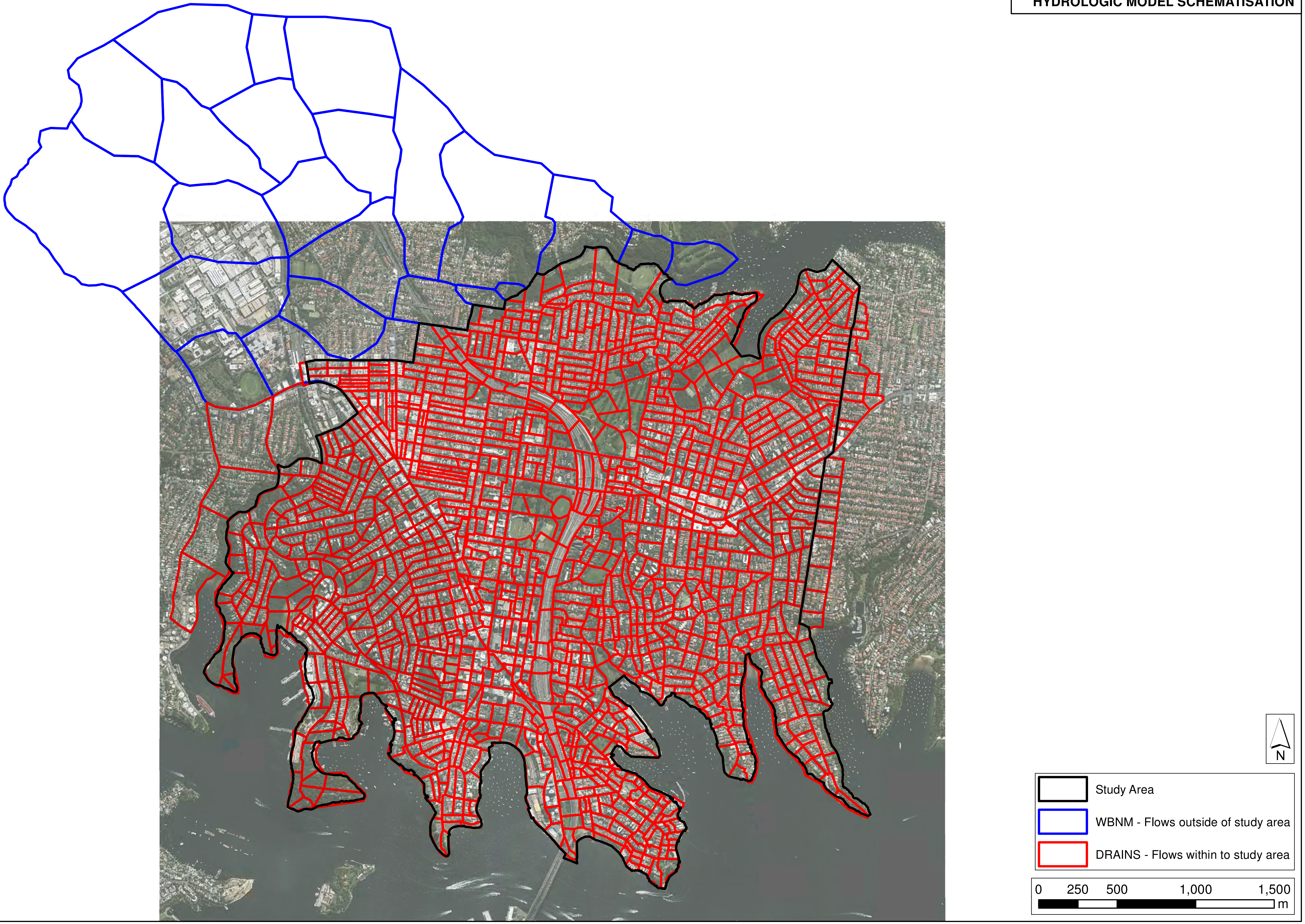

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FIGURE 9
HYDRAULIC MODEL SCHEMATISATION

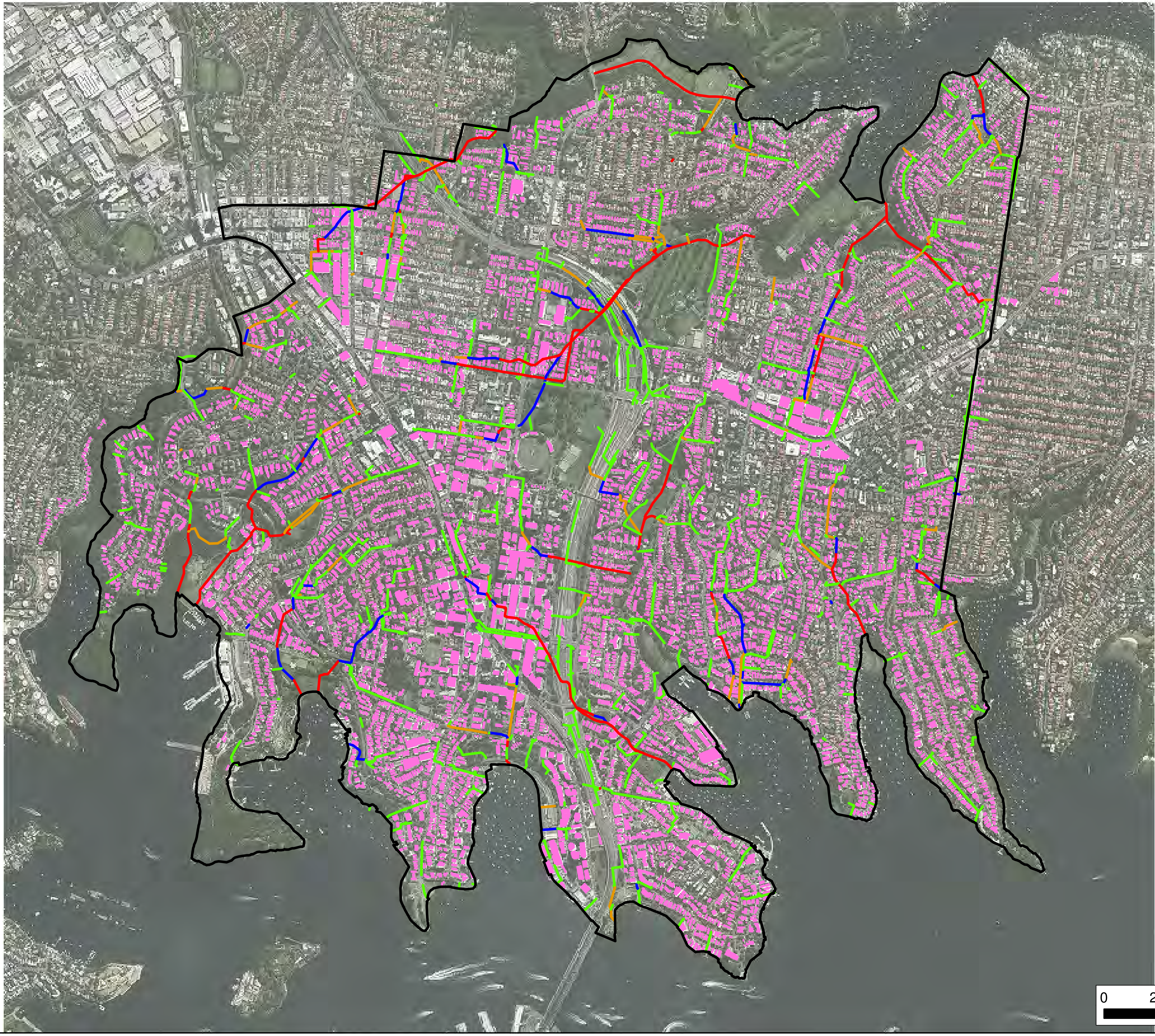

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FIGURE 10
HYDRAULIC MODEL ROUGHNESS VALUES

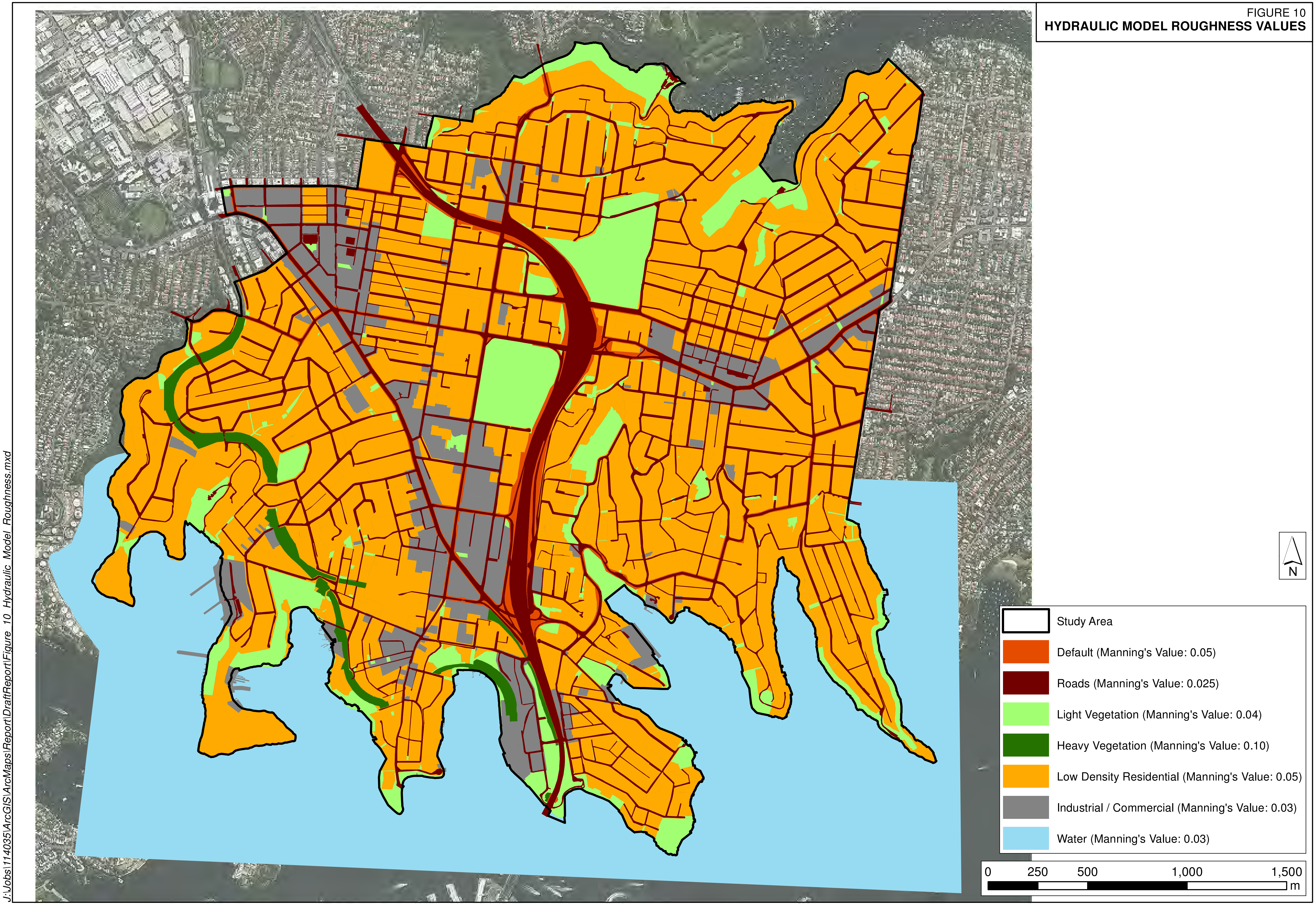

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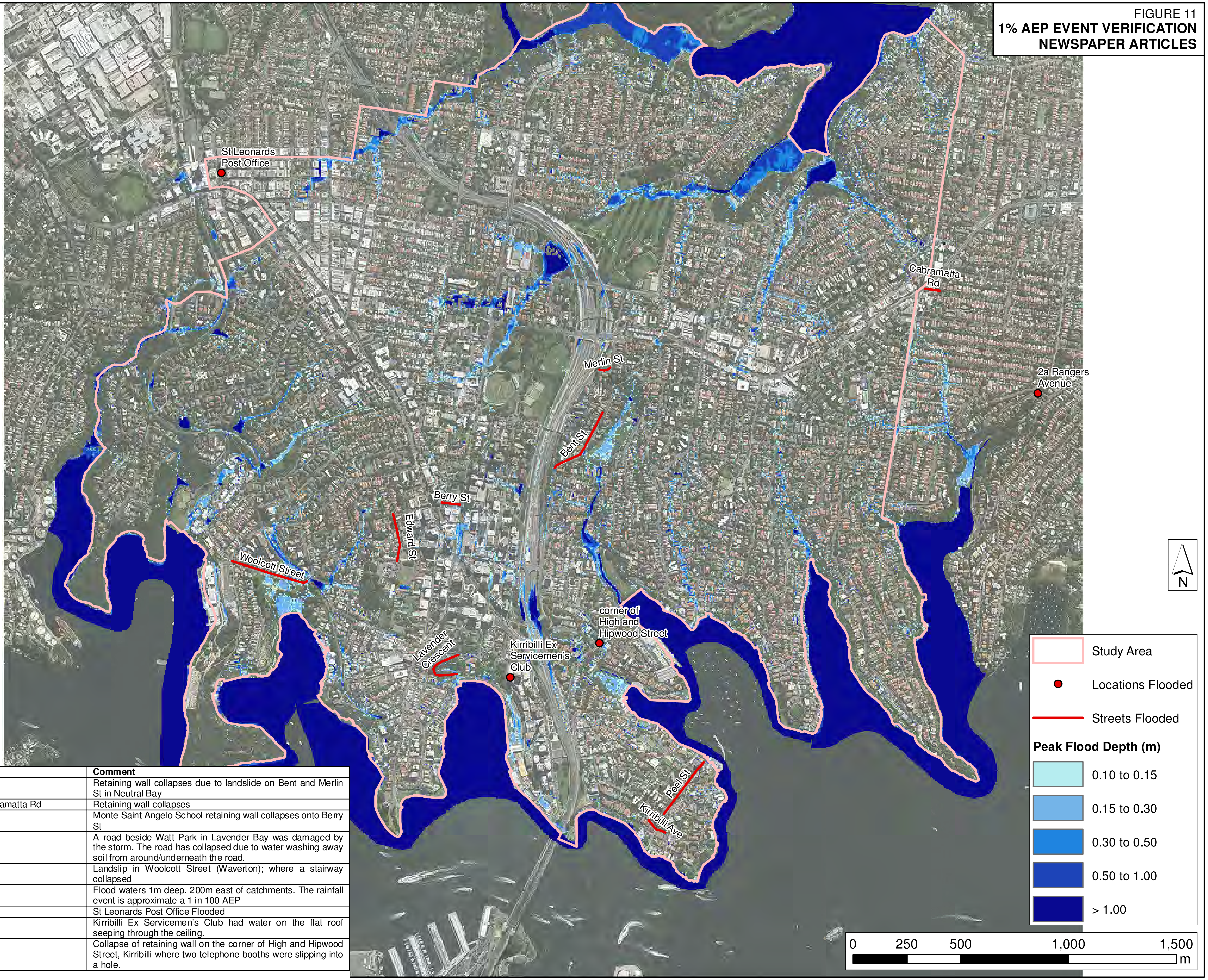
FIGURE 11
1% AEP EVENT VERIFICATION NEWSPAPER ARTICLES



Location	Comment
Bent and Merlin St	Retaining wall collapses due to landslide on Bent and Merlin St in Neutral Bay
Edward St, Peel St, Kirribilli Ave and Cabramatta Rd	Retaining wall collapses
Berry St	Monte Saint Angelo School retaining wall collapses onto Berry St
Lavender Crescent	A road beside Watt Park in Lavender Bay was damaged by the storm. The road has collapsed due to water washing away soil from around/underneath the road.
Woolcott Street	Landslip in Woolcott Street (Waverton); where a stairway collapsed
2a Rangers Avenue	Flood waters 1m deep. 200m east of catchments. The rainfall event is approximate a 1 in 100 AEP
St Leonards Post Office	St Leonards Post Office Flooded
Kirribilli Ex Servicemen's Club	Kirribilli Ex Servicemen's Club had water on the flat roof seeping through the ceiling.
corner of High and Hipwood Street	Collapse of retaining wall on the corner of High and Hipwood Street, Kirribilli where two telephone booths were slipping into a hole.

