RUSHCUTTERS BAY

FLOOD STUDY

FINAL REPORT

APRIL 2016

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Project **RUSHCUTTERS BAY FLOOD STUDY**	Project Number **112022**
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FOREWORD

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

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

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

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

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

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

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

EXECUTIVE SUMMARY

The Rushcutters Bay catchment area within the City of Sydney local government area (LGA) includes the suburbs of Potts Point, Elizabeth Bay, Kings Cross, Darlinghurst, Paddington and Rushcutters Bay (Figure 1). The catchment is drained by a series of Sydney Water pipes, overland flow paths and open channels into Rushcutters Bay.

The key objective of this Flood Study is to develop a suitable hydraulic model that can be used as a basis for a Floodplain Risk Management Plan for the Study area, and to assist City of Sydney to undertake flood-related planning decisions for existing and future developments. Previous hydraulic modelling of the study area was limited in extent, and did not estimate flood levels in the City of Sydney portions of the catchment.

The primary objectives of the study are:

- to determine the flood behaviour including design flood levels and velocities over the full range of flooding up to and including the PMF from storm runoff in the study area;
- to provide a model that can establish the effects of future development on flood behaviour;
- to assess the sensitivity of flood behaviour to potential climate change effects such as increases in rainfall intensities and sea level rise; and
- to assess the hydraulic categories and undertake provisional hazard mapping.

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

- a summary of available flood related data;
- establishment and validation of the hydrologic and hydraulic models;
- sensitivity analysis of the model results to variation of input parameters;
- potential implications of climate change projection;
- the estimation of design flood behaviour for existing catchment conditions; and
- a flood damages assessment.

A glossary of flood related terms is provided in Appendix A.

FLOODING HISTORY

Significant catchment development occurred in the latter part of the 19th century. The 1861 census indicated a population of 2,700 which rose to 19,000 by 1890. In that time the number of houses increased from approximately 500 to 3,800. The current catchment population is of the order of 15,000 (Reference 1). Early references clearly identify parts of the lower catchment as low lying and swampy. There was also mention of surface and stormwater problems (flooding and water quality).

The effect of urbanisation on the quantity (and quality) of runoff from the catchment has not been assessed but would have been significant. As the catchment is already heavily urbanised any new developments are unlikely to produce further significant increases in peak flows.

There have been many instances of flooding in the past with 8-9 November 1984, 6 January 1989 and 26 January 1991 being some of the more significant storm events causing extensive flooding throughout the catchment. Section 3.4.1 provides details on a number of these past rainfall events responsible for the above mentioned floods.

OUTCOMES

The hydrological and hydraulic modelling undertaken for this study has defined flood behaviour for the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI design floods, as well as the Probable Maximum Flood (PMF). Due to the limited available data for calibration, a limited verification of the models to anecdotal historical information was undertaken. Sensitivity analyses were undertaken to assess the influences of modelling assumptions on key outputs, and the potential impacts of future climate change. Provisional hazard mapping has been completed for the 10 year, 20 year and 100 year and PMF events. Hydraulic category mapping has been completed for the 100 year ARI event.

The design flood modelling indicates that significant flood depths may occur in a number of locations such as Sims Street, Taylor Street, Sturt Street, Oxford Street, Boundary Street, Barcom Avenue, McLachlan Avenue and Womerah Avenue which is supported by a limited calibration and anecdotal reports of flooding.

1. INTRODUCTION

1.1. Background

The Rushcutters Bay catchment within the City of Sydney local government area (LGA) includes the suburbs of Potts Point, Elizabeth Bay, Kings Cross, Darlinghurst, Paddington and Rushcutters Bay (Figure 1). The catchment is drained by a series of Sydney Water pipes, overland flow paths and open channels into Rushcutters Bay.

The present Flood Study has been commissioned by City of Sydney (CoS), with assistance from the NSW Office of Environment and Heritage (OEH). This study considers flooding in the Rushcutters Bay catchment within the City of Sydney's LGA from local storm runoff and continued development means it is important that appropriate tools and information to assess flood risks are available to City of Sydney for planning future development in the area.

1.2. Objectives

The key objective of this Flood Study is to develop a suitable hydraulic model that can be used as a basis for a Floodplain Risk Management Plan for the Study area (Figure 2), and to assist City of Sydney to undertake flood-related planning decisions for existing and future developments. Previous hydraulic modelling of the study area was limited in extent, and did not estimate flood levels in the City of Sydney portions of the catchment.

The primary objectives of the study are:

- to determine the flood behaviour including design flood levels and velocities over the full range of flooding up to and including the PMF;
- to provide a model that can establish the effects of flood behaviour of future development;
- to assess the sensitivity of flood behaviour to potential climate change effects such as increases in rainfall intensities and sea level rise; and
- to assess the hydraulic categories and undertake provisional hazard mapping.

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

- a summary of available flood related data;
- establishment and validation of the hydrologic and hydraulic models;
- sensitivity analysis of the model results to variation of input parameters;
- potential implications of climate change projection;
- the estimation of design flood behaviour for existing catchment conditions; and
- a flood damages assessment.

A glossary of flood related terms is provided in Appendix A.

2. BACKGROUND

2.1. Catchment Description

The Rushcutters Bay catchment is located in the suburbs of Potts Point, Elizabeth Bay, Kings Cross, Darlinghurst, Paddington and Rushcutters Bay. The region lies within the City of Sydney Local Government Area (LGA) and has been extensively developed for urban usage.

The land usage within the study area is predominantly urban residential development, comprising a mixture of pre-1900 terrace buildings (mostly south of William Street) and new high-rise apartment buildings, including several medium- and high-density developments (mostly north of William Street). The non-residential development in the catchment includes several schools, parks (including the Rushcutters Bay Park and Weigall Sportsgrounds), churches and community buildings including St Vincents Hospital. There are no major industrial developments, and commercial developments are primarily concentrated in the upper catchment areas around Oxford Street and Kings Cross. There are some larger commercial sites such as car dealerships/workshops in the lower part of the catchment near Weigall Sportsgrounds.

The catchment covers an area of approximately 92 hectares draining to Sydney Water's major trunk drainage systems to route flows from the upper regions of the catchment. The area drains into Sydney Harbour at Rushcutters Bay via the Sydney Water open channel, which generally runs in a north-westerly direction between the Weigall and White City sports complexes. The channel floodplain is largely contained within a series of parks and open spaces. The trunk drainage system is linked to Council's local drainage system consisting of covered channels, in-ground pipes, culverts and kerb inlet pits. Further information on the drainage system is presented in Section 3.3.

The topography of the catchment is steep with the greatest relief occurring at the top of the catchment along Oxford Street at elevations of 65 mAHD which slopes north-east at grades of approximately 5% to 10%. The downstream end of the study area is also the flattest part of the catchment, comprising reclaimed lands within Rushcutters Bay Park, which has a relatively gentle ground gradient of 1%.

2.1.1. Flooding History

Significant catchment development occurred in the latter part of the 19th century, alongside a major increase in the broader Sydney population between 1860 and 1890. The current catchment population is of the order of 15,000 (Reference 1). Early references clearly identify parts of the lower catchment as low lying and swampy. There was also mention of surface and stormwater problems (flooding and water quality).

The effect of urbanisation on the quantity (and quality) of runoff from the catchment has not been assessed but would have been significant. As the catchment is already heavily urbanised any new developments are unlikely to produce further significant increases in peak flows.

There have been many instances of flooding in the past with 8-9 November 1984, 6 January 1989 and 26 January 1991 being some of the more significant storm events causing extensive flooding throughout the catchment. Section 3.4.1 provides details on the rainfall events responsible for the above mentioned floods.

2.2. Previous Studies

2.2.1. Rushcutters Bay SWC No. 84 Catchment Management Study

The Rushcutters Bay SWC No. 84 Catchment Management Study, 1991 (Reference 1) was undertaken as an overall investigation of stormwater drainage and water pollution issues in the catchment. The full length of the open channel and piped system controlled by Sydney Water, Woollahra and the City of Sydney Councils was examined.

A large part of the report covered water quality issues not relevant to this Flood Study. However the study included a comprehensive questionnaire survey (8,900 sent out), the results of which have been reproduced in this study (Section 3.8) as they are still relevant.

An ILSAX hydrological model and HEC-2 hydraulic model were developed and based on the results a cost-benefit analysis was undertaken to assess measures to reduce flooding. The main recommendations from the report (relating to stormwater drainage) were to provide new and duplicate pipe systems. The study found many of the pipes in the catchment had a 1 in 1 year ARI capacity.

2.2.2. Rushcutters Bay Catchment Flood Study

This report (Reference 2) was prepared for Woollahra Municipal Council by WMAwater and examines flooding issues for the portion of the Rushcutters Bay catchment within the Woollahra LGA.

Flood discharges and levels were determined for the Rushcutters Bay catchment using the DRAINS and TUFLOW computer models. At the downstream end of the model, a tailwater level of 1.0 mAHD was adopted after consideration of historic tidal records in Sydney Harbour at Fort Denison.

The study indicates that floodwaters inundate Trumper Park and the White City tennis complex in 5 year ARI and greater events. The yards of many private properties adjoining the open channel would also be inundated.

3. AVAILABLE DATA

3.1. Topographic Survey

Airborne Light Detection and Ranging (LiDAR) survey of the catchment and its immediate surroundings was provided for the study by City of Sydney and is shown on Figure 3. The data was a combination of data collected in 2007 and 2008 with a 1.3m average point separation. For hard flat surfaces these data typically have accuracy in the order of:

- ±0.15m in the vertical direction (to one standard deviation); and
- ±0.25m in the horizontal direction (to one standard deviation).

When interpreting the above, it should be noted that the accuracy of the ground definition can be adversely affected by the nature and density of vegetation and/or the presence of steeply varying terrain.

3.2. Open Channel

An open channel system within the Rushcutters Bay catchment is located downstream of Glenmore Road. The system is owned and administered by Sydney Water. In the past parts of the drainage system acted as a combined stormwater and sewerage system. However Sydney Water has undertaken works to largely separate these systems.