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FIGURE 12
1% AEP EVENT VERIFICATION
UNIT FLOW RATE MAP

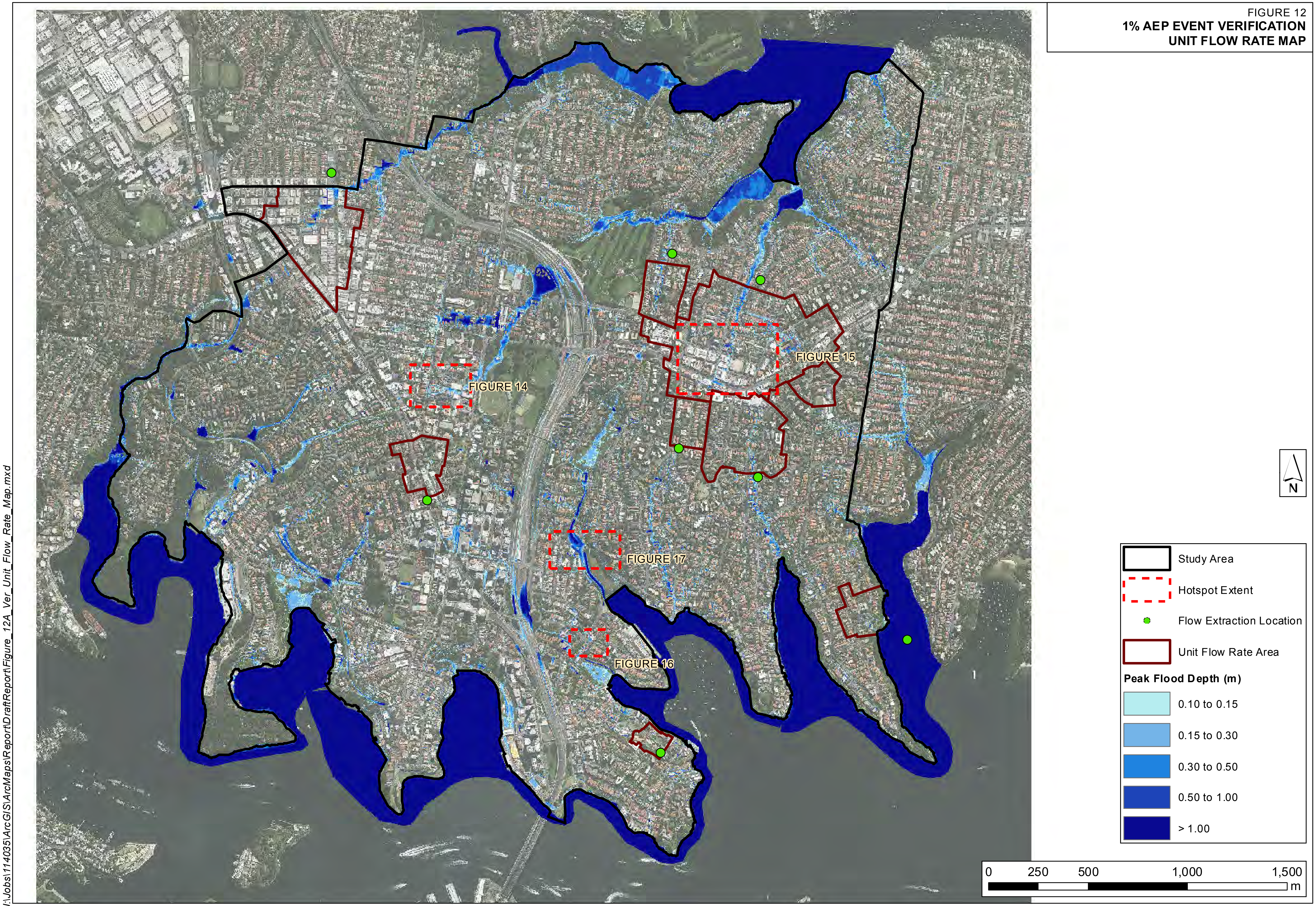

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FIGURE 13
1% AEP EVENT VERIFICATION COMMUNITY CONSULTATION

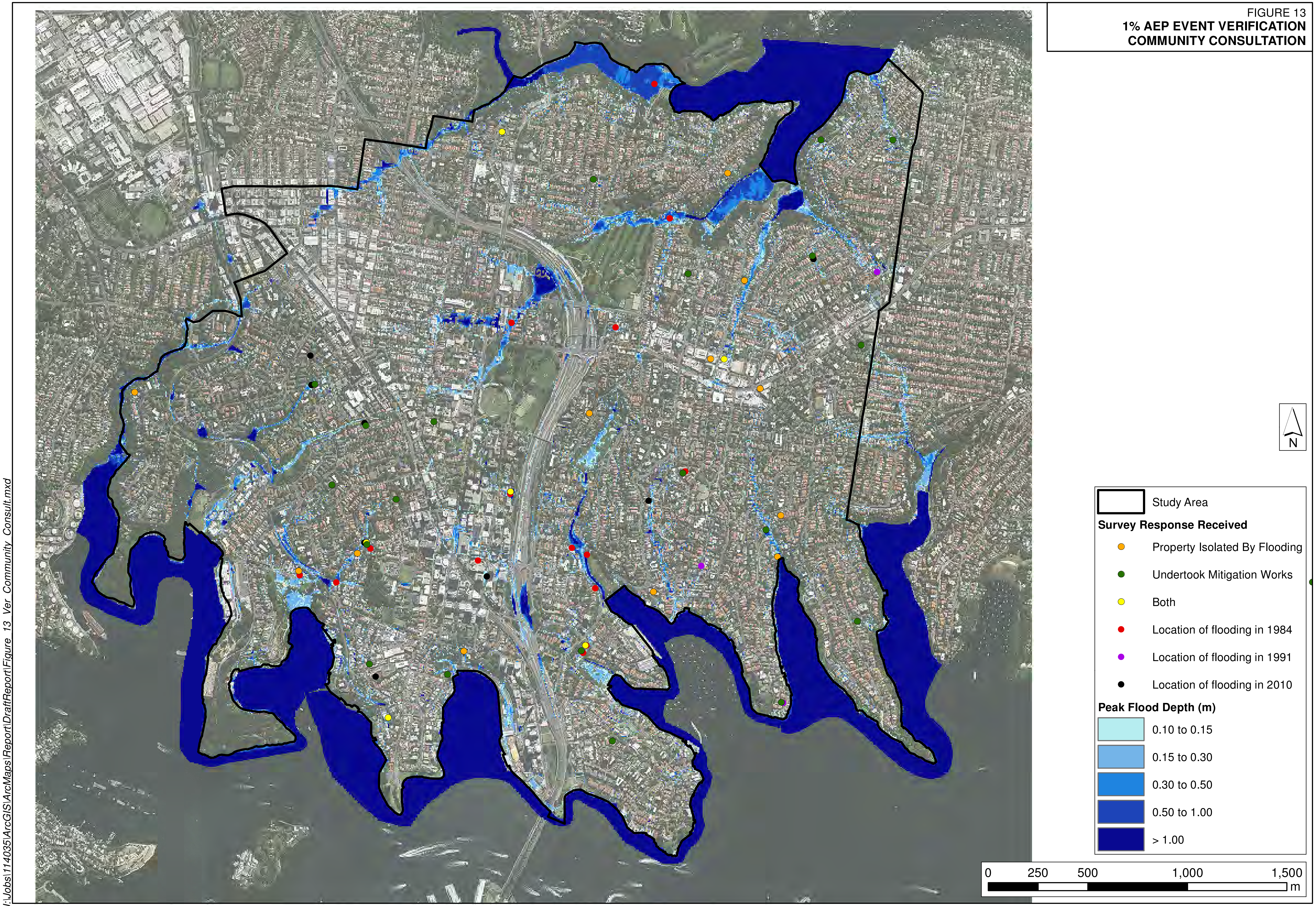

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FIGURE 14
RESULTS LAYOUT

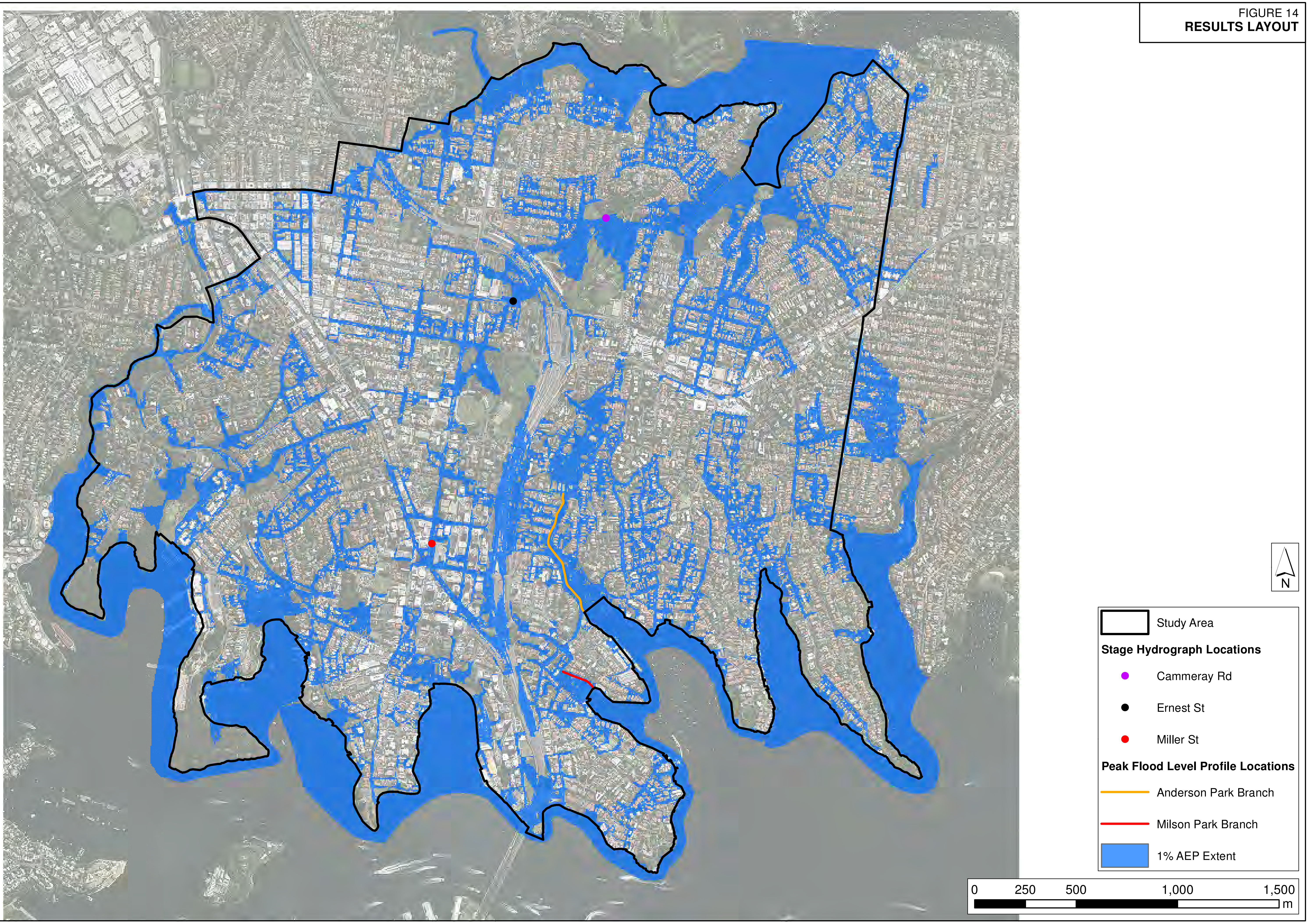

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FIGURE 15A
PEAK FLOOD LEVEL PROFILES