The open channel is at the very downstream of the Rushcutters Bay study area and design flow conditions within the channel have been established in Reference 2. Additional details of the channel may be found in Reference 2.

3.3. Pit and Pipe Data

The catchment is serviced by a major/minor drainage system. Property drainage is directed to the Kerb and Gutter system where it is then able to enter the Council owned minor street drainage network. At the bottom of the catchment, flow is routed into the Sydney Water Corporation (SWC) owned and maintained trunk drainage system that crosses under New South Head Road and drains to Rushcutters Bay.

When the capacity of the drainage system is exceeded, flow occurs along road reserves and other overland flow paths, with the potential for velocities and/or flow depths combining to generate high hazard flood conditions in some locations. For the catchment branch south of William Street, the main drainage paths in the road network include Victoria Street, Barcom Avenue, West Street, Womerah Avenue, McLachlan Avenue and Neild Avenue. North of William Street, the main flow paths include Bayswater Road, Roslyn Street/Gardens, and Waratah Street.

City of Sydney provided an asset database including dimensions and invert elevations for the majority of stormwater conduits within the study area. The following datasets were used to define stormwater infrastructure in modelling for this study:

- pipe asset database “WMA_DataSupply.gdb: Pipes_Survey” (received 16/03/2012);

- pit asset database “WMA_DataSupply.gdb: Pits_Survey” (received 16/03/2012);
- pit and pipe data from Reference 2.

A summary of pit and pipe survey data used within the study is provided in Table 1.

Table 1: Modelled Pipe and Pipe Network

Pit Type	Number
Outlet	4
Kerb or Grate Inlets	357
Junctions	379

Pipe Diameter (mm)	Number	Total Length (m)
< 450	552	8260
450 - 750	122	2580
750 - 1000	29	900
1000 - 2400	52	1730
> 2400	13	580

3.4. Rainfall

3.4.1. Historical Rainfall

Table 2 presents a summary of the official rainfall gauges (provided by the Bureau of Meteorology located close to or within the catchment. These gauges are operated either by Sydney Water (SW) or the Bureau of Meteorology (BoM). There may also be other private gauges in the area (bowling clubs, schools) but data from these has not been collected as there is no public record of their existence. Of the 45 gauges listed in Table 2 over 58% (26) have now closed. The gauge with the longest record is Observatory Hill, operating from 1858 to the present. The closest pluviometer gauge to the study area catchment is Paddington, which has been in operation from 1968. Locations of rainfall stations are shown on Figure 4.

Table 2: Rainfall Stations with a 6km Radius of Paddington Gauge

Station No.	Owner	Station	Elevation (mAHD)	Distance from Paddington (km)	Date Opened	Date Closed	Type
66139	BOM	Paddington	5	0.0	Jan-1968	Jan-1976	Daily
566041	SW	Crown Street Reservoir	40	0.8	Feb-1882	Dec-1960	Daily
566032	SW	Paddington (Composite Site)	45	1.0	Apr-1961		Continuous
566032	SW	Paddington (Composite Site)	45	1.0	Apr-1961		Daily
566009	SW	Rushcutters Bay Tennis Club	-	1.3	May-1998		Continuous
566042	SW	Sydney H.O. Pitt Street	15	1.5	Aug-1949	Feb-1965	Continuous
66015	BOM	Crown Street Reservoir		1.5	Feb-1882	Dec-1960	Daily
66006	BOM	Sydney Botanic Gardens	15	1.9	Jan-1885		Daily
66160	BOM	Centennial Park	38	2.1	Jun-1900		Daily
566011	SW	Victoria Park @ Camperdown	-	2.4	May-1998		Continuous
66097	BOM	Randwick Bunnerong Road		2.4	Jan-1904	Jan-1924	Daily
66062	BOM	Sydney (Observatory Hill)	39	2.7	??		Continuous

Station No.	Owner	Station	Elevation (mAHD)	Distance from Paddington (km)	Date Opened	Date Closed	Type
66062	BOM	Sydney (Observatory Hill)	39	2.7	Jul-1858	Aug-1990	Daily
66033	BOM	Alexandria (Henderson Road)	15	2.8	May-1962	Dec-1963	Daily
66033	BOM	Alexandria (Henderson Road)	15	2.8	Apr-1999	Mar-2002	Daily
66073	BOM	Randwick Racecourse	25	2.9	Jan-1937		Daily
566110	SW	Erskineville Bowling Club	10	3.4	Jun-1993	Feb-2001	Continuous
566010	SW	Cranbrook School @ Bellevue	-	3.4	May-1998		Continuous
566015	SW	Alexandria	5	3.5	May-1904	Aug-1989	Daily
66066	BOM	Waverley Shire Council		3.6	Sep-1932	Dec-1964	Daily
66149	BOM	Glebe Point Syd. Water Supply	15	3.6	Jun-1907	Dec-1914	Daily
566099	SW	Randwick Racecourse	30	3.7	Nov-1991		Continuous
66052	BOM	Randwick Bowling Club	75	3.7	Jan_1888		Daily
566141	SW	SP0057 Cremorne Point	-	4.0			Continuous
66021	BOM	Erskineville	6	4.0	May-1904	Dec-1973	Daily
	SW	Gladstone Park Bowling Club	-	4.1	Jan-1901		Continuous
566114	SW	Waverley Bowling Club	-	4.1	Jan-1995		Continuous
566043	SW	Randwick (Army)	30	4.3	Dec-1956	Sep-1970	Continuous
566077	SW	Bondi (Dickson Park)	60	4.4	Dec-1989	Feb-2001	Continuous
566065	SW	Annandale	20	4.5	Dec-1988		Continuous
66098	BOM	Royal Sydney Golf Club	8	4.5	Mar-1928		Daily
66005	BOM	Bondi Bowling Club	15	4.6	Jul-1939	Dec-1982	Daily
66178	BOM	Birchgrove School	10	4.8	May-1904	Dec-1910	Daily
66075	BOM	Waverton Bowling Club	21	5.1	Dec-1955	Jan-2001	Daily
66187	BOM	Tamarama (Carlisle Street)	30	5.1	Jul-1991	Mar-1999	Daily
66179	BOM	Bronte Surf Club	15	5.2	Jan-1918	Jan-1922	Daily
566130	SW	Mosman (Reid Park)	-	5.3	Jan-1998	Jun-1998	Continuous
566030	SW	North Sydney Bowling Club	80	5.5	Apr-1950	Sep-1995	Daily
66007	BOM	Botany No.1 Dam	6	5.5	Jan-1870	Jan-1978	Daily
66067	BOM	Wollstonecraft	53	5.8	Jan-1915	Jan-1975	Daily
66061	BOM	Sydney North Bowling Club	75	5.8	Apr-1950	Dec-1974	Daily
566027	SW	Mosman (Bradleys Head)	85	5.8	Jun-1904		Continuous
566027	SW	Mosman (Bradleys Head)	85	5.8	Jun-1904		Daily
566006	BOM	Bondi (Sydney Water)	10	5.9	Jun-1997		Operational
66175	BOM	Schnapper Island	5	5.9	Mar-1932	Dec-1939	Daily

BOM = Bureau of Meteorology
SW = Sydney Water

3.5. Analysis of Daily Read Data

Table 3: Daily Rainfall greater than 150 mm

Centennial Park		
Records since 1900		
Rank	Date	Rainfall (mm)
1	28/03/1942	302
2	06/08/1986	236
3	03/02/1990	222
4	12/08/1975	221
5	13/10/1975	205
6	31/01/1938	201
7	30/04/1988	193
8	10/02/1956	192
9	23/01/1933	189
10	09/02/1958	185
11	11/10/1975	184
12	07/07/1931	177
13	09/04/1945	177
14	07/08/1998	162
15	17/05/1943	159
16	04/02/1990	156
17	10/07/1957	155
18	14/11/1969	155
19	01/05/1955	154
20	09/02/1992	151
21	28/07/2008	150
22	13/01/2011	150

Randwick Bowling Club (66052)		
Records since Jan 1888		
Rank	Date	Rainfall (mm)
1	06/08/1986	297
2	29/10/1959	265
3	28/03/1942	243
4	03/02/1990	225
5	10/02/1956	213
6	31/01/1938/	213
7	11/03/1975	201
8	17/01/1988	178
9	12/10/1902	178
10	28/04/1966	177
11	04/02/1990	175
12	19/11/1900	164
13	09/02/1992	162
14	28/07/1908	161
15	09/02/1958	158
16	29/05/1906	155
17	30/08/1963	152
18	27/04/1901	150

Randwick Racecourse (66073)		
Records since Jan 1937		
Rank	Date	Rainfall (mm)
1	10/02/1992	294
2	20/11/1961	270
3	30/10/1959	267
4	06/08/1986	263
5	11/03/1975	261
6	14/05/1962	258
7	10/02/1958	256
8	05/02/1990	248
9	03/02/1990	244
10	09/11/1984	240
11	20/03/1978	237
12	06/11/1984	223
13	28/03/1942	213
14	31/01/1938	211
15	10/02/1956	195
16	30/04/1988	175
17	30/08/1963	174
18	07/08/1967	171
19	10/01/1949	170
20	14/11/1969	160
21	05/02/2002	157
22	16/06/1952	156
23	04/03/1977	155
24	03/05/1948	154
25	04/04/1988	152
26	28/04/1966	151
27	05/03/1979	151

For the purposes of this study, an analysis of daily rainfall data was undertaken to identify and place past storm events in some context. All daily rainfall depths greater than 150 mm recorded at Centennial Park (112 years of record), Randwick Bowling Club (124 years of record) and Randwick Racecourse (75 years of record) have been ranked and shown in Table 3.