ANDERSON PARK BRANCH

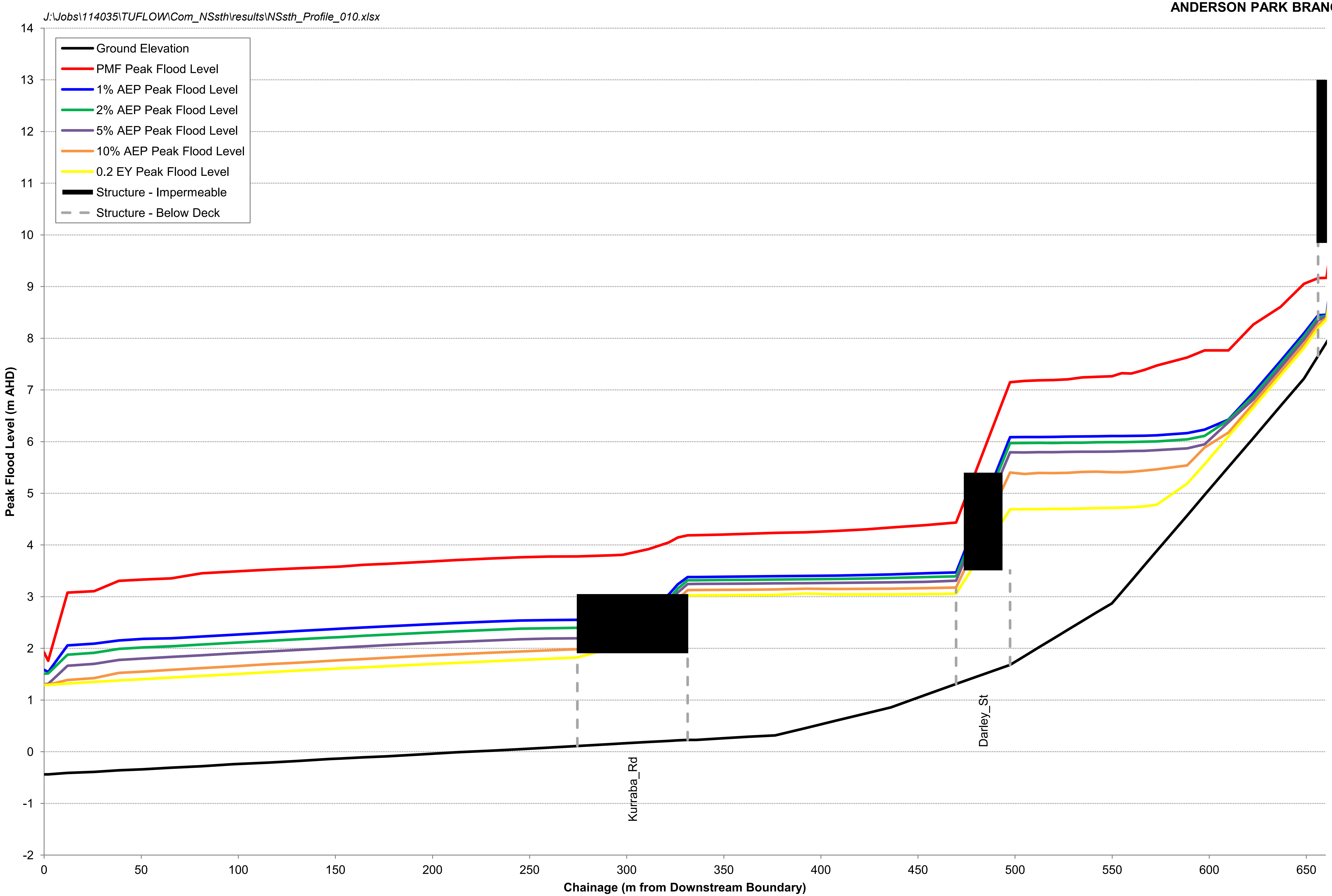
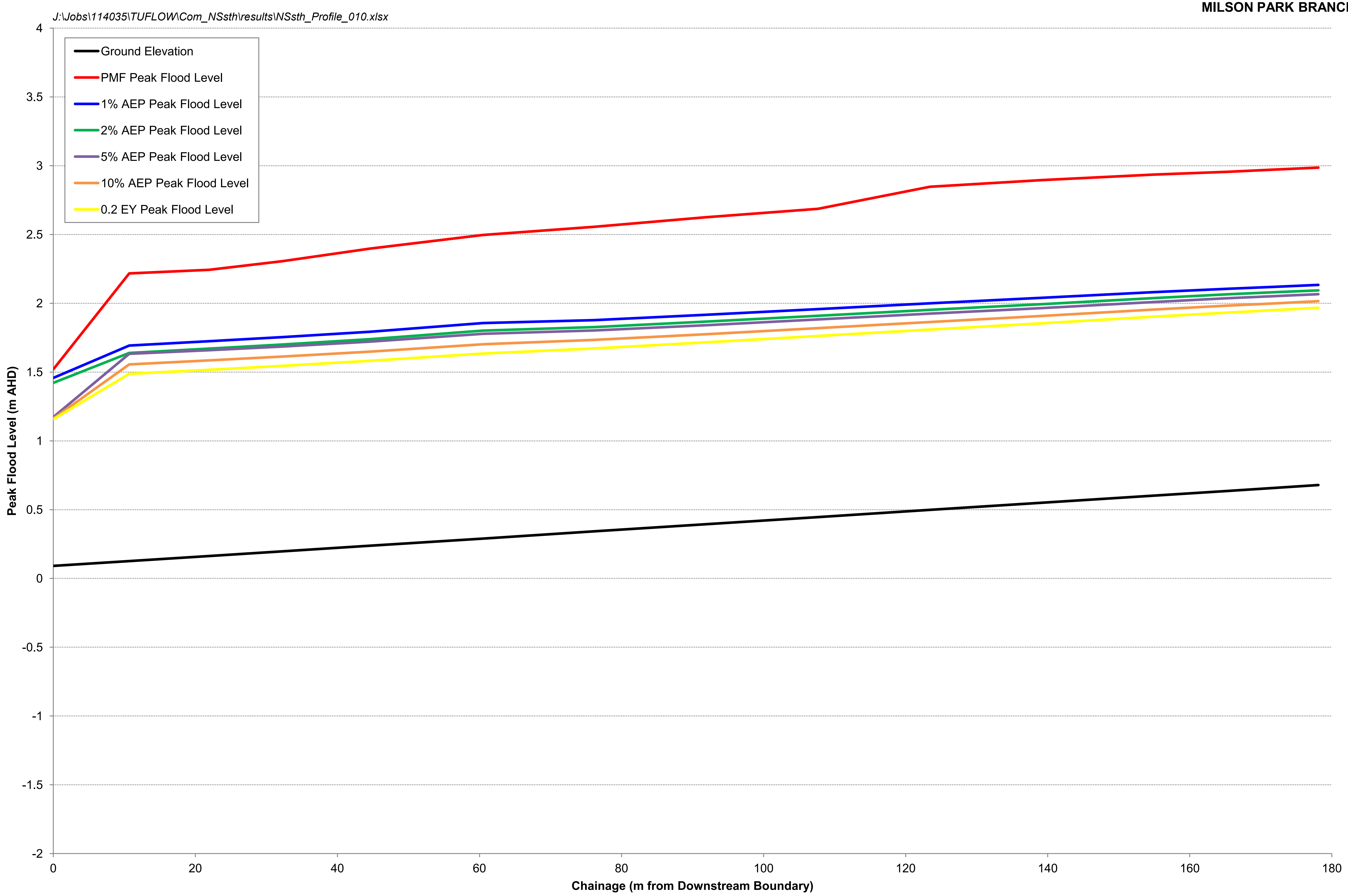


FIGURE 15B
PEAK FLOOD LEVEL PROFILES
MILSON PARK BRANCH



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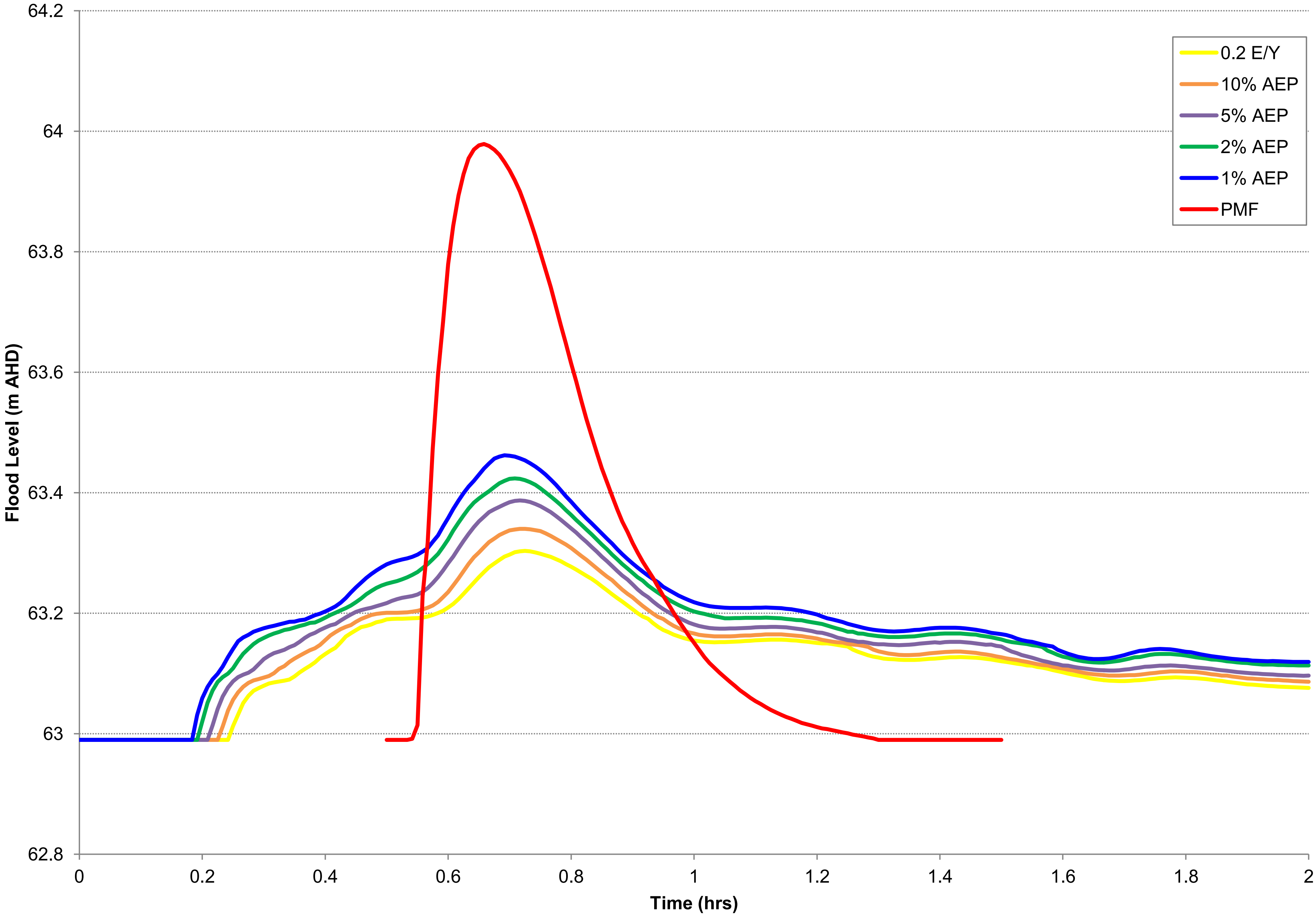

FIGURE 16A

DESIGN STAGE HYDROGRAPH

MILLER STREET

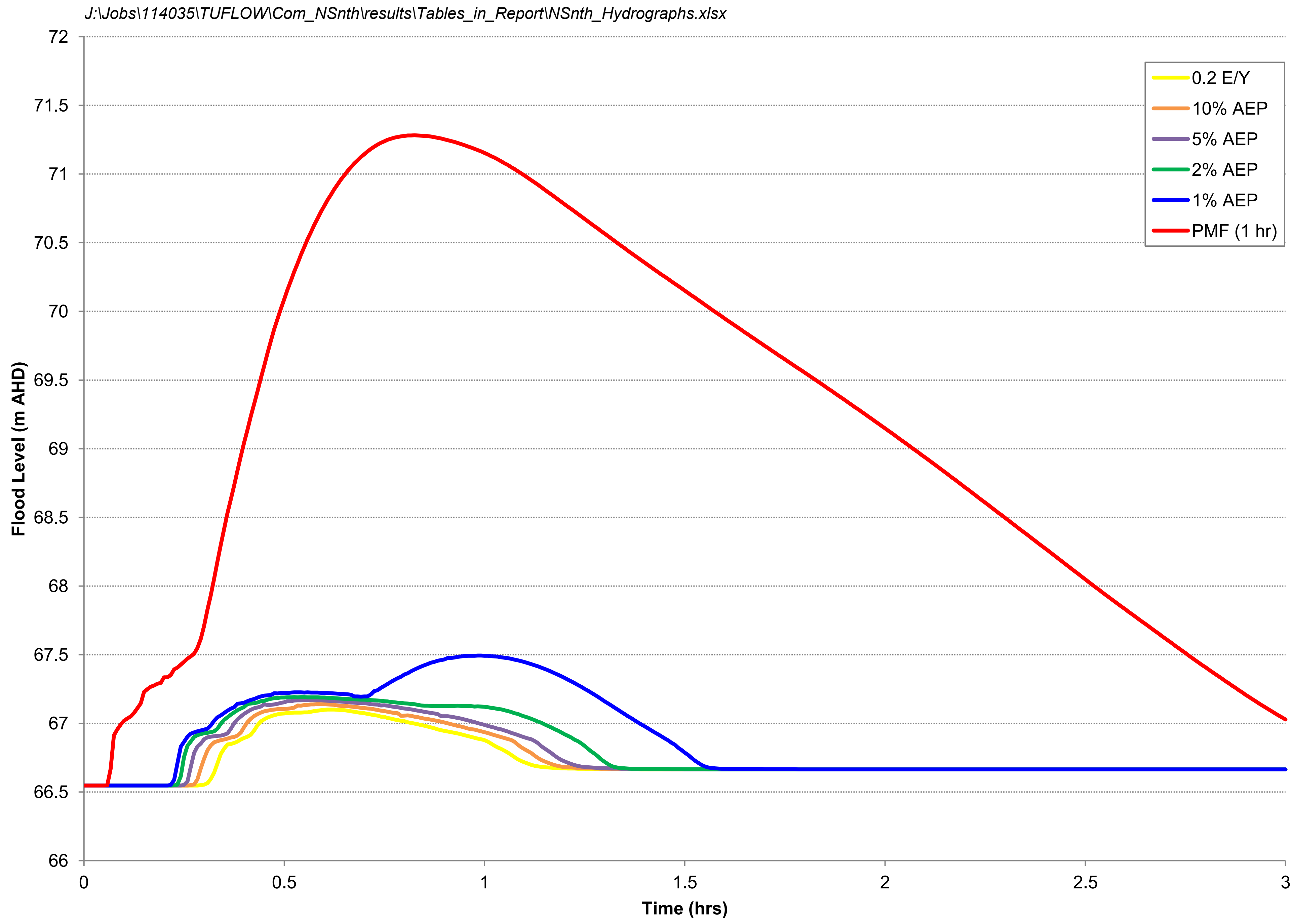

FIGURE 16B
DESIGN STAGE HYDROGRAPH
ERNEST STREET

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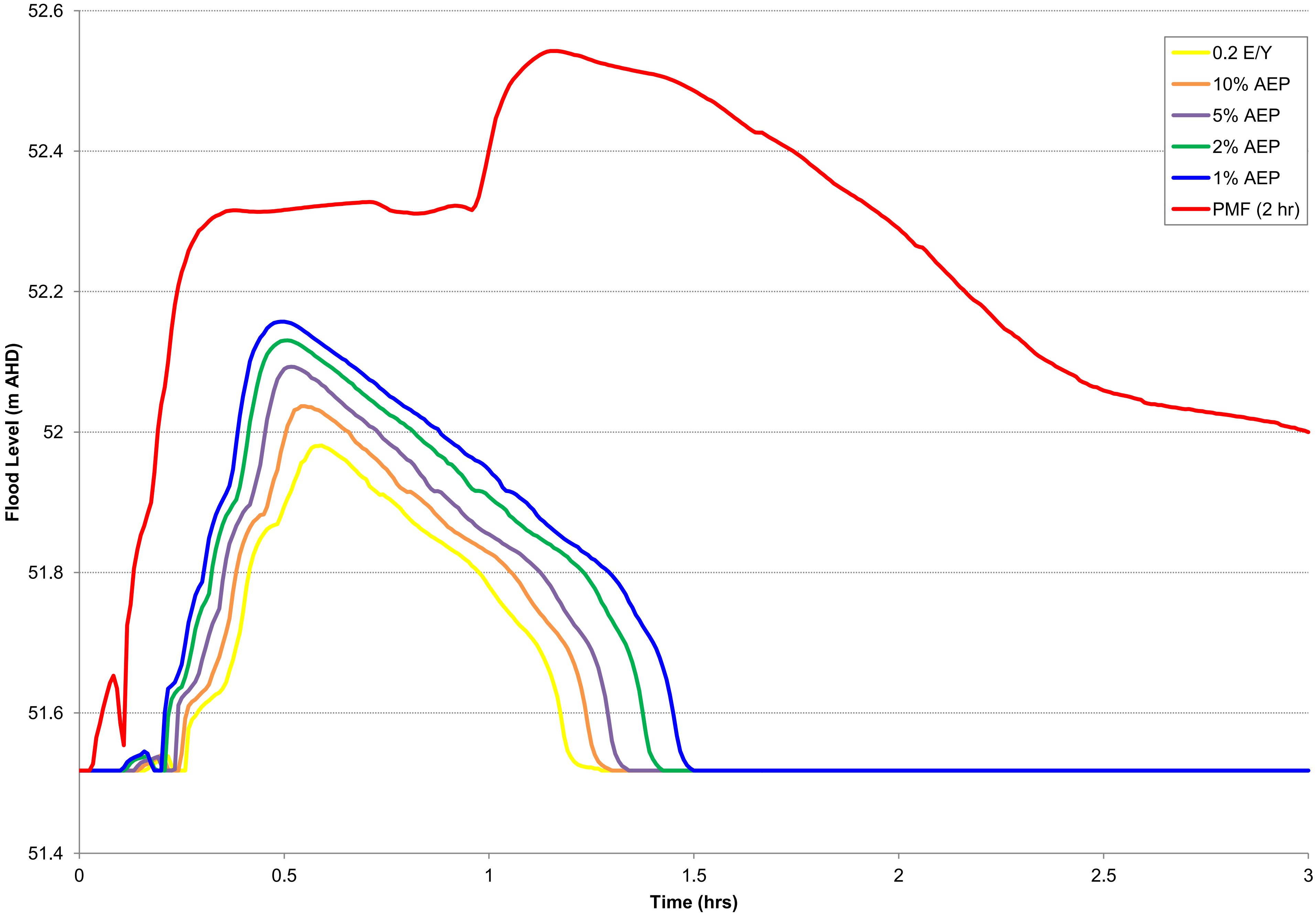

FIGURE 16C
DESIGN STAGE HYDROGRAPH
CAMMERAY ROAD

FIGURE 17
PIPE CAPACITY

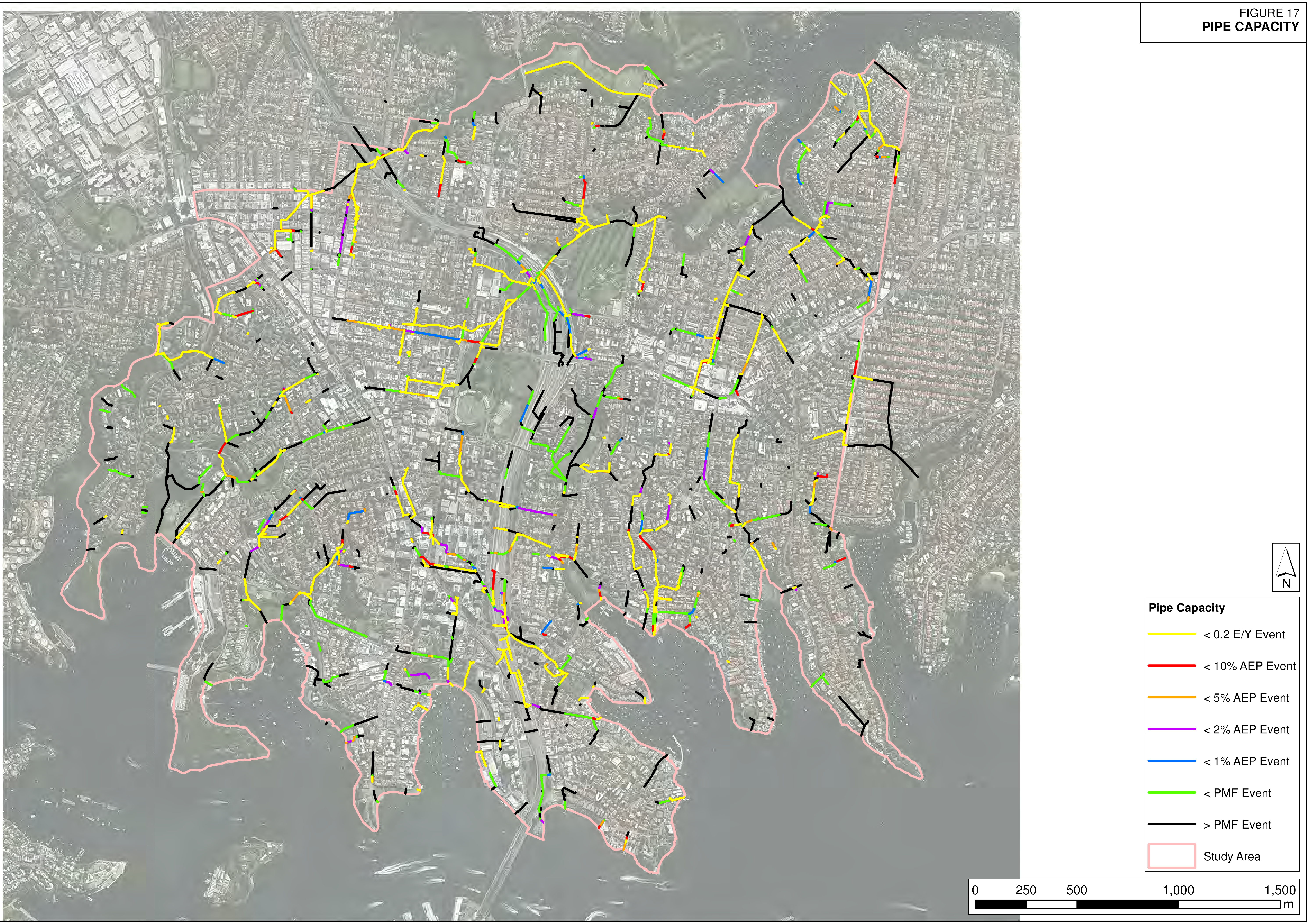

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FIGURE 18

PEAK FLOOD DEPTH AND FLOOD LEVEL CONTOURS 20% AEP

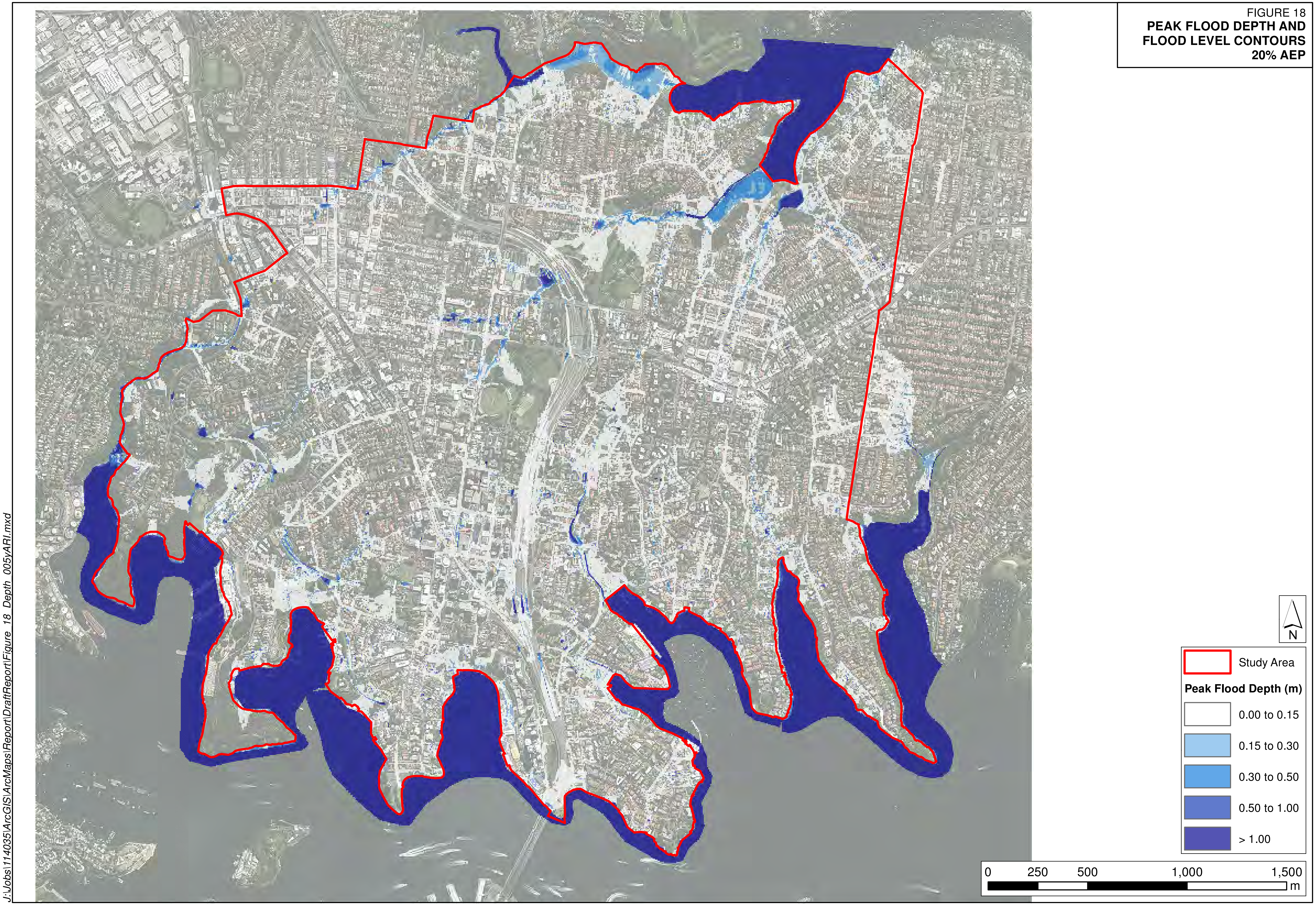

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FIGURE 19
PEAK FLOOD DEPTH AND FLOOD LEVEL CONTOURS 10% AEP

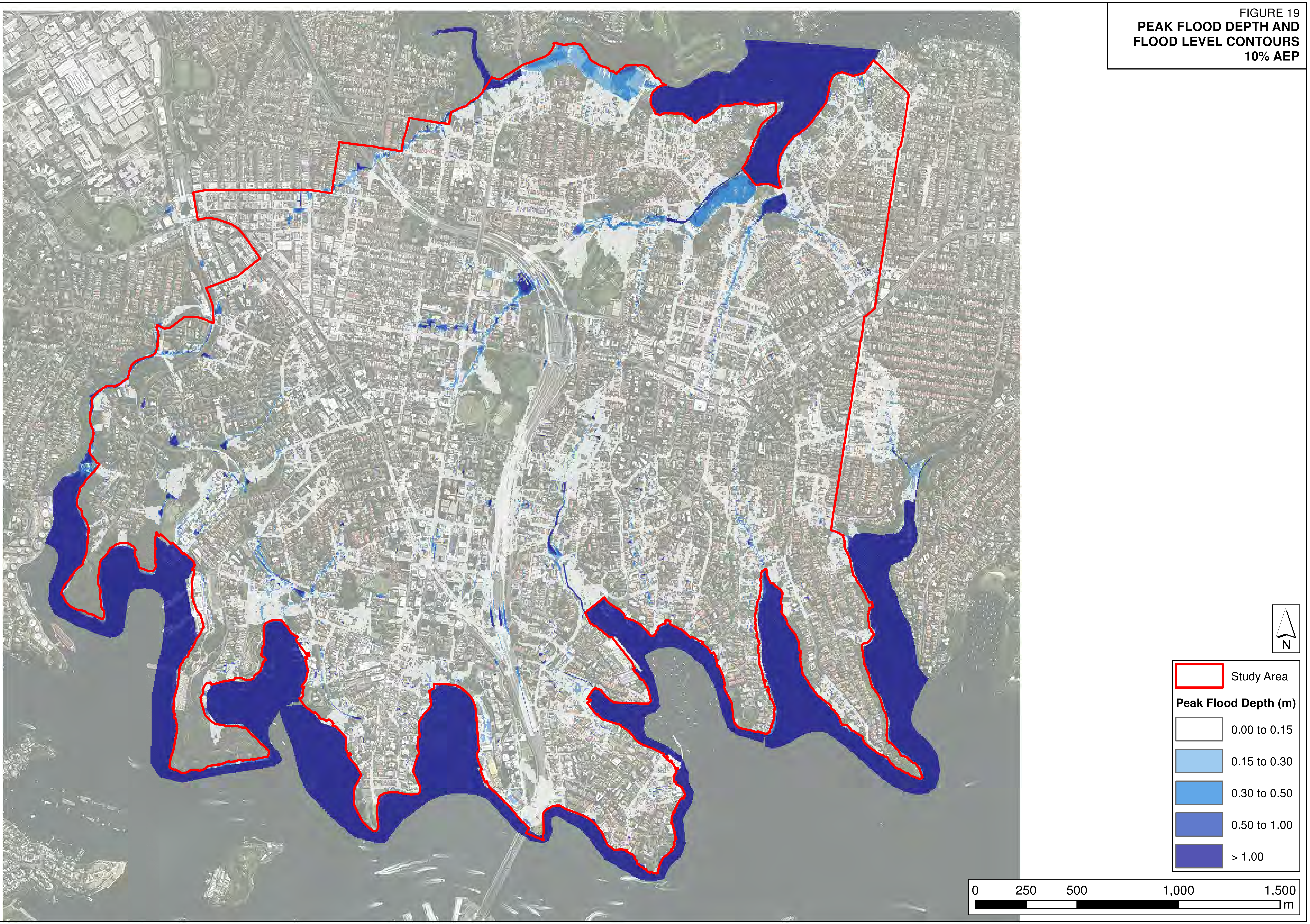

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FIGURE 20
PEAK FLOOD DEPTH AND FLOOD LEVEL CONTOURS 5% AEP

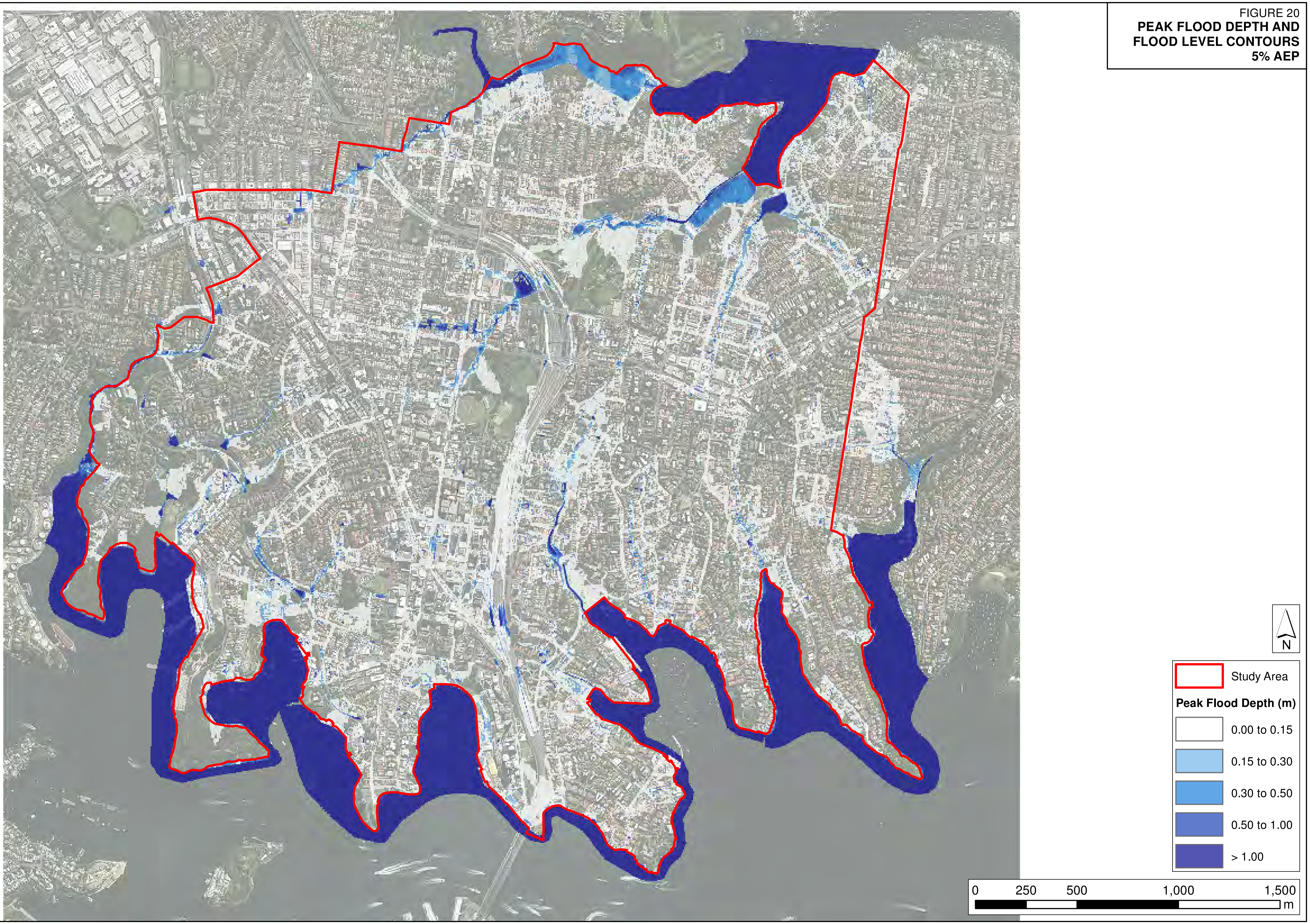

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FIGURE 21
PEAK FLOOD DEPTH AND FLOOD LEVEL CONTOURS 2% AEP