The main points regarding these data are:

- February 1990 was in the top 10 for all gauges, showing very similar rainfalls at each gauge (between 220 and 245 mm);
- August 1986 looks like the most significant widespread daily rainfall event;
- March 1942 and August 1986 were the largest daily events recorded for the Centennial Park and Randwick Bowling Club gauges with approximately 300 mm. Randwick

Racecourse also recorded high rainfall for these days, although some spatial variation is shown;

- February 1992 showed a significant difference between the three gauges (151 mm, 162 mm and 294 mm). Analysis of the Botanic Gardens and Observatory Hill gauges show rainfalls of 264 mm and 190 mm for this day, implying a wide spatial range of rainfall depths;
- Data for the November 1984 event, which was known to produce flooding in the study area, is available at the Randwick Racecourse gauge and the Paddington gauge where it ranked 10th for total daily rainfall.

3.6. Analysis of Pluviometer Data

Pluviometer records provide a more detailed description of temporal variations in rainfall for sub-daily durations. Table 4 lists the maximum storm intensities for the four largest recent rainfall events from both the pluviometers and the daily read gauges.

Table 4: Maximum Recorded Storm Depths (in mm)

	5 Nov 1984		8/9 Nov 1984		6 Jan 1989		26 Jan 1991	
Station Location	30 min	60 min	30 min	60 min	30 min	60 min	30 min	60 min
Paddington	36	51	54	91	53	54	52	53
Observatory Hill	20	32	90	119	42	42	60	65
UNSW (Avoca Street)(1)	65	112	41	58	-	-	-	-
UNSW (Storey Street) (1)	65	90	33	46	-	-	-	-

Station Location	5 Nov 1984	8 Nov 1984	9 Nov 1984	6 Jan 1989	26 Jan 1991
Royal Botanic Gardens (daily)	-	37	248	49	59
Observatory Hill (daily)	121	44	234	47	65
Paddington (daily)	108	71	208	63	54

Notes:

(1) From Reference 3.

The above data indicate that for January 1989, March 1989 and January 1991 the peak 30 minute rainfall comprised the majority of the daily rainfall. However, for November 1984 the 30 minute peak was part of a much larger rainfall event, for both the storms investigated.

Storm intensities and durations recorded at the Paddington gauging station for significant historical storm events are given in Table 5.

Table 5: Paddington Pluviometer Storm Intensities (mm/h)

Duration	**6 min**	**10 min**	**20 min**	**30 min**	**60 min**	**120 min**
12 Aug 1983	175	156	106	84	48	28
(approx. ARI)	*(10)*	*(20)*	*(10)*	*(10)*	*(5)*	*(2)*
5 Nov 1984	120	108	84	72	52	39
(approx. ARI)	*(2)*	*(2)*	*(5)*	*(5)*	*(5)*	*(10)*
8-9 Nov 1984	125	123	114	108	91	74
(approx. ARI)	*(2)*	*(5)*	*(10)*	*(25)*	*(75)*	*(>100)*
6 Jan 1989	215	195	155	108	56	30
(approx. ARI)	*(50)*	*(50)*	*(50)*	*(25)*	*(5)*	*(5)*
9 Mar 1989	140	138	114	85	54	28
(approx. ARI)	*(5)*	*(10)*	*(15)*	*(10)*	*(5)*	*(2)*
21 Apr 1989	140	120	78	54	29	14
(approx. ARI)	*(5)*	*(5)*	*(2)*	*(2)*	*(1)*	*(1)*
26 Jan 1991	190	162	138	103	53	27
(approx. ARI)	*(20)*	*(2)*	*(40)*	*(20)*	*(5)*	*(2)*

Data taken from Reference 2.

3.6.1. November 1984 Storm

The 8-9th November 1984 storm was a significant rainfall event across the Sydney and Wollongong region and is well documented in Reference 4. Table 6 shows that this storm had an approximate 100 Year ARI intensity across several locations in Sydney. The storm was separated into two distinct bursts (6:00am to 10:00am and 9:00pm to midnight). The latter was the most intense period and flooding was reported throughout the catchment, though the actual timing of the flooding is unknown.

Table 6: ARI Estimates of the 8-9th November 1984 Rainfall (From Reference 2)

Station	**Rainfall Duration**				
	0.5 hour	1 hour	2 hour	3 hour	6 hour
Sydney – Observatory Hill	100y	100y	100y	100y	100y
Mosman	20y	50y	100y	20y	10y
Vaucluse	100y	100y	50y	20y	10y

At the Paddington gauge the 8-9th November 1984 storm had similar intensity of the 30 minute duration as the January 1989 and January 1991 storms. However, anecdotal information indicates that the 8-9th November 1984 event produced greater flooding than other recent events in downstream areas of the catchment. Possibly this is because the event was part of an extended period of rainfall that partially "filled" the lower floodplain areas prior to the peak storm burst.

3.6.2. January 1989 and January 1991 Storms

The 6th January 1989 and 26th January 1991 storm events were both high intensity, short duration events which occurred over the period of an hour. Although not as large as the 8-9th November 1984 storm in terms of volume or longer duration intensity, the 1989 and 1991 storm events had a higher intensity for durations up to the 20 minute burst and caused extensive flooding throughout the catchment. For the most intense 20 minute rainfall burst the 6 January 1989 event had an approximate ARI of 50 years, and the 26 January 1991 event had an ARI of approximately 40 years. For upper catchment areas with short critical durations, these shorter more intense rainfall events are more likely to cause flooding throughout the majority of the study area.

3.7. Design Rainfall Data

Design rainfall depths and temporal patters for various storm durations at the study area were obtained from Australian Rainfall and Runoff 1987 (ARR87), for events up to and including the 100 Year ARI event. Probable Maximum Precipitation estimates were derived according to Bureau of Meteorology (BoM) guidelines (Reference 5). A summary of the design rainfall depths is provided in Table 7 and a comparison of the design rainfall Intensity-Frequency Duration (IFD) data and significant historic storms in the catchment is shown on Figure 5.

Table 7: Rainfall Intensity-Frequency Duration Data

Duration	**Design rainfall Intensity (mm/hr)**						
	1 Year	2 Years	5 Years	10 Years	20 Years	50 Years	100 Years
5 minute	106	134	168	188	213	247	272
10 minute	80.9	103	131	146	167	194	214
20 minute	59.5	76.5	98.1	111	127	149	165
30 minute	48.5	62.5	80.9	91.7	106	124	138
1 hour	32.7	42.4	55.4	63	73	86.2	96.2
2 hour	21.1	27.3	35.8	40.8	47.4	56	62.6
3 hour	16	20.8	27.3	31.1	36	42.6	47.6
6 hour	10	13	17	19.3	22.4	26.4	29.5
12 hour	6.35	8.21	10.7	12.2	14.1	16.6	18.5
24 hour	4.11	5.31	6.93	7.87	9.1	10.7	12
48 hour	2.64	3.41	4.45	5.06	5.85	6.9	7.69
72 hour	1.96	2.54	3.3	3.74	4.33	5.1	5.69

3.8. Historical Flood Information

A data search was carried out to identify the dates and magnitudes of historical floods. The search concentrated on the period since approximately 1970 as data prior to this date would generally be of insufficient quality and quantity for model calibration. Unfortunately there were no stream height gauges in the catchment. The following sources were used:

- Woollahra Municipal Council records,
- Sydney Water database,
- previous reports,

- questionnaire issued in November 2012,
- follow-up conversations with local residents.

A summary of flood events is listed in Table 8, with descriptions of historical flood information provided in Table 9 and locations of recorded flooding shown on Figure 9.

Table 8: Historical Floods

Event	Depth estimate	Qualitative description	Total
18 February 1959	2	0	2
19 November 1961	1	0	1
December 1970	0	1	1
1 March 1975	0	1	1
1 March 1977	1	0	1
4 March 1977	2	0	2
1 November 1979	0	1	1
1 February 1980	0	1	1
1 February 1981	0	1	1
12 August 1983	2	0	2
8 November 1984	2	1	3
March 1989	0	1	1
April 1989	0	1	1
6 January 1989	11	0	12
26 January 1991	7	0	7
9 April 1998	1	2	3
Unknown	2	1	3

Table 9: Historical Flood Information

ID	Location	Description	Flood Event	Observed Depth (m)	Comments	Source
1	Oxford Street (East)	Above floor inundation	January 1989	1.0	Depth observed above footpath level	Reference 1
			March 1989	-		Reference 1
			April 1989	-		Reference 1
			January 1991	-		Reference 1
2	Taylor Street low point	Properties adjacent to low point flooded	1984	1.3	Depth observed in the road	Reference 1
			January 1989	<1.3	Flooding marginally less than 1984 and January 1991 events	Reference 1
			January 1991	1.3	Depth observed in the road	Reference 1
3	Sturt Street low point	Properties adjacent to low point flooded	1984	1.6	Depth observed in the road	Reference 1
			January 1989	<1.6	Flooding marginally less than 1984 and January 1991 events	Reference 1
			January 1991	1.6	Depth observed in the road	Reference 1
4	Taylor Street	Property flooded	26 January 1991	-	Overtopped 0.5 m high fence at front of property peak at floor level at rear of property	Reference 1
5	Oxford Street (West)	Property flooded	4 March 1977	0.15	Flooding at depth above footpath	SWC database
			January 1991	0.45	Water up to 0.45 m above adjacent footpath to No. 84	Reference 1
6	Oxford Street (West)	Property flooded	4 March 1977	0.15	Flooding at depth above footpath	SWC database
			January 1991	0.45	*as for previous*	Reference 1