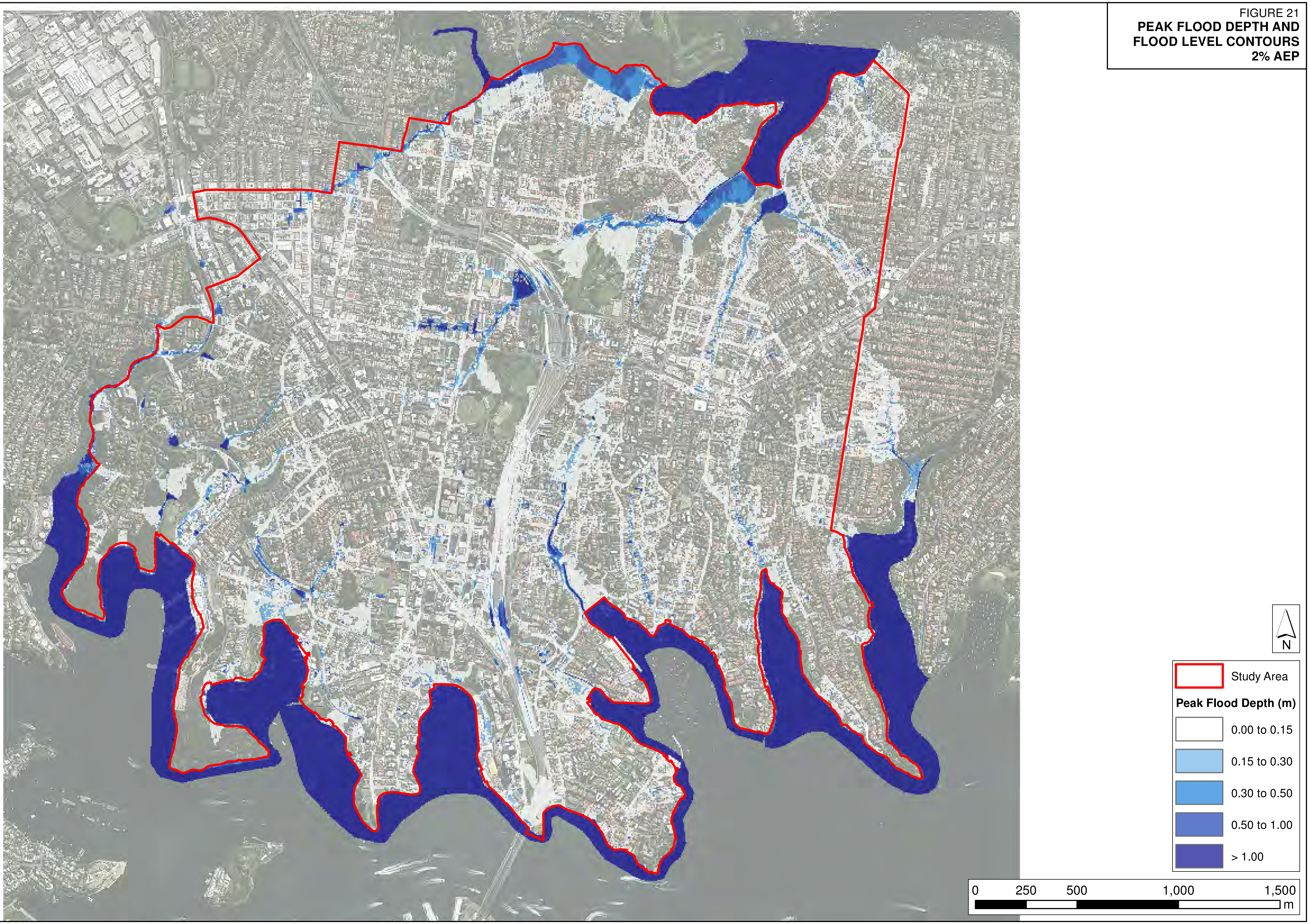

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FIGURE 22
PEAK FLOOD DEPTH AND FLOOD LEVEL CONTOURS 1% AEP

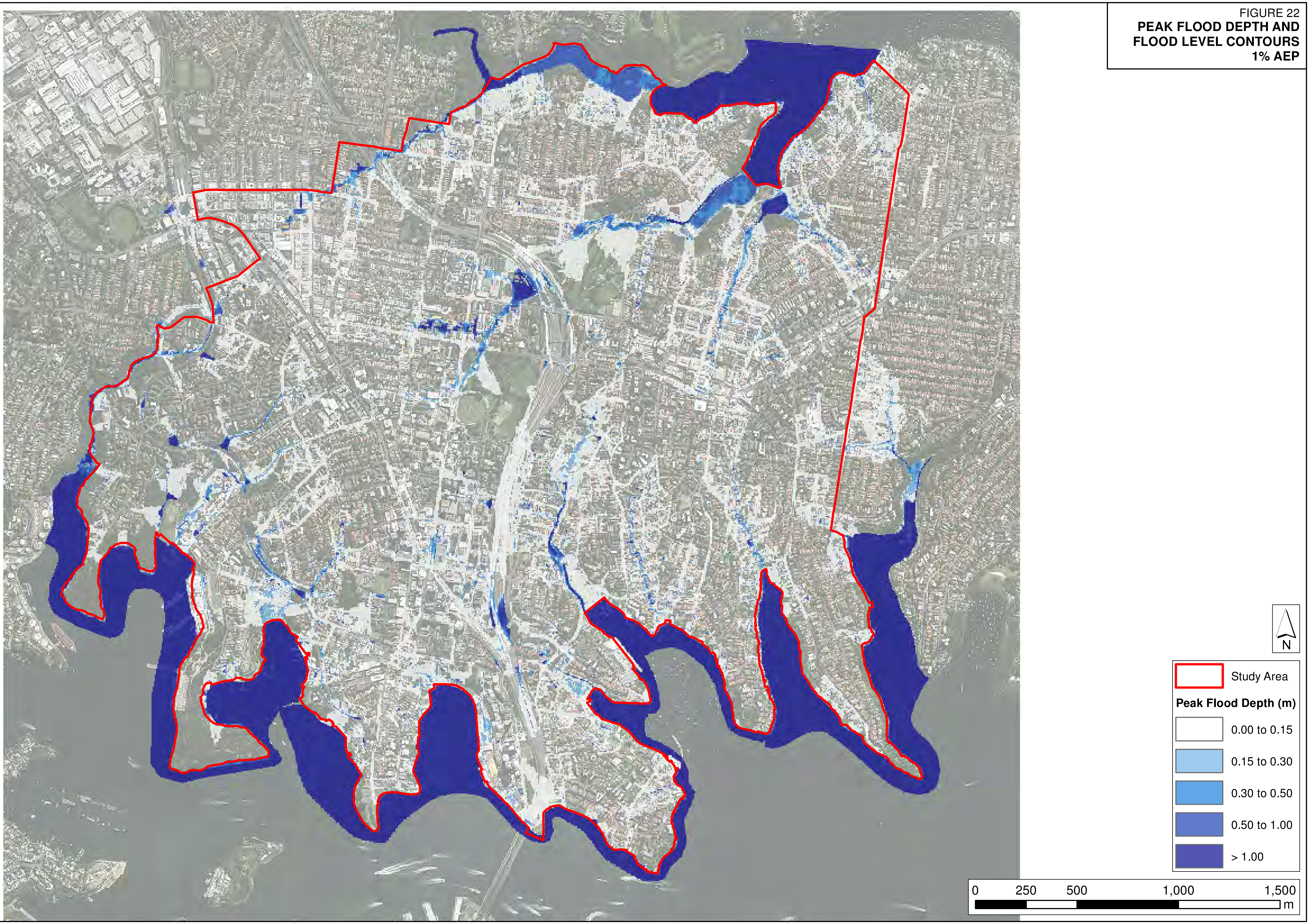

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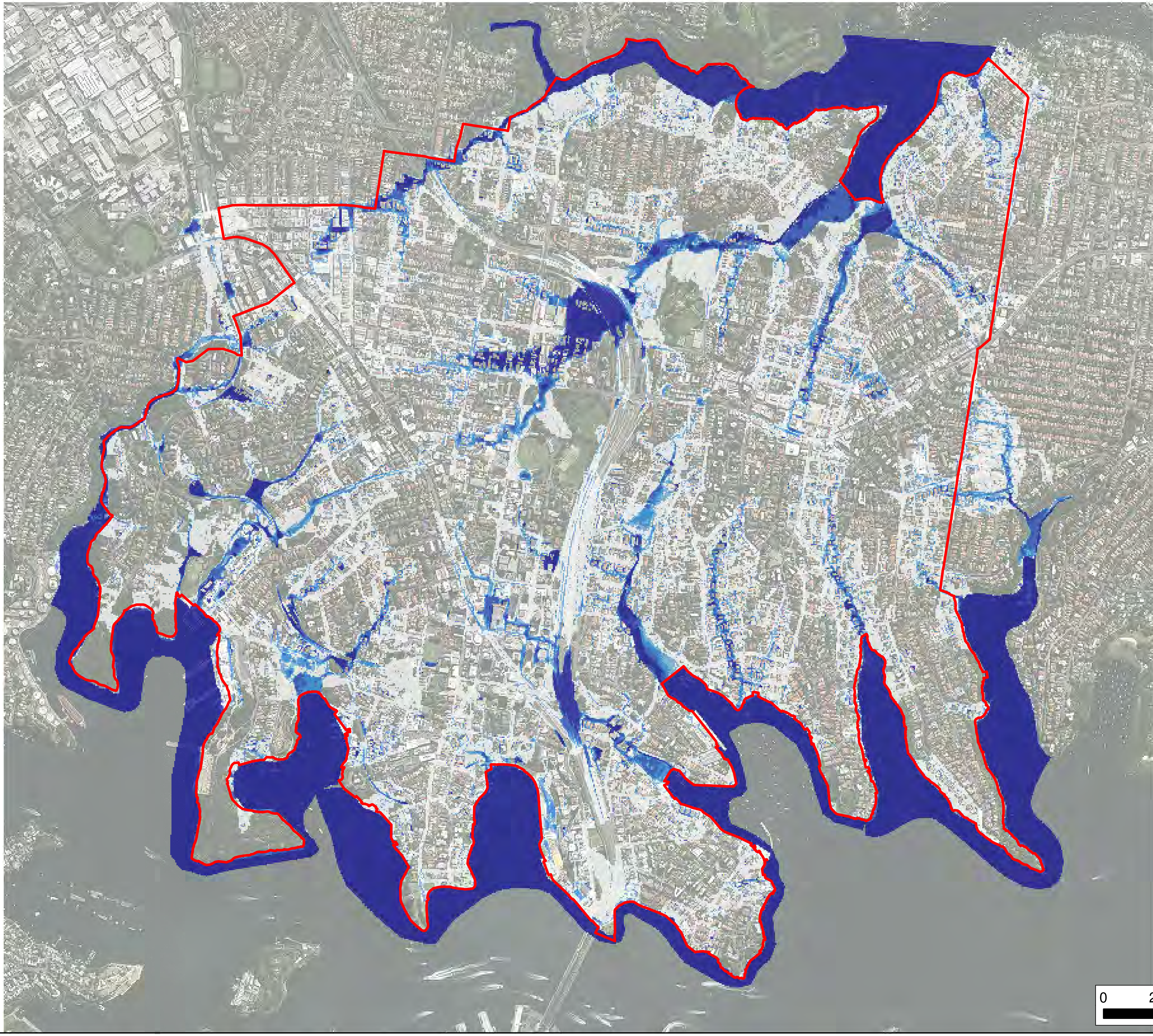
FIGURE 23
PEAK FLOOD DEPTH AND FLOOD LEVEL CONTOURS PMF



N

Study Area

Peak Flood Depth (m)

0.00 to 0.15
0.15 to 0.30
0.30 to 0.50
0.50 to 1.00
> 1.00

0 250 500 1,000 1,500 m

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FIGURE 24
PROVISIONAL HYDRAULIC HAZARD
20% AEP

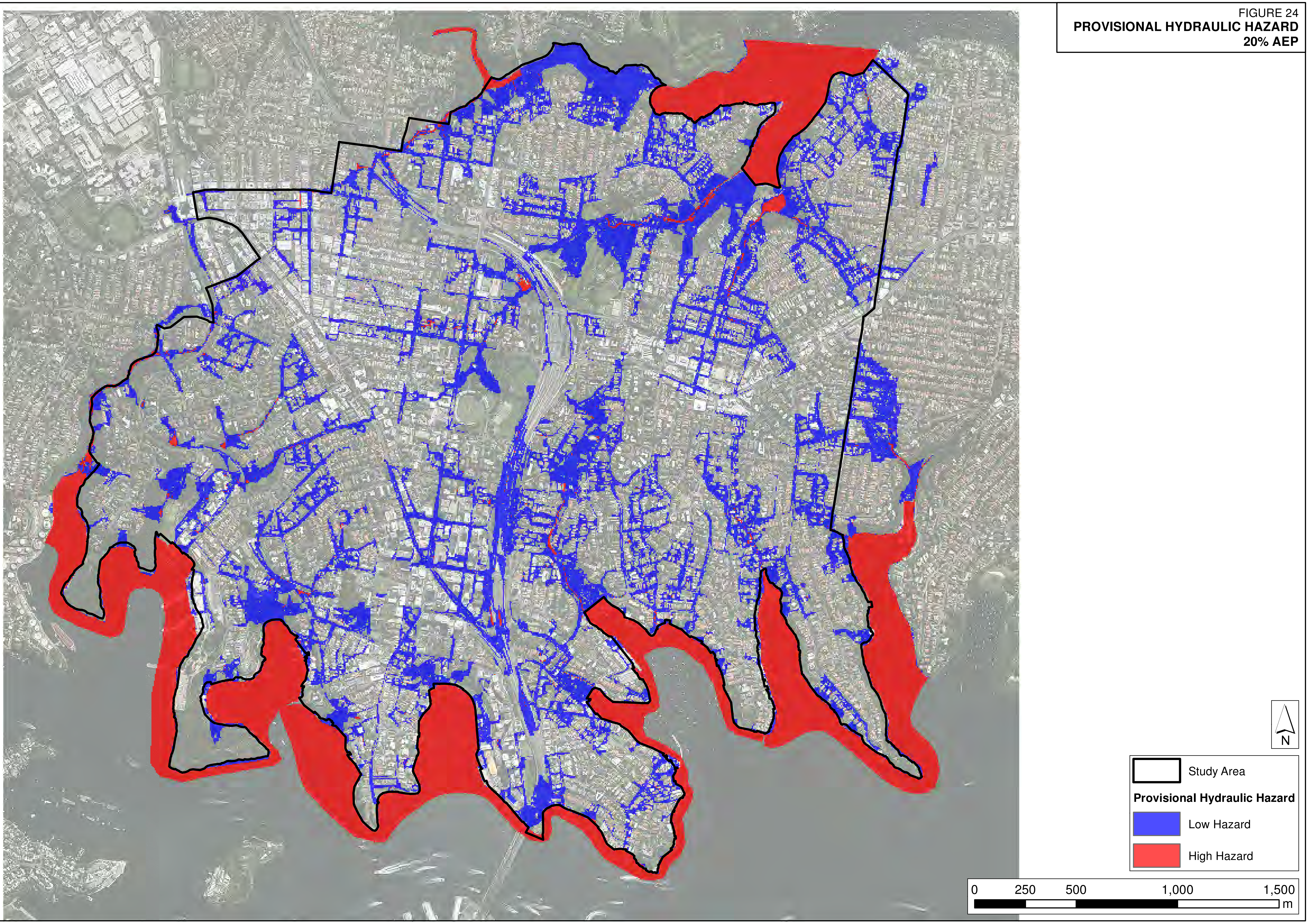

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FIGURE 25
PROVISIONAL HYDRAULIC HAZARD
5% AEP

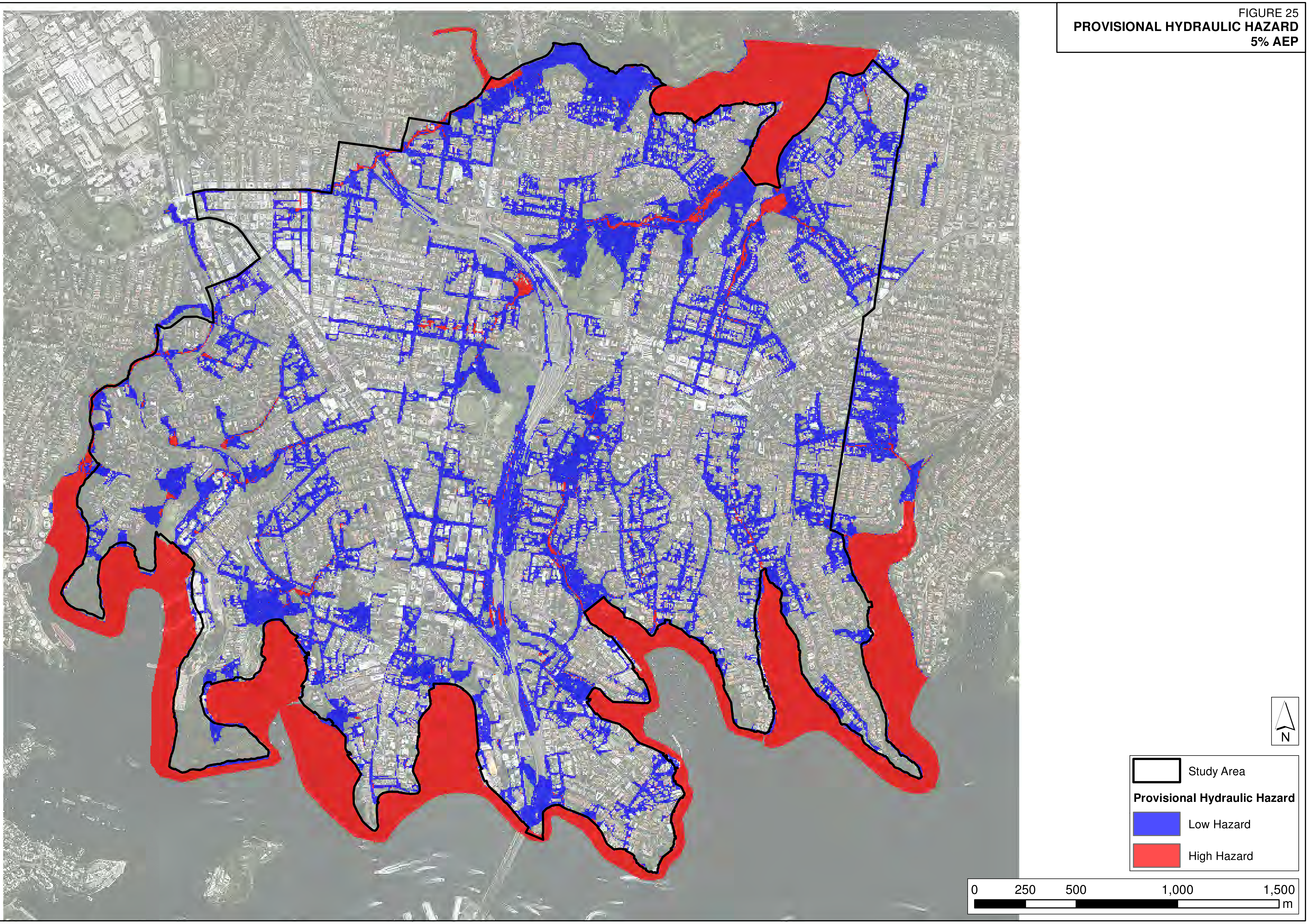

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FIGURE 26
PROVISIONAL HYDRAULIC HAZARD
1% AEP

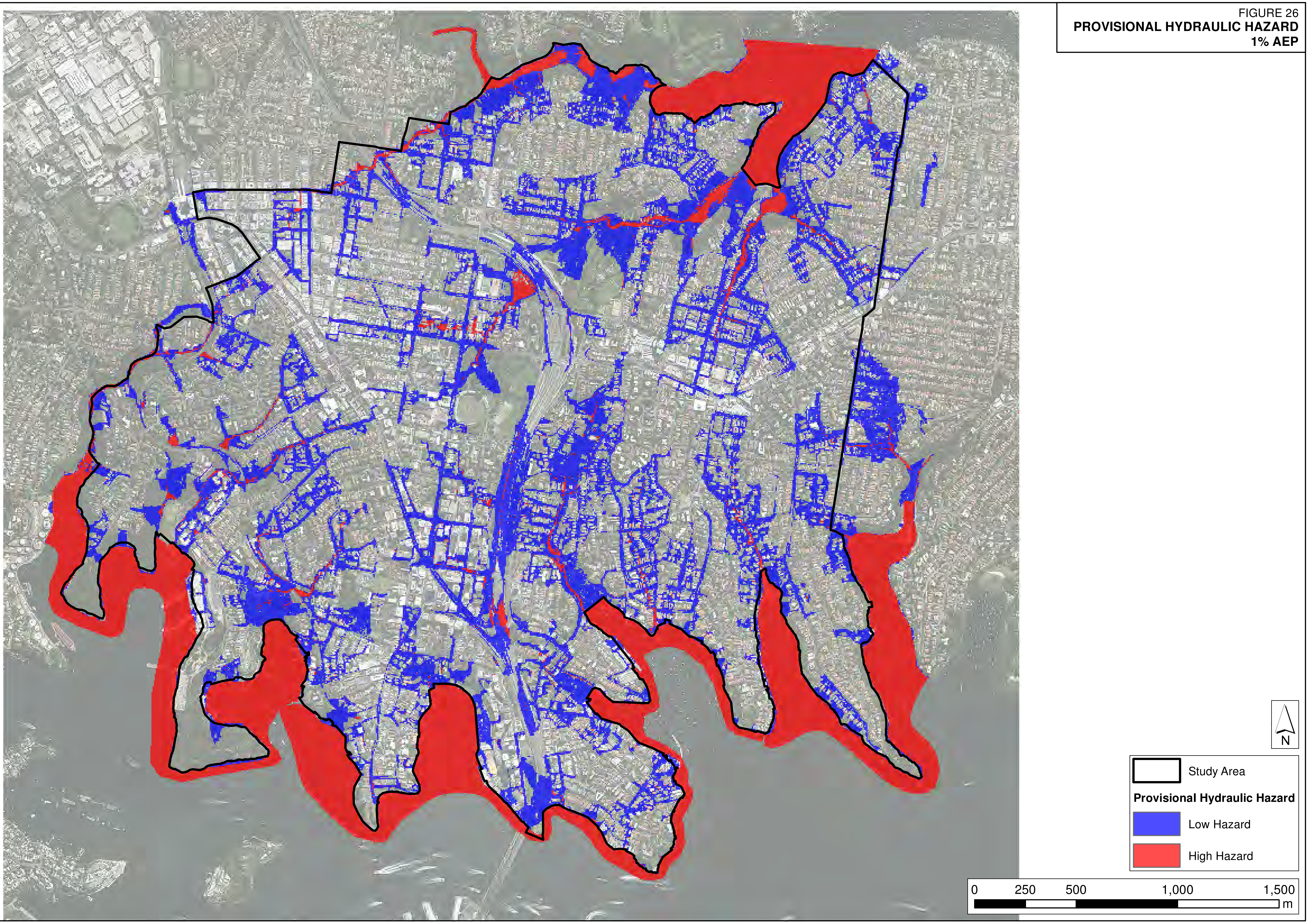

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FIGURE 27
PROVISIONAL HYDRAULIC HAZARD
PMF