ID	Location	Description	Flood Event	Observed Depth (m)	Comments	Source
7	Oxford Street (West)	Property flooded	January 1991	0.45	*as for previous*	Reference 1
8	St Vincents Hospital	Building flooded	December 1970	-	Caused by drainage backwater. Since then the Hospital has isolated its drainage system from the main line.	SWC database
		Basement flooded	January 1989	-		Reference 1
9	Boundary Street	Property flooded	January 1989	0.15	flows along adjacent path routing through the property	Reference 1
10	Boundary and Liverpool Street	Severe street flooding	6 January 1989	0.5	Depth/velocity such that 16 cars were washed 100 m down the street	Reference 1
11	Intersection of Womerah Avenue and Liverpool Street	Eastern footpath flooded	-	0.15	Caused by street litter and park cars	Reference 1
12	Corner of Boundary Street and McLachlan Avenue	Above floor inundation	-	-	Above floor inundation of properties adjacent to McLachlan Street eastern low point and extending south for 20 m.	Reference 1
13	Neild Avenue low point	Properties flooded	January 1989	0.5	Flow routed through Neild Avenue properties to west of McLachlan Avenue low point	Reference 1
14	Neild Avenue intersection with New South Head Road	Road flooded	18 February 1959	0.05	depth above coping	Reference 1
			19 November 1961	0.05	depth above coping	Reference 1
			12 August 1983	0.45	depth on road	Reference 1
				1.3	depth above coping	Reference 1
			January 1989	0.4	Southern carriage way inundated	Reference 1

ID	Location	Description	Flood Event	Observed Depth (m)	Comments	Source
			January 1991	0.4		Reference 1
15	Waratah Street low point	Road flooded	January 1989	0.5	Surcharging of street drains and ponding of above to of kerb	Reference 1
16	Bayswater Road	Above floor inundation	-	-	Carpark runoff enters ground level shop car-park	Reference 1
17	Bayswater Road	Property flooded	18 February 1959	0.05	Depth above coping of channel	SWC database
18	Oxford Street	Property flooded	1 March 1975	-	Sydney Water comments indicate "greater than 100 Year ARI storm"	SWC database
20	Taylor Street	Property flooded	1 November 1979	-	Gutter flow by-passes gully in South Dowling Street and drains into sag in front of house due to blockage in gully"	SWC database
			1 February 1980	-		SWC database
			1 February 1981	-		SWC database
21	McLahlan Avenue	Above floor flooding	12 August 1983	0.2	depth above floor through showrooms of Monaco Motors	SWC database
				0.6	Above coping of adjacent channel	SWC database
			26 January 1991	-	Monaco Motors flooded	Reference 1
22	Roylston Street	Above floor inundation	6 January 1989	0.9	Depth above floor with houses and garages flooded and extensive damage to property.	SWC database
23	Roylston Street	Above floor inundation	6 January 1989			SWC database
24	Cecit Street	Above floor inundation	6 January 1989			SWC database
25	Barcom Avenue	Above floor inundation	9 April 1998	0.5	Depth above back gate, adjacent Boundary road. Lower floor flooded	SWC database
26	Barcom Avenue	Above floor inundation	9 April 1998	-	Back yard and lower floor flooded	SWC database
27	Barcom Avenue			-		SWC database

ID	Location	Description	Flood Event	Observed Depth (m)	Comments	Source
28	Corner of Greens Street and Oxford Street	Road flooded	1 March 1977	0.5	depth above footpath	CoS Database
			4 March 1977	0.5		CoS Database
29	City East Mail Centre	Above floor inundation	8 November 1984	-	Lower rooms flooded	CoS Database
30	Sims St	Road flooded	19 February 2012	0.6	Occurred at rear of Taylor Street properties	Community Consultation
31	Taylor St Low Point	Property Inundation - Above Floor Level	19 February 2012	0.05	Property flooded from Sims Street through to Taylor Street	Community Consultation
32	Taylor St Low Point	Road Flooding	Regularly	0.15	Occurs on Taylor Street	Community Consultation
33	Boundary Street	Property Inundation - Backyard	-	0.9	Inundation occurred in back yard and moved to front of property.	Community Consultation
34	Barcom Ave	Road Flooded	14 to 16 June 2007	0.15	Approximate height above footpath	Community Consultation
35	Sims St	Road Flooding	12 February 2010	0.6	Occurred at bend in road	Community Consultation
36	McLachlan Ave	Above Floor Inundation	-	0.15	Flooding above floor level on ground level and on below ground levels	Community Consultation

4. COMMUNITY CONSULTATION

In collaboration with Council, a questionnaire and newsletter were distributed to residents and owners of property within the study area by post, describing the role of the Flood Study in the floodplain risk management process, and requesting records of historical flooding. A total of 792 surveys were distributed with reply paid envelopes, and 36 responses were received (a return rate of 5%).

The information requested in the survey included details about length of residency in the catchment, descriptions of any experiences of flooding, and evidence of flood heights or extents such as photographs of flood marks.

The occasions when respondents recalled being affected by flooding are summarised in Table 10. The most frequently recalled flood related to the June 2007 storm, although other events were also mentioned by a significant number of respondents. A summary of responses received is shown on Figure 6 and Figure 7.

Table 10: Summary of Reported Incidents of Flooding

Flood Event	Total Reponses	House Flooded (above floor)	Other Buildings Flooded (above floor)	Other Descriptions of Flooding
January 1989	1	0	0	1
February 1993	1	0	0	1
April 1998	1	0	0	1
February 2001	1	0	1	1
June 2007	2	0	2	1
February 2009	1	0	0	1
February 2010	1	1	0	1

The flood experiences described in the survey responses generally related to nuisance flooding, such as ponding of stormwater in roadways or gardens, although one instance of above floor flooding was also reported. February 2010 was the only storm with reported above floor inundation of a residential property. Photographs showing flooding in Victoria Street Paddington from 1989 are shown on Figure 8.

A copy of the questionnaire and newsletter is provided in Appendix B.

5. STUDY METHODOLOGY

5.1. General Approach

The approach adopted in flood studies to determine design flood levels largely depends upon the objectives of the study and the quantity and quality of the data (survey, flood, rainfall, flow etc). High quality survey datasets were available for this study, which enabled a detailed topographic model of the catchment to be established. However the historical hydrologic data (such as rainfall patterns and stream-flows) were relatively limited.

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

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

When historical flood data is available it can be used to allow calibration of the models, and increase confidence in the estimates. The calibration process is undertaken by altering model input parameters to improve the reproduction of observed catchment flooding. Recorded rainfall and stream-flow data area 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.

There are no stream-flow records in the catchment, so the use of a flood frequency approach for the estimation of design floods is not possible.

Flood estimation in urban catchments generally presents challenges for the integration of the hydrologic and hydraulic modelling approaches, which have been treated as two distinct tasks as part of traditional flood modelling methodologies. As the main output of a hydrologic model is the flow at the outlet of a catchment or sub-catchment, it is generally used to estimate inflows from catchment areas upstream of an area of interest, and the approach does not lend itself well to estimating flood inundation in mid- to upper-catchment areas, as required for this study. The aim of identifying the full extent of flood inundation can therefore be complicated by the separation of hydrologic and hydraulic processes into separate models, and these processes are increasingly being combined in a joint modelling approach.

In view of the above, 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 Reference 6, and the runoff hydrographs were then used in a hydraulic model to estimate flood depths, velocities and hazard in the study area.

The sub-catchments in the hydrologic model were kept small (less than a typical residential block) such that the overland flow behaviour for the study was generally defined by the hydraulic model. This joint modelling approach was calibrated against observed historical flood levels. Additionally, the estimated flows at various points in the catchment were validated against previous studies and alternative methods.

5.2. 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,
- 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 pipe and open channels is calculated using unsteady flow hydraulics. Open channel flow uses the simpler Hydraulic Grade Line method. 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.

It should be noted that the version of DRAINS used in this study is not a true unsteady flow model as it does not account for the attenuation effects of routing through temporary floodplain storage in overland areas (down streets or in yards).

5.3. Hydraulic Model

The availability of high quality LiDAR data means that the study area is suitable for two-dimensional (2D) hydraulic modelling. Various 2D software packages are available (SOBEK, TUFLOW, Mike FLOOD) and the TUFLOW package (Reference 7) was adopted as it is widely used in Australia and was considered most suitable for use in this study.

The Rushcutters Bay study area consists of a wide range of development, with residential, commercial and open space areas. Overland flood behaviour in the catchment is generally two-dimensional, with flooding along road reserves and areas prone to ponding (e.g. Taylor Street). For this catchment, the study objectives required accurate representation of the overland flow system including kerbs and gutters and defined drainage controls.

The 2D model is capable of dynamically simulating complex overland flow regimes and interactions with sub-surface drainage systems. 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 underground piped and overland flow behaviour.

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 flow-paths, and
- inherently represent the available flood 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).

5.4. Design Flood Modelling

Following validation of the hydrologic model against previous studies with similar catchment characteristics and alternative calculation methods, the following steps were undertaken:

- design runoff hydrographs for localised sub-catchments were obtained from the DRAINS hydrologic model and applied as inflows to the TUFLOW model;
- sensitivity analysis was undertaken to assess the relative effect of changing various modelling parameters; and
- design floods were modelled in TUFLOW using parameters selected to provide a sensible match between design flood levels and available recorded peak flood levels from historical events.

6. HYDROLOGIC MODELLING

6.1. Sub-catchments

A hydrological model of the study catchment was established using the DRAINS software package (Reference 8).

Sub-catchment areas were delineated based on LiDAR survey and making the assumptions that:

- properties generally drain to streets or inlet pits; and
- flow in streets is along gutters and uni-directional.

The DRAINS hydrologic runoff-routing model was used to determine hydraulic model inflows for the local sub-catchments within the study area. The catchment layout for the model is shown on Figure 10.

6.2. Key Model Parameters

6.3. Impervious Areas

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete aprons occurs significantly faster than from natural surfaces, resulting in a faster concentration of flow at the bottom of a catchment, and increased peak flow in some situations. It is therefore necessary to estimate the proportion of a catchment area that is covered by such surfaces.

For each sub-catchment the proportion of pervious (grassed and landscaped), impervious (paved) and supplementary areas (paved not directly connected to pipe system) were determined from field and aerial photographic inspections. The adopted values are summarised in Table 11.

Table 11: Summary of Catchment Imperviousness values used in DRAINS

Area	Area (ha)	%
Paved Area	67.5	74
Grassed Area	19.4	21
Supplementary	4.6	5
TOTAL	91.5	100

6.4. Rainfall Losses

Methods for modelling the proportion of rainfall that is “lost” to infiltration are outlined in AR&R. The methods are of varying complexity, with the more complex options only suitable if sufficient data are available (such as detailed soil properties). An industry accepted method used for design flood estimation is the Horton Infiltration loss model used within DRAINS software.

Losses from a paved or impervious area are considered to comprise only an initial loss (an amount sufficient to wet the pavement and fill minor surface depressions). Losses from grassed areas are comprised of an initial loss and a continuing loss. The continuing loss was calculated from infiltration curves based on work by Horton in the 1930's which decreases as the storm duration progresses and is determined using the estimated representative soil type and antecedent moisture condition.

It has been assumed that the soil in the catchment has a moderate infiltration rate potential and the antecedent moisture condition was considered to be rather wet. The latter was justified by the fact that for many historical storms in the catchment, the peak rainfall burst typically occurs within a longer event that possibly has a duration of a few days. The adopted parameters are summarised in Table 12.

Table 12: Adopted Hydrologic Loss Parameters

RAINFALL LOSSES	
Paved Area Depression Storage (Initial Loss)	1.0 mm
Grassed Area Depression Storage (Initial Loss)	5.0 mm
SOIL TYPE	3
Moderate infiltration rates and moderately well drained. This parameter, in conjunction with the Antecedent Moisture Condition, determines the continuing loss (defined by Horton's infiltration equation).	
ANTECENDENT MOISTURE CONDITIONS	3
Description	Rather Wet
Total Rainfall in 5 Days Preceding the Storm	12.5 to 25 mm

6.5. Time of Concentration

The surface runoff from each sub-area contributing to a pit has a particular *time of concentration*. This is defined as the time it takes for runoff from the upper part of a sub-area to start contributing as inflow to the pit. It is mainly related to the flow path distance, slope and surface type over which the runoff has to travel.

The time of concentration was defined as overland flow time based on the Kinematic wave equation. The flow time was defined using a flow length based on the sub-catchment slope and the size and shape of the contributing catchment. The relationship was developed based on a catchment of similar characteristics within the Sydney region and is generally suitable for application in the present investigation.

Time of concentration can have a significant bearing upon the accumulated peak flows achieved further downstream. Sensitivity to these assumptions was assessed in Section 10.

6.6. Validation of Methodology

Ideally hydrologic models are calibrated and validated against observed stream flow information; however for the study area no such data was available. Thus verification is undertaken in which results from the current study were compared with similar studies in adjacent catchments and specific and general expectations of catchment flooding behaviour.

Flow results from the Kensington – Centennial Park Flood Study, June 2011 (Reference 3) and the Rushcutters Bay Flood Study, October 2007 (Reference 2) were compared to those used in the current study for individual sub-catchments.

Table 13 provides the model comparisons for 3 random sub-catchments from each model.

Table 13: Comparison of 20 and 100 Year ARI DRAINS Results with References 3 and 2

Model	Catchment Name	Area (ha)	Impervious %	20 Year ARI		100 Year ARI	
				Peak Discharge (m^3/s)	Specific Yield ($m^3/s/ha$)	Peak Discharge (m^3/s)	Specific Yield ($m^3/s/ha$)
Current Study	RB049	4.6	76	1.9	0.4	2.5	0.5
Current Study	RB048	0.7	92	0.3	0.5	0.4	0.6
Current Study	RB003	3.3	92	1.5	0.5	1.9	0.6
Reference 3	F-G	3.3	95	1.8	0.5	2.3	0.7
Reference 3	E1-E2	2.3	80	1.0	0.5	1.3	0.6
Reference 3	AN2Det	3.5	83	1.6	0.5	2.1	0.6
Reference 2	aP24AA2	14.7	90	8.2	0.6	10.1	0.7
Reference 2	aP7Z7	0.4	90	0.2	0.6	0.3	0.7
Reference 2	aP3A1	2.7	90	1.5	0.5	1.9	0.7

Discrepancies between the compared specific yields can be attributed to a number of reasons such as the variance in loss parameters, differences in land use and difference in the applied routing method (peak flow also correlates to catchment area, but not linearly).

Specific yield for the 100 year ARI event in the current study was found to vary from 0.5 to 0.6 m^3/s per hectare and averaging at 0.6 m^3/s per hectare. The range of values is largely dependent on land use with more urbanised sub-catchments producing higher specific yields.
The results are comparable for the studies considered.

7. HYDRAULIC MODELLING

7.1. Terrain Model

A computational grid cell size of 2 m by 2 m was adopted, as it provided an appropriate balance between providing sufficient detail for roads and overland flow paths, while still resulting in practicable computational run-times. The model grid was established by sampling from a triangulation of filtered ground points from the LiDAR dataset.

Permanent buildings and other significant structures likely to act as significant flow obstructions were incorporated into the terrain model. These features were identified from the available aerial photography and modelled as impermeable obstructions to the flood flow (i.e. they were removed from the model grid).

7.2. Boundary Conditions

The model schematisation is illustrated on Figure 11, including the location of the stormwater pits and pipes. In addition to runoff from the catchment, the reach of the open channel downstream of Glenmore Road can also be influenced by backwater effects from high water level in Rushcutters Bay. These two distinct mechanisms produce flooding in Rushcutters Bay as well as in the open channel but may not result from the same storm. Under some circumstances it can be expected that tidal influences will occur in conjunction with rainfall events. Consideration must therefore be given to accounting for the join probability of coincident flooding from both catchment runoff and backwater effects from Rushcutters Bay.

A full joint probability analysis is beyond the scope of the present study, and research into this issue for the east coast of Australia has not yet led to a comprehensive approach for modelling the combined mechanisms. 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.

NSW government guidelines (Reference 10) specify approaches for setting the tailwater at an ocean level boundary for flood risk assessment. The guideline provides three approaches to the development of appropriate tailwater levels for open entrances, for consideration in flood risk assessments. The first two approaches involve a fixed and dynamic boundary condition with a maximum level of 2.6 mAHD. The third requires a site specific assessment, which is recommended where the first 2 options are considered too conservative. The Consideration of Sea Level Rise in Flood and Coastal Risk Assessment paper presented at the NSW Floodplain Management Authorities Conference (McLuckie et al, 2011) states:

> *“Where the* [2.6 mAHD] *fixed approach is likely to be too conservative for the resultant decision, either the dynamic ocean boundary provided in the guideline or one specifically developed for the location and the associated conditions should be used to assess flood behaviour. Studies undertaken under the State’s Floodplain Management Program are not to use the conservative fixed ocean boundary condition unless*

specifically agreed to by DECCW."

It was therefore considered appropriate to determine a site specific ocean water level boundary condition for this study. Rushcutters Bay is in a highly sheltered portion of Sydney Harbour. The large size of Sydney Harbour significantly reduces the potential for wave setup to increase harbour water levels (as there is enough depth at the entrance for ocean wave inflows to flow back out through the entrance).

As a result of the estuary size and the protected location of Rushcutters Bat, the influence of ocean level components such as wave action and associated potential for wave setup are significantly reduced. These effects have a relatively short duration and are more important for smaller coastal catchments with an exposed entrance. Therefore for this study the wave setup was assumed to be negligible. For Rushcutters Bay, the principal components to be considered in setting tailwater levels are tides and barometric effects (storm surge).

The annual high astronomical tide (due to gravitational effects of celestial bodies) on the NSW coast is around 1.1 mAHD to 1.2 mAHD. The highest recorded tide at Fort Denison in Sydney Harbour is 1.5 mAHD, which included barometric effects (storm surge) from a low pressure cell, and the 1% AEP level at Fort Denison is 1.45 mAHD.

A table of design tailwater scenarios adopted for this study is given in Table 14 with design ocean levels taken from Reference 11.

Table 14 – Adopted Co-incidence of Ocean and Rainfall Events

OCEAN Event		DESIGN EVENT (ARI)	RAINFALL Event	
Peak Design Ocean Level (m AHD)	Co incident Design Rainfall Event (ARI)		Co incident Design Ocean Event (ARI)	Co incident Design Ocean Level (m AHD)
1.45	100 year	**PMF**	100 year	1.43
1.43	20 year	**100 year**	20 year	1.40
1.42	20 year	**50 year**	20 year	1.40
1.40	20 year	**20 year**	20 year	1.40
1.20	10 year	**10 year**	10 year	1.20
1.20	5 year	**5 year**	5 year	1.20
1.20	2 year	**2 year**	2 year	1.20

For ocean level events smaller than a 20 year ARI event, the relevant design flows are used in conjunction with a level of 1.2 mAHD, slightly higher than the Highest Astronomical Tide within Sydney Harbour.

Along the LGA boundary, which coincides with Nield Avenue and the Sydney Water open channel, design flood levels from Reference 2 were adopted as a boundary condition. Results from Reference 2 were unavailable for the 2 year ARI event and therefore a 5 year ARI downstream boundary condition was adopted for this event.

For historic events, sensitivity analyses of boundary conditions were undertaken with the following scenarios shown in Table 15. It was found that the tailwater boundaries had very little impact on results. This is because even the low-lying reclaimed areas of the catchment are generally above 2 m, which is above the range of adopted tailwater levels.

Table 15 – Boundary Condition Scenarios for Historic Rainfall Events

Scenario	Weigall Tailwater	Ocean Level (mAHD)
1	5 year	0.0
2	5 year	1.0
3	100 year	0.0
4	100 year	1.0

A sensitivity analysis of the relative impacts of assuming different tailwater conditions due to climate change is presented in Section 10.3.

7.3. Hydraulic Roughness

The adopted roughness values are consistent with typical values in the literature (References 6, 12, and 13) and previous experience with modelling similar catchment conditions. The sensitivity of model results to changes the roughness values is discussed in Section 10.