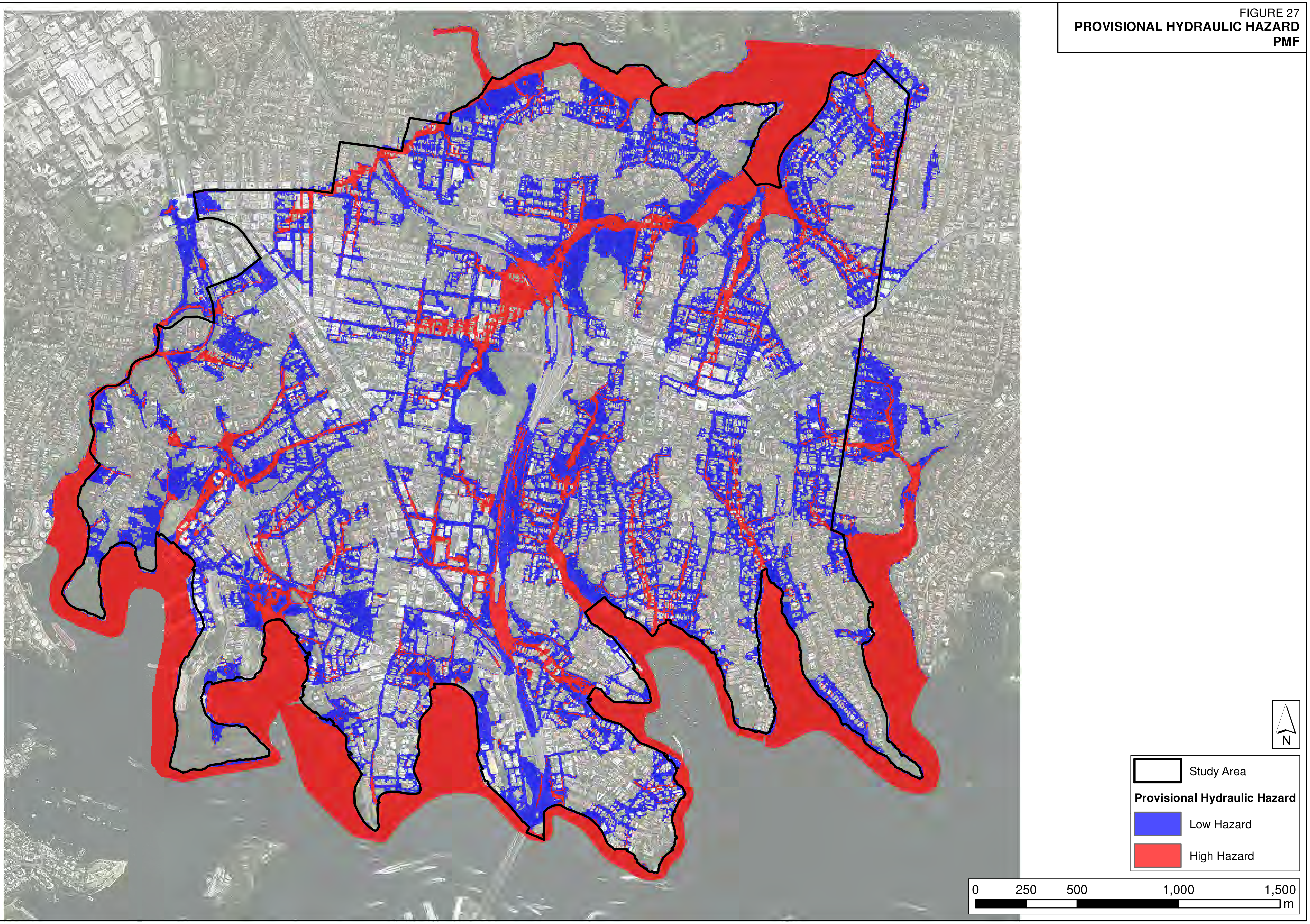

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FIGURE 28
PROVISIONAL HYDRAULIC CLASSIFICATION
20% AEP

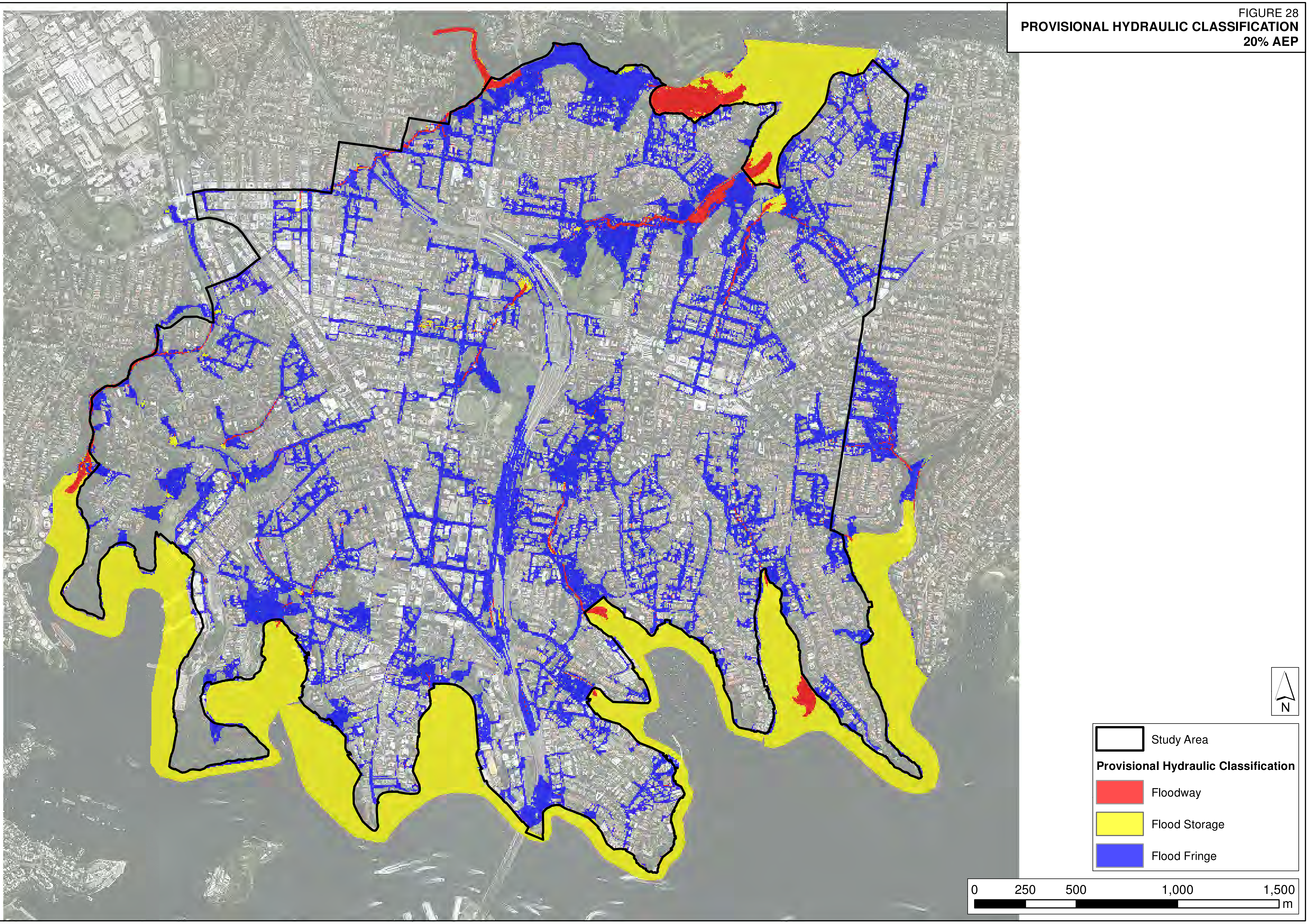

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FIGURE 29
PROVISIONAL HYDRAULIC CLASSIFICATION
5% AEP

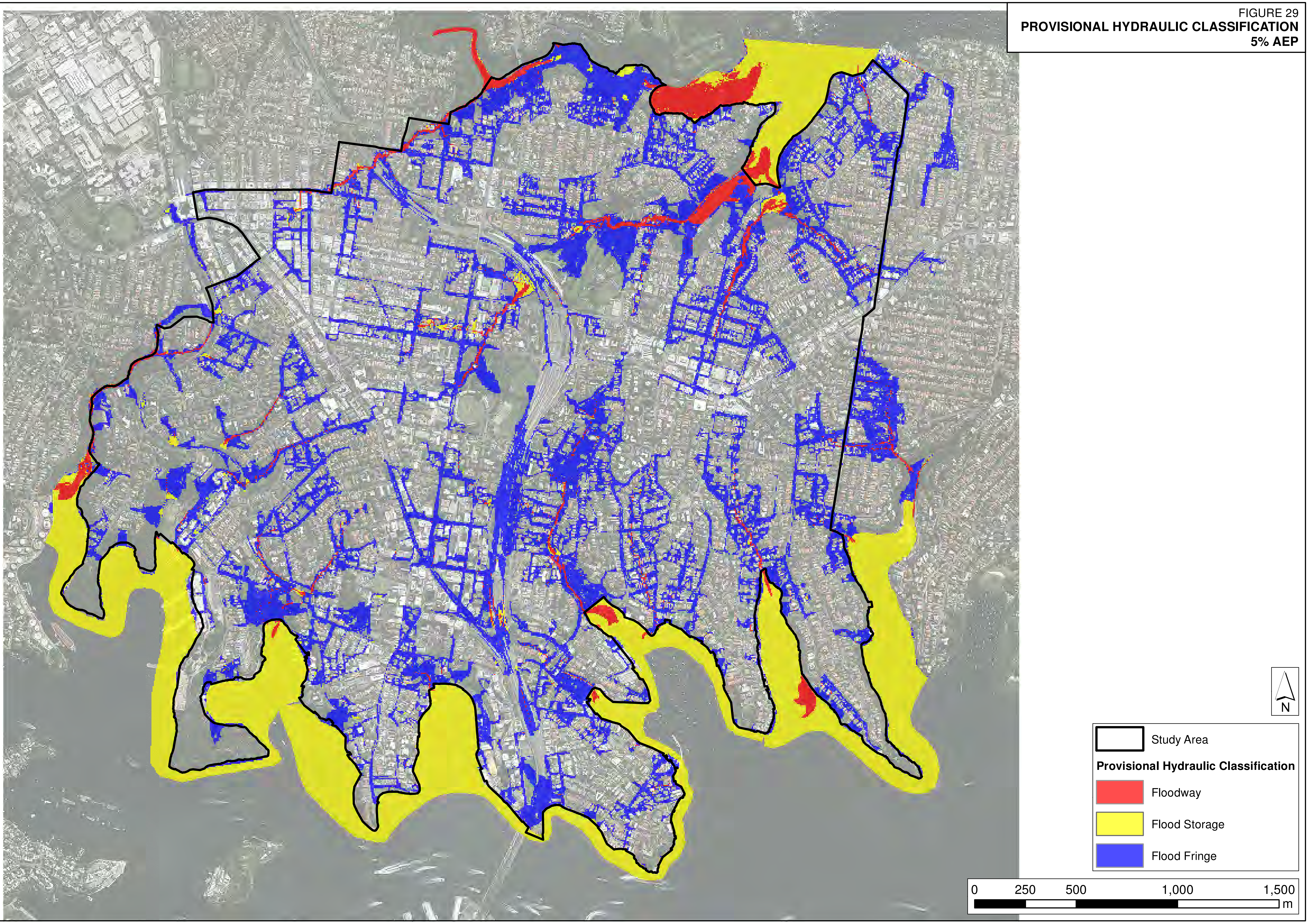

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FIGURE 30
PROVISIONAL HYDRAULIC CLASSIFICATION
1% AEP

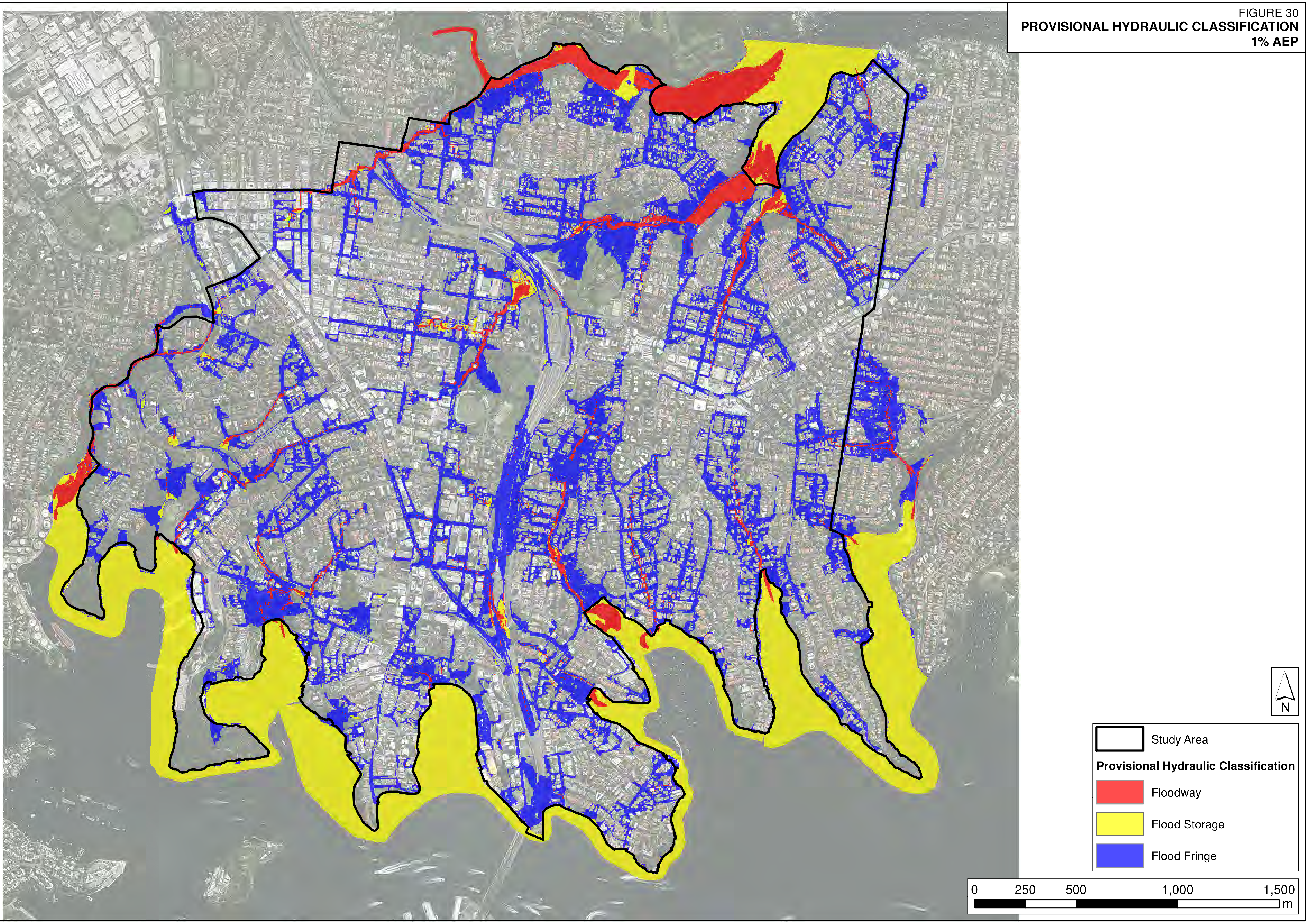

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FIGURE 31
PROVISIONAL HYDRAULIC CLASSIFICATION PMF

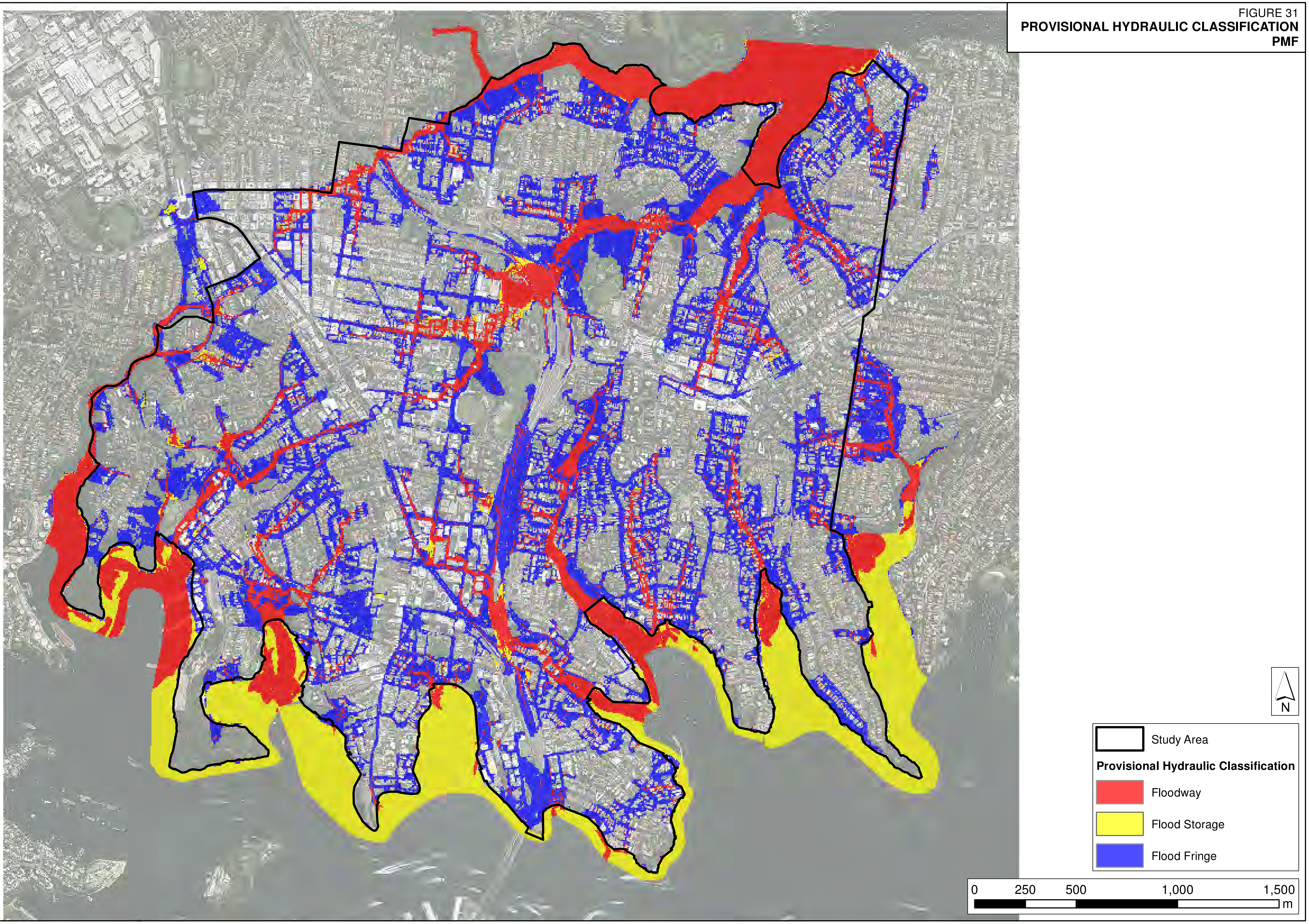

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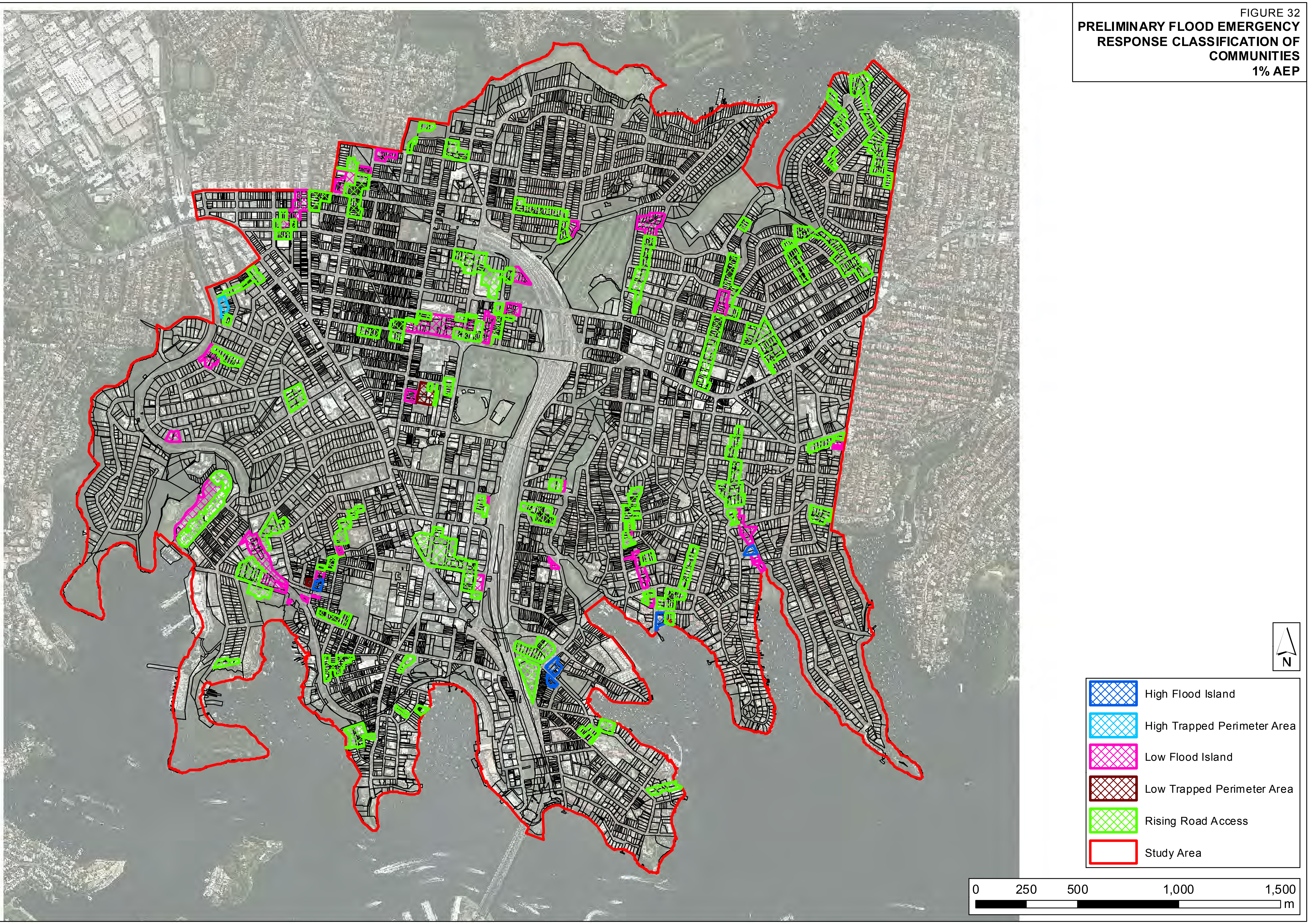
FIGURE 32
PRELIMINARY FLOOD EMERGENCY RESPONSE CLASSIFICATION OF COMMUNITIES
1% AEP