Table 16 - Mannings ‘n’ values

Surface Type	Manning’s “n” value
Very short grass or sparse vegetation	0.035
General overland areas, gardens, roadside verges, low density residential lots etc. (default)	0.045
Medium density vegetation	0.060
Heavy vegetation	0.100
Roads, paved surfaces	0.025
Concrete pipes	0.013

Culvert Type	Manning’s “n” value
Concrete pipes	0.013
Clay Pipes	0.025
Brick	0.014
PVC	0.011

7.4. Blockage Assumptions

Blockage of hydraulic structures is an important issue in the design and management of drainage systems. Blockage is produced by a range of different processes and can reduce the capacity of drainage systems by partially or completely closing the drainage structure.

Inlet pits are critical parts of drainage systems, and collect the runoff from the streets and other parts of the urban catchment and convey these to the piped underground system. Stormwater inlets are especially prone to blockage and temporary blockage may occur during a storm due to a range of issues. All materials that may occur naturally on the road can end up in the pit inlets; the most common material is leaves and other small vegetation as well as general litter. Other obstructions include parked cars or trucks. Blockage was applied to inlet pits rather than pipes for this study.

It is impossible to accurately estimate the degree of blockage during a storm and for this reason a conservative approach has been applied which generally assume trunk drainage pipes of diameter smaller than 450 mm do not convey flow in the TUFLOW modelling. In some locations the trunk drainage system had no direct connection to inlet pits and under these circumstances Council pipes smaller than 450mm linking inlet pits to the trunk drainage system assumed to be clear of blockage in order to more accurately model the trunk drainage system capacity. Pipes smaller than 450mm in diameter were also included in the modelling where they represented the only means of drainage from an areas (such as a trapped low point).

Blockage to inlet pits was applied as per the Queensland Urban Drainage Manual (Reference 14) and Project 11 of the AR&R revision project (Table 17).

Table 17 – Theoretical capacity of inlet pits based on blockage assumptions

Sag Inlet Pit	
Kerb Inlet	80%
Grated Inlet	50%
Combination	grate assumed 100% blocked
On-Grade Inlet Pit	
Kerb Inlet	80%
Grated Inlet	60%
Combination	90%

The sensitivity of the catchment's drainage response to blockage of assumptions within the underground drainage network is assessed in Section 10.

8. MODEL CALIBRATION

8.1. Overview

It is preferable to test the performance of the hydrological/hydraulic models against observed flood behaviour from past events within the catchment. The assumed model parameters can be adjusted so that the modelled behaviour best represents the historical patterns of flooding. The process of adjusting model parameters to best reproduce observed flood behaviour is known as model calibration. Usually, the models are calibrated to a single flood event for which there is sufficient flood data available (e.g. peak-flood levels, observations regarding flowpaths or flood extents etc). The performance of the calibrated model can then be tested by simulating other historical floods and comparing the ability of the calibrated models to reproduce the observed behaviour. This process is known as model validation.

To calibrate/validate the models requires a sufficient amount of flood data within the model extent. There is no stream gauge within the catchment and therefore it is not possible to conduct a thorough calibration of modelled flows to observed data. The largest flood events known to have occurred within the catchment occurred on 8-9th November 1984, 6 January 1989 and 26 January 1991. For these major events, there is limited flood height data, and only anecdotal or approximate depths were available. As a result the hydrologic and hydraulic models were validated against observed flood behaviour and limited emphasis was placed on tuning the models to exactly match depths.

When flooding occurs within the catchment in future, it is recommended that Council undertake to collect any available information (rainfall data, flood heights etc) as soon as practicable after the event.

8.2. Validation Results

The modelled results for the historical events were compared to observed flood behaviour and depth information documented in Reference 1 and additional observations were collected as part of the Community Consultation process. A comparison of this data against the model results for 8-9th November 1984, 6th January 1989 and 26th January 1991 is provided in Table 18 and Figure 12, Figure 13 and Figure 14.

Table 18 – Comparison of Historic Flood Data to Modelled Results

Location	Flood Event	Description	Observed		Modelled		Difference (m)
			Level (mAHD)	Depth (m)	Level (mAHD)	Depth (m)	
Taylor Street Low Point	Nov 1984	Depth in road	-	1.3	47.1	0.5	-0.8
Sturt Street Low Point	Nov 1984	Depth in road	-	1.6	46.6	1.8	0.2
Oxford Street (East)	Jan 1989	Depth above footpath	-	1.0	63.9	0.9	-0.1
Taylor Street Low Point	Jan 1989	Depth in road	-	< 1.3	47.2	0.6	-0.7
Sturt Street Low Point	Jan 1989	Depth in road	-	< 1.6	46.6	1.8	0.2
Boundary Street	Jan 1989	Flow through property	-	0.15	-	-	-
Boundary and Liverpool St	Jan 1989	Street Flooding	-	0.5	21.5	0.5	0.0
Neild Ave Low Point	Jan 1989	Properties Flooded	-	0.5	6.0	0.5	0.0
Intersection of Neild Ave and New South Head Rd	Jan 1989	Southern Carriageway Inundated	-	0.4	5.0	0.5	0.1
Waratah Street Low Point	Jan 1989	Depth in Road	-	0.5	2.5	0.4	-0.1
Oxford Street (West)	Jan 1991	Depth above adjacent footpath	-	0.45	46.4	0.4	-0.05
Oxford Street (West)	Jan 1991	Depth above adjacent footpath	-	0.45	46.4	0.5	0.05
Oxford Street (West)	Jan 1991	Depth above Adjacent footpath	-	0.45	46.4	0.4	-0.05
Oxford Street (East)	Jan 1991	Depth above footpath	-	1.0	63.8	0.8	-0.2
Taylor Street Low Point	Jan 1991	Depth in road	-	1.3	47.2	0.5	-0.8
Taylor Street Low Point	Jan 1991	Overtopped front fence	> 47.4	-	47.2	0.4	-0.2
Sturt Street Low Point	Jan 1991	Depth in road	-	1.6	46.6	1.8	0.2
Intersection of Neild Ave and New South Head Rd	Jan 1991	Southern Carriageway Inundated	-	0.4	5.0	0.4	0.0

In the January 1991 event, water overtopped the 0.5 m high front fence near the Taylor Street low point and at the rear of the property lapped at floor level. This information was converted to an approximate height in mAHD based on surrounding LiDAR data.

Properties within Sims, Taylor and Sturt Streets have experienced substantial road flooding in the past with reported depths of greater than 1 m. The lowest available flow-path from Taylor Street to Sturt Street is through a property along Taylor Street. Photo 1 shows the existing fence with a gap underneath, however it is not known whether the same fence was in place in historic events. Given the difference in peak flood depths between Taylor Street and Sturt Street low points, it is quite likely that the flow-path through Taylor Street was historically more blocked (by fences/gates for example) than under current conditions, which would have increased flood levels within Taylor Street.

Photo 1: Flow path from Taylor Street to Sturt Street

Property flooding at Boundary Street was observed in January 1989. Reference 1 states that the flooding is likely a local runoff problem and that flows along the adjacent path routed through the property from the rear and into Boundary Street. Survey information within this area is not sufficiently defined in order for the hydraulic model to be able to replicate this flow path and as such modelled results do not match observed flooding at this location.

Recorded flood levels were also compared against design flood levels (in Table 19), to provide some perspective as to whether the modelled range of design flood levels was consistent with observed historical variability. Recorded flood levels near the Weigall Sportsground open channel have not been included as part of this assessment as downstream flood levels have been adopted from Reference 2.

Table 19 – Comparison of Historic Flood Data to Design Results

Location	Flood Event	Observed Depth (m)	Modelled Flood Depth (mAHD)			
			2Y ARI	10Y ARI	20Y ARI	100Y ARI
Oxford Street (West)	Jan 1989	1.0	0.5	0.8	0.9	1.1
Oxford Street (East)	Mar 1977	0.15	0.3	0.4	0.4	0.4
Oxford Street (East)	Jan 1991	0.45	0.3	0.4	0.4	0.4
Oxford Street (East)	Mar 1977	0.15	0.4	0.5	0.5	0.5
Oxford Street (East)	Jan 1991	0.45	0.4	0.5	0.5	0.5

Location	Flood Event	Observed Depth (m)	Modelled Flood Depth (mAHD)			
			2Y ARI	10Y ARI	20Y ARI	100Y ARI
Oxford Street (East)	Jan 1991	0.45	0.4	0.4	0.4	0.5
Sims Street Low Point	Feb 2012	0.6	0.5	0.7	0.8	0.9
Sims Street Low Point	Feb 2010	0.6	0.5	0.7	0.8	0.9
Taylor Street Low Point	Nov 1984	1.3	0.4	0.5	0.6	0.6
Taylor Street Low Point	Jan 1989	< 1.3	0.4	0.5	0.6	0.6
Taylor Street Low Point	Jan 1991	1.3	0.4	0.5	0.6	0.6
Taylor Street Low Point	-	0.15	0.1	0.1	0.1	0.2
Taylor Street Low Point	Jan 1991	> 0.6	0.3	0.4	0.5	0.5
Sturt Street Low Point	Nov 1984	1.6	1.6	1.8	1.8	1.8
Sturt Street Low Point	Jan 1989	< 1.6	1.6	1.8	1.8	1.8
Sturt Street Low Point	Jan 1991	1.6	1.6	1.8	1.8	1.8
Boundary Street	Jan 1989	0.15	-	-	-	-
Boundary Street	-	0.9	-	-	-	-
Barcom Avenue	June 2007	0.15	0.2	0.2	0.2	0.2
Barcom Avenue	April 1998	0.5	0.9	0.9	0.9	0.9
Boundary and Liverpool St	Jan 1989	0.5	0.3	0.4	0.5	0.6
Intersection of Womerah Ave and Liverpool St	-	0.15	0.1	0.1	0.2	0.2
McLachlan Avenue	Aug 1983	0.2	0.7	0.8	0.9	0.9
Neild Ave Low Point	Jan 1989	0.5	0.4	0.5	0.5	0.6
Intersection of Neild Ave and New South Head Rd	Aug 1983	0.45	0.3	0.4	0.5	0.5
Intersection of Neild Ave and New South Head Rd	Jan 1989	0.4	0.3	0.4	0.5	0.5
Intersection of Neild Ave and New South Head Rd	Jan 1991	0.4	0.3	0.4	0.5	0.5
Waratah St Low Point	Jan 1989	0.5	0.3	0.4	0.4	0.5

Given the lack of surveyed flood levels and the general paucity of detailed data the modelled results correspond reasonably well with anecdotal flooding observations and general catchment flow behaviour.

9. DESIGN FLOOD MODELLING

9.1. Critical Duration

To determine the critical storm duration for various parts of the catchment, modelling of the 100 Year ARI event was undertaken for a range of design storm durations from 15 minutes to 12 hours, using temporal patterns from Reference 6. 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.

The critical duration within the catchment varies. A significant portion of the catchment has a critical duration of 30 minutes, including along the majority of Barcom Avenue where flood levels vary by ±0.05 m for the range of durations. Along Boundary Street and McLachlan Avenue the critical duration was found to be 120 minutes, with flood levels varying by ±0.05 m generally. Along Victoria Street where the critical duration was found to be 60 minutes, with levels varying by up to 0.1 m for other durations. The difference between peak flood levels between the 60 minute and 120 minute duration event however was found to be less than ±0.02m. The 120 minute duration was assessed as the critical storm duration for the catchment generally, as even in upper catchment areas the flood levels were only slightly lower (within 0.05 m) than shorter durations.

9.2. Overview of Results

The results from this study are provided in the following outputs:

- Peak flood level profiles on Figure 15 to Figure 17,
- Peak flood depths and levels on Figure 18 to Figure 24,
- Provisional flood hazard on Figure 25 to Figure 28,
- Preliminary hydraulic categorisation on Figure 29 to Figure 32.

Results have been provided to Council in digital format compatible with Council's Geographic Information System (GIS).

9.3. Results at Key Locations

The results at key locations for peak flood flows, velocities, levels and depths are shown on Table 20 and Table 21 (refer to Figure 11 for locations).

Table 20 – Peak Flows (m^3/s) at Key Locations

ID	Location	Name	Type	2y ARI	5y ARI	10y ARI	20y ARI	50y ARI	100y ARI	PMF
1	Victoria Street U/S St Vincents Hospital	RB028	Overland	0.5	0.7	1.2	2.4	3.3	4.2	20.3
2	Barcom Street near Oxford St	RB027	Overland	0.0	0.0	0.0	0.0	0.1	0.1	2.6
		DRAP10737	Piped	0.9	1.2	1.2	1.2	1.3	1.4	2.0
		DRAP10760	Piped	0.5	0.6	0.7	0.7	0.7	0.6	1.0
5	Hopewell Street Near Oxford St	RB018	Overland	0.5	0.9	1.1	1.4	1.7	2.1	12.2
		DRAP11186	Piped	0.1	0.2	0.2	0.2	0.2	0.2	0.4
6	Boundary Street below Burton St	RB042	Overland	3.3	5.4	6.8	8.5	10.2	12.6	53.3
		DRAP10836B	Piped	1.9	2.1	2.1	2.2	2.2	1.6	2.6
7	Womerah Avenue	RB101	Overland	0.2	0.2	0.3	0.3	0.4	0.4	1.2
8	Boundary Street near Dillan St	RB048	Overland	5.4	9.1	11.2	13.8	16.5	19.9	82.6
		DRAP10660B	Piped	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		DRAP10791	Piped	2.5	2.8	2.9	3.1	3.2	2.5	4.0
9	McLachlan Ave (West)	RB099	Overland	3.0	5.1	6.2	7.6	9.0	10.8	39.5
		DRAP10807B	Piped	3.0	3.5	3.6	3.8	3.9	3.3	4.9
10	McLachlan Ave (East)	RB073	Overland	2.4	4.4	5.5	6.8	7.9	9.4	30.1
		DRAP10807D	Piped	3.6	4.0	4.2	4.4	4.5	3.9	5.2
11	Neild Ave D/S of Boundary Street	RB060	Overland	4.6	7.3	9.1	11.2	13.4	16.5	77.9
		DRAP10897	Piped	0.2	0.2	0.2	0.2	0.2	0.2	0.2
		DRAP11062	Piped	0.5	0.5	0.5	0.5	0.6	0.6	0.6
		DRAP11161	Piped	0.4	0.4	0.4	0.5	0.5	0.5	0.5
12	Roslyn Gardens	RB082	Overland	0.4	0.6	0.8	1.0	1.2	1.5	7.8
		DRAP14439A	Piped	0.0	0.1	0.1	0.1	0.2	0.2	0.3

Table 21 – Peak flood levels (m AHD) and depths (m) at key locations for all design events

ID	Location	2 year ARI		5 year ARI		10 year ARI		20 year ARI		50 year ARI		100 year ARI		PMF	
		Level	Depth	Level	Depth	Level	Depth	Level	Depth	Level	Depth	Level	Depth	Level	Depth
1	Sims Street	49.0	0.6	49.2	0.8	49.2	0.8	49.3	0.9	49.4	1.0	49.5	1.1	49.7	1.4
2	Oxford Street (West)	63.5	0.4	63.7	0.6	63.8	0.7	63.9	0.8	64.0	0.9	64.1	1.0	64.7	1.7
3	Victoria Street	61.8	1.4	62.2	1.7	62.2	1.7	62.2	1.7	62.2	1.7	62.2	1.8	62.6	2.2
4	Taylor Street	47.0	0.3	47.1	0.4	47.1	0.4	47.2	0.5	47.2	0.5	47.2	0.5	48.2	1.5
5	Sturt Street	46.4	1.4	46.5	1.6	46.6	1.6	46.6	1.6	46.6	1.7	46.7	1.7	47.1	2.2
6	Victoria St adjacent St Vincents Hospital	43.9	0.4	44.1	0.5	44.3	0.7	44.6	1.1	44.8	1.2	44.9	1.3	45.4	1.8
7	Boundary Street	12.8	0.4	12.8	0.5	12.8	0.5	12.9	0.5	12.9	0.6	13.0	0.6	13.7	1.4
8	McLachlan Ave	6.2	0.5	6.3	0.6	6.3	0.7	6.4	0.7	6.4	0.8	6.5	0.8	7.0	1.4
9	Neild Ave and New South Head Rd	4.8	0.2	4.9	0.2	4.9	0.2	4.9	0.3	5.0	0.3	5.0	0.3	5.3	0.7
10	Kellett Place	33.0	0.7	33.1	0.7	33.1	0.7	33.2	0.8	33.2	0.8	33.2	0.8	33.6	1.3
11	Waratah Street	2.3	0.3	2.4	0.3	2.4	0.4	2.5	0.4	2.5	0.5	2.5	0.5	2.8	0.8

9.4. Provisional Flood Hazard and Preliminary True Hazard

Maps of provisional hydraulic hazard are presented on Figure 25 (10 Year ARI) to Figure 28 (PMF). Hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual (Reference 15).

The provisional hazards were reviewed in this study to consider other factors such as rate of rise of floodwaters, duration, threat to life, danger and difficulty in evacuating people and possessions and the potential for damage, social disruption and loss of production. These factors and related comments are given in Table 22.

Table 22: Weightings for Assessment of True Hazard

Criteria	Weight [1]	Comment
Rate of Rise of Floodwaters	High	The rate of rise in the creek channels and onset of overland flow along roads would be very rapid, which would not allow time for residents to prepare.
Duration of Flooding	Low	The duration for local catchment flooding will generally be less than around 6 hours, resulting in inconvenience to affected residents but not generally a significant increase in hazard.
Effective Flood Access	High	Roads within the catchment will generally be inundated prior to property inundation, which may restrict vehicular access during a flood.
Size of the Flood	Moderate	The hazard can change significantly at some locations with the magnitude of the flood, particularly in the residential areas near Sims, Taylor and Sturt Streets and along Oxford Street. However, these higher hazard areas are generally captured by mapping a range of events using the provisional hazard criteria.
Effective Warning and Evacuation Times	High	There is very little, if any, warning time. During the day residents will be aware of the heavy rain but at night (if asleep) residential and non-residential building floors may be inundated with no prior warning.
Additional Concerns such as Bank Erosion, Debris, Wind Wave Action	Low	The main concern would be debris blocking culverts or bridges. This is considered to have a high probability of occurrence and will significantly increase the hazard. There is also the possibility of vehicles being swept into the main channels (as occurred in Newcastle in June 2007) causing blockage. However design modelling for this study includes significant blockage and the provisional hazard classification therefore includes this factor. Wind wave action is unlikely to be an issue but waves from traffic may be, due to the proximity of flood prone properties to main traffic routes.
Evacuation Difficulties	Low	Given the quick response of the catchment evacuation is not considered to be necessary (it is safer to remain than to cross fast flowing floodwaters) except in a few instances and therefore was not given significant weight for assessing true hazard.
Flood Awareness of the Community	Low	The flood awareness of the community is quite high due to the frequency of recent flood events. As a result of this awareness of problem flood areas, this factor is assigned a low weight in assessing true flood hazard.
Depth and Velocity of Floodwaters	High	In areas of overland flow roads are subject to fast flowing water. There is always a risk of a car or pedestrian being swept into flood waters. However this factor is largely included in the provisional hydraulic hazard calculation metrics.

Note: [1] Relative weighting in assessing the preliminary true hazard.

For the Rushcutters Bay catchment within the City of Sydney LGA, the factors with high weighting in relation to assessment or true hazard are generally related to the lack of flood warning, and the potential for flooding of access to residential properties prior to above-floor flooding of buildings occurring. In most cases, it is likely that remaining inside the property will present less risk to life than attempting evacuation via flooded routes, as refuge can generally be taken upstairs, or on furniture etc. There may be some properties where remaining inside would present a high risk to life due to very high flood depths, but these properties will generally already be classified as high hazard using provisional hazard criteria.