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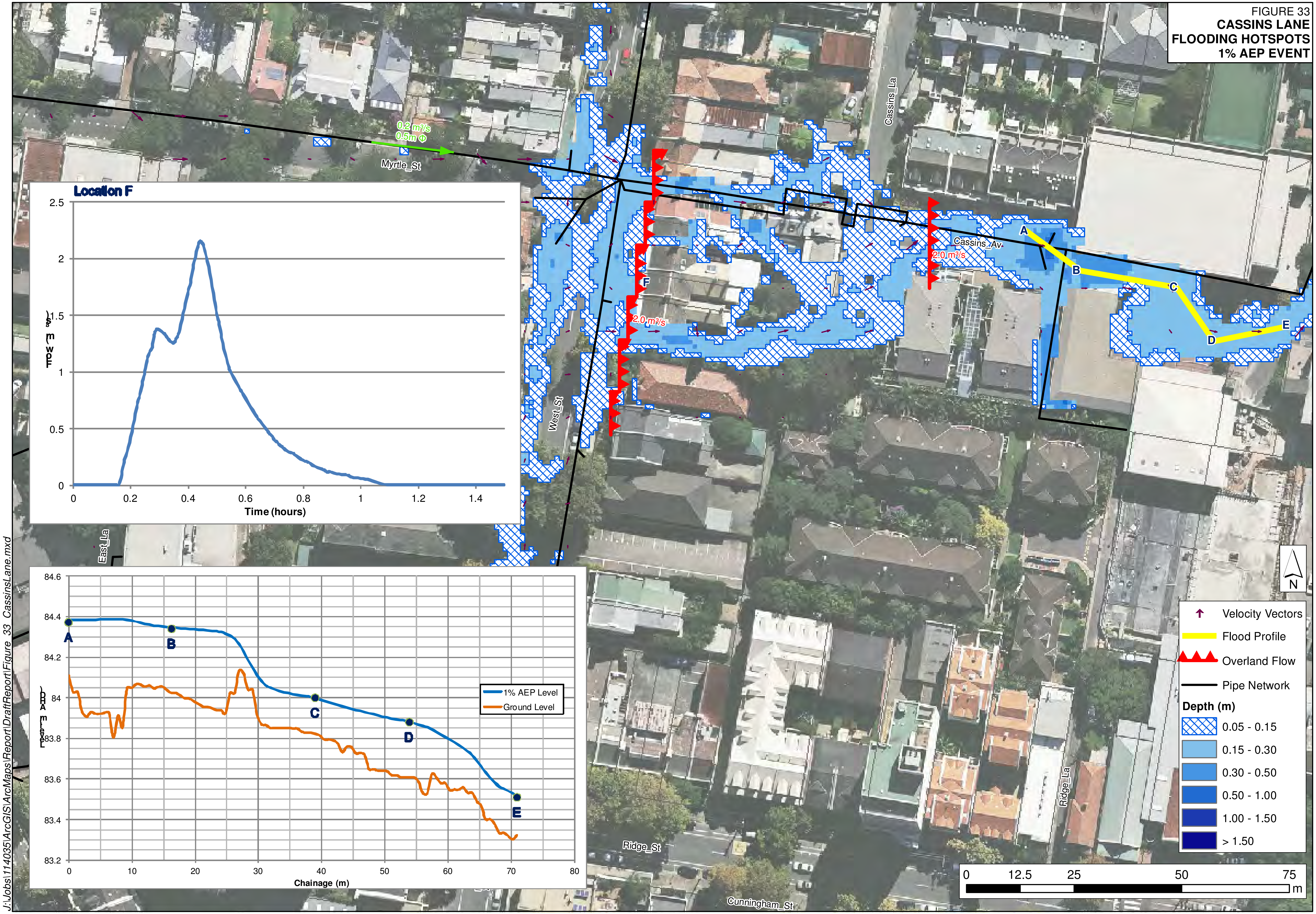

FIGURE 33
CASSINS LANE FLOODING HOTSPOTS 1% AEP EVENT

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FIGURE 34
COOPER LANE FLOODING HOTSPOTS 1% AEP EVENT

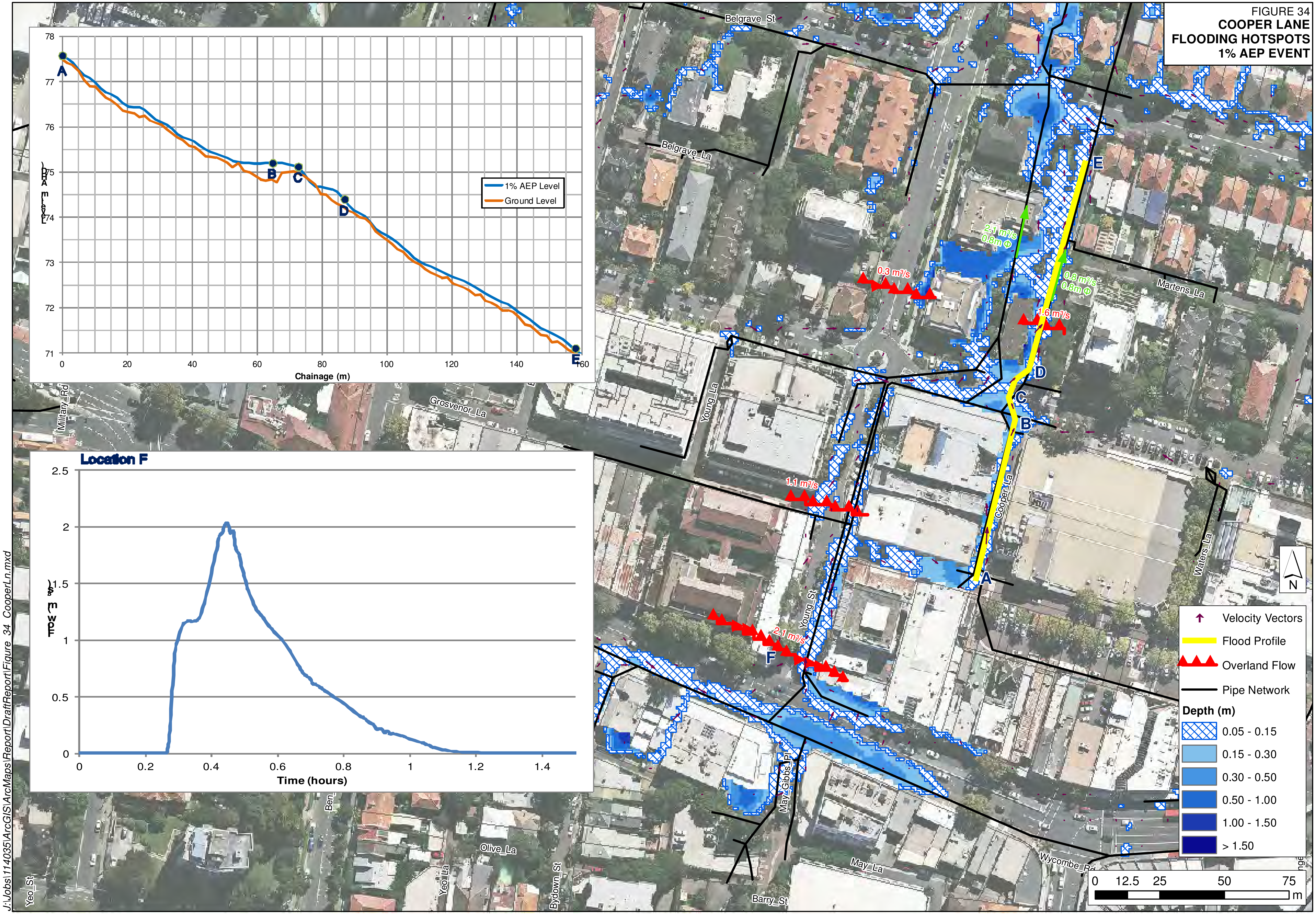

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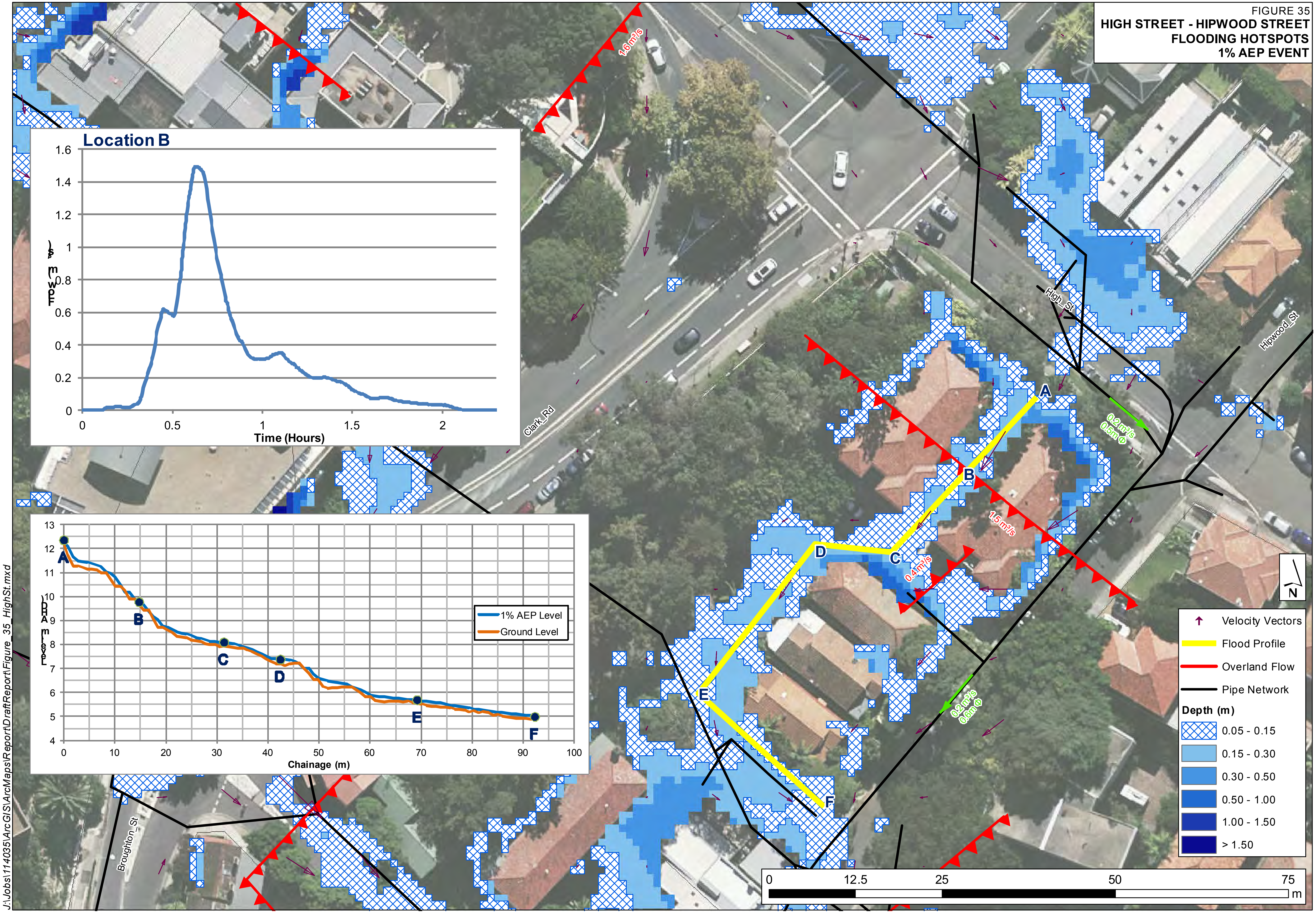
FIGURE 35
HIGH STREET - HIPWOOD STREET
FLOODING HOTSPOTS
1% AEP EVENT



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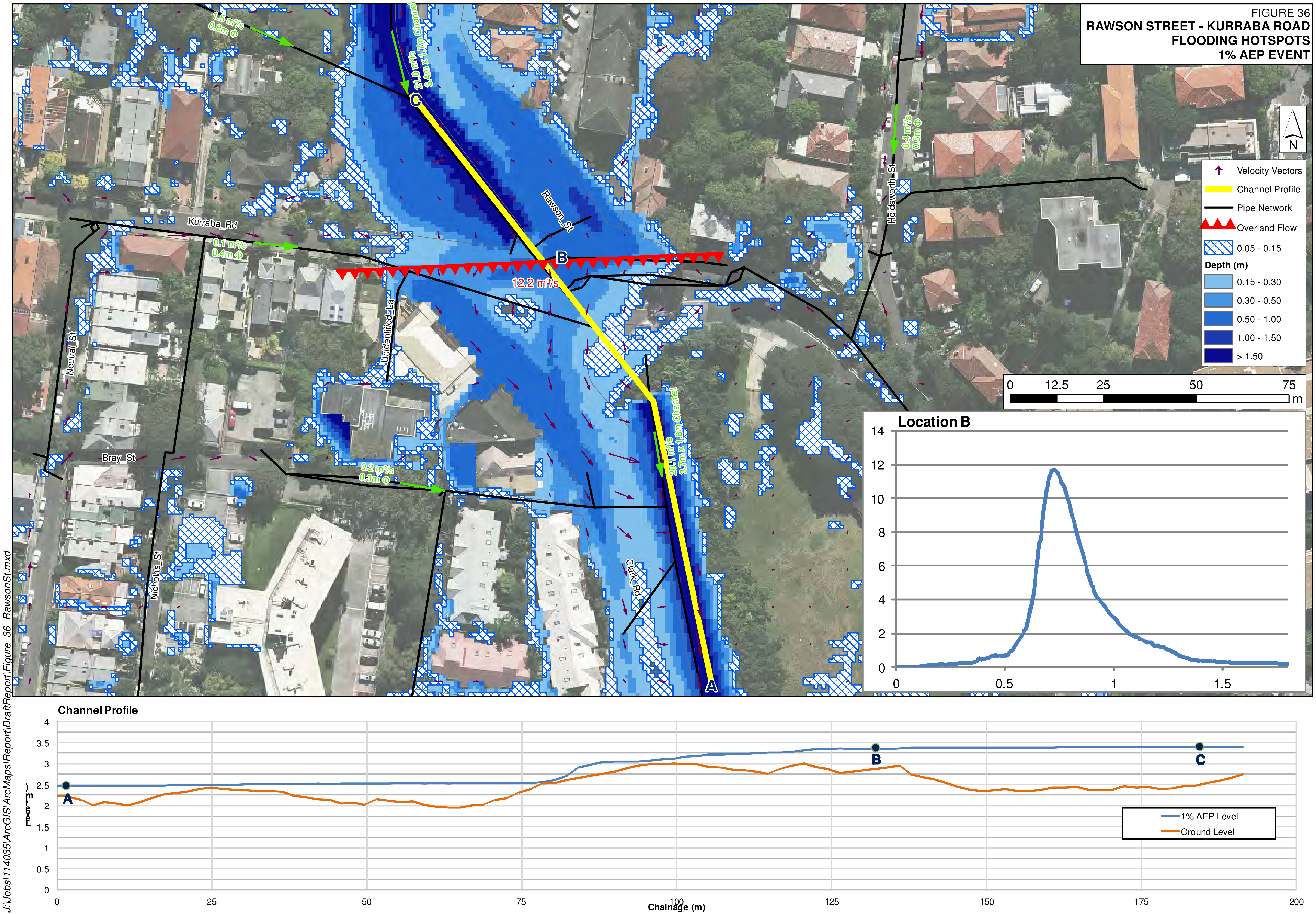

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Appendix A

APPENDIX A: GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.

	redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk

management process that forms the basis for physical works to modify the impacts of flooding.

floodplain — Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.

floodplain risk management options — The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.

floodplain risk management plan — A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.

flood plan (local) — A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.

flood planning area — The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the “flood liable land” concept in the 1986 Manual.

Flood Planning Levels (FPLs) — FPL’s are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the “standard flood event” in the 1986 manual.

flood proofing — A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.

flood prone land — Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.

flood readiness — Flood readiness is an ability to react within the effective warning time.

flood risk — Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below.

existing flood risk: the risk a community is exposed to as a result of its location on the floodplain.

future flood risk: the risk a community may be exposed to as a result of new development on the floodplain.

continuing flood risk: the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.

flood storage areas — Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood

	storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: ▪ the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or ▪ water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or ▪ major overland flow paths through developed areas outside of defined

drainage reserves; and/or

- the potential to affect a number of buildings along the major flow path.

mathematical/computer models

The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.

merit approach

The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State•s rivers and floodplains.

The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.

minor, moderate and major flooding

Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood:

minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded.

moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered.

major flooding: appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.

modification measures

Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.

peak discharge

The maximum discharge occurring during a flood event.

Probable Maximum Flood (PMF)

The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.

Probable Maximum Precipitation (PMP)

The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a

	particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to “water level”. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.