In general it was found that areas where a high flood hazard would be justified based on consideration of the high weight criteria in Table 22, the area was already designated high hazard as a result of the depth/velocity criteria used to develop the provisional hazard. However, additional information (particularly detailed flood level survey) may warrant revision of the true hazard categories at various properties during the Floodplain Risk Management Study phase.

9.5. Preliminary Hydraulic Categorisation

Preliminary hydraulic categorisations for the 10, 20, 100 year ARI and PMF events are provided on Figure 29 to Figure 32. 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 catchment in question.

For this study, preliminary hydraulic categories were defined using the approach adopted in Howells et al (Reference 16) and the following criteria were applied:

- 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 > 0.15m

  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 < 0.5m.

9.6. Preliminary Flood ERP 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).

Table 23 (taken from Reference 17) provides an indication of the response required for areas with different classifications. However, these may vary depending on local flood characteristics and resultant flood behaviour i.e. in flash flooding or overland flood areas. The criteria for classification of floodplain communities outlined in Reference 17 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.

Table 23: 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
Areas 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

In urban areas like the Rushcutters Bay catchment, flash flooding from local catchment and overland flow will generally occur as a direct response to intense rainfall without significant warning. At most flood affected properties in the catchment, remaining inside the home or building is likely to present less risk to life than attempting to drive or wade through floodwaters, as flow velocities and depths are likely to be greater in the roadway.

Figure 33 shows the preliminary ERP classification within the study area. A large proportion of the study area has been classified as high flood island, due to the reasonably high depths that would occur in road reserves surrounding properties, prior to inundation of the properties themselves.

10. SENSITIVITY ANALYSIS

10.1. Overview

Due to lack of historical data suitable for undertaking a thorough model calibration, a number of assumptions have been made for the selection of the design approach/parameters, primarily relying on default parameter values or values used in similar studies. The following sensitivity analyses were undertaken for the 100 Year ARI event to establish the variation in design flood level that may occur if different assumptions were made:

- Rainfall Losses: Varying rainfall losses in the hydrologic model were assessed;
- Impervious Percentage: Changed the impervious fraction of each hydrologic sub-catchment by ±20%;
- Manning's "n": The roughness values were increased and decreased by 20% at all locations;
- Inflows / Climate Change: Sensitivity to rainfall/runoff estimates was assessed by increasing the rainfall intensities by 10%, 20% and 30% as recommended under current guidelines. Sea Level Rise scenarios for 2050 and 2100 were considered. Refer to Section 10.3 below for discussion;
- Pipe Blockage: Sensitivity of blocking all pipes by 25% and 50% were considered.

It should be noted that the parameters are not independent and adjustment of one parameter (Manning's "n") would generally require adjustment of other values (such as inflows) in order for the model to produce the same level at a given location.

10.2. Results of Sensitivity Analyses

Table 24 and Table 25 on the following page provide a summary of peak flood level changes at various locations for the sensitivity scenarios. Overall results were shown to be relatively insensitive to routing, roughness and blockage with results tending to be ± 0.05 m which can generally be accommodated within the 0.5 m freeboard applied to the 100 Year ARI results to determine the Flood Planning Levels (FPLs).

The sensitivity testing thus provides confidence that provided the model emulates ground conditions and hydraulic structures, within a range of typical values for parameters, the model will produce reasonably accurate and reliable design flood levels.

Table 24 – Results of Sensitivity Analyses – 100 Year ARI Event Flows (m^3/s)

ID	Location	100 Year ARI Peak Flood Flow (m^3/s)	Imperviousness increased by 20%	Imperviousness decreased by 20%	AMC = 1	AMC = 4	Soil = 1	Roughness increased by 20%	Roughness decreased by 20%	Blockage 25%	Blockage 50%
			Difference with 100 Year ARI base case (m^3/s)								
1	Victoria Street U/S St Vincents Hospital	4.2	-0.2	-0.3	-0.1	0.1	0.0	-0.1	0.0	0.4	0.6
2	Barcom Street near Oxford St	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		1.4	0.0	-0.1	-0.1	0.0	0.0	-0.1	0.0	-0.4	-0.7
		0.6	0.0	0.2	0.2	0.0	0.0	0.2	0.0	0.0	0.0
3	Hopewell Street Near Oxford St	2.1	-0.1	-0.1	-0.4	0.0	0.0	-0.1	0.0	0.0	0.0
		0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.1
4	Boundary Street below Burton St	12.6	-0.3	-1.0	-1.4	0.3	-0.1	-0.8	0.0	0.0	-0.1
		1.6	0.0	0.7	0.7	0.0	0.0	0.8	0.0	-0.3	-0.6
5	Womerah Avenue	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	Boundary Street near Dillan St	19.9	-0.3	-1.3	-1.4	0.4	-0.1	-1.1	0.0	0.2	0.4
		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		2.5	0.0	0.7	0.7	0.0	0.0	0.8	0.0	-0.5	-1.1
7	McLachlan Ave (West)	10.8	-0.1	-0.6	-0.8	0.2	-0.1	-1.1	0.0	0.1	0.4
		3.3	0.0	0.7	0.7	0.0	0.0	0.8	0.0	-0.6	-1.4
8	McLachlan Ave (East)	9.4	-0.1	-0.5	-0.6	0.1	-0.1	-1.1	0.0	0.2	0.7
		3.9	0.0	0.7	0.6	0.0	0.0	0.8	0.0	-0.7	-1.9
9	Neild Ave D/S of Boundary Street	16.5	-0.4	-0.9	-1.3	0.3	-0.5	-0.2	0.0	0.3	0.7
		0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1
		0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1
		0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	-0.2
10	Roslyn Gardens	1.5	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.1

Note: a dash (-) indicates no significant change

Table 25 – Results of Sensitivity Analyses – 100 Year ARI Event Depths (m)

ID	Location	100 Year ARI Peak Flood Depth (m)	Imperviousness increased by 20%	Imperviousness decreased by 20%	AMC = 1	AMC = 4	Soil = 1	Roughness increased by 20%	Roughness decreased by 20%	Blockage 25%	Blockage 50%
			Difference with 100 Year ARI base case (m)								
1	Sims Street	1.1	-	-	-	-	-	-	-	-	0.02
2	Oxford Street (West)	1.0	-0.03	-0.04	-0.02	-	-	-	-	0.13	0.22
3	Victoria Street	1.8	-	-	-	-	-	-	-	-0.01	-0.03
4	Taylor Street	0.5	-	-	-	-	-	-	-	0.02	0.04
5	Sturt Street	1.7	-	-	-	-	-	-	-	0.01	0.03
6	Victoria St adjacent St Vincents Hospital	1.3	-0.02	-0.03	-	-	-	-	-	0.05	0.09
7	Boundary Street	0.6	-	-0.02	-0.02	-	-	0.04	-	-	-
8	McLachlan Ave	0.8	-	-0.02	-0.02	-	-	-	-	-	0.03
9	Neild Ave and New South Head Rd	0.3	-	-	-	-	-	-	-	-	0.02
10	Kellett Place	0.8	-	-	-	-	-	-	-	-	-
11	Waratah Street	0.5	-	-	-	-	-	-	-	0.03	0.05

Note: a dash (-) indicates no significant change

10.3. Climate Change

10.3.1. 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 (Reference 18).

Any change 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 (Reference 19). 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 Rushcutters Bay catchment under warmer climate scenarios.

In light of this uncertainty, the NSW State Government advice (Reference 18) 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.

10.3.2. Sea Level Rise

In October 2009 the NSW Government issued its Policy Statement on Sea Level Rise (Reference 20) which states”

“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.

Sea level rise is an incremental process and will have medium to long-term impacts. The best national and international projections of sea level rise along the NSW coast are for a rise relative to 1990 mean sea levels of 40 cm by 2050 and 90 cm by 2100. However, the 4th Intergovernmental Panel on Climate Change in 2007 also acknowledged that higher rates of sea level rise are possible";

In August 2010, the former NSW Department of Environment, Climate Change and Water issued the Flood Risk Management Guide (Reference 10) – *Incorporating sea level rise benchmarks in flood risk assessments*. In addition an accompanying document *Derivation of the NSW Government's sea level rise planning benchmarks* provided technical details on how the sea level rise assessment was undertaken.

Although there are some minor variations in the sea levels predicted in these studies, policies, and guides, they all agree on an ocean level rise on the NSW coast of around 0.9 metre by the year 2100 relative to 1990 levels.

The previous guideline, the NSW Sea Level Rise Policy Statement (2009) (Reference 20) and associated guides, indicated a 0.9 metre sea level rise by the year 2100 and a 0.4 metre rise by the year 2050. It should be noted that climate change and the associated rise in sea levels will continue beyond 2100. Recent changes have taken away NSW State Government endorsement of sea level rise predictions. Unless Council adopts something else, a 0.9 metre sea level rise by the year 2100 and a 0.4 metre rise by the year 2050 will continue to be used.

10.3.3. Results

The effect of increasing the design rainfalls by 10%, 20% and 30% has been evaluated for the 100 year ARI event, resulting in a relatively insignificant impact on peak flood levels in the study area. Generally speaking, each incremental 10% increase in flow results in a 0.05 m increase in peak flood levels at most of the locations analysed. A 30% increase in rainfalls would therefore not exceed the typical freeboard for most residential properties.

The 100 year ARI event with a rainfall increase of 30% is approximately equivalent to a 500 year ARI event in present day conditions. In flow paths and trapped low points, flood levels were typically found to increase by 0.05 to 0.20 m.

Sea level rise scenarios have very little impact on flood levels within the catchment with a 0.9 m sea level increase by 2100 only increasing downstream flood levels within the Waratah Street low point adjacent to Rushcutters Bay Park by 0.05 m.

Table 26 and Table 27 show the change in peak flows and flood levels due to the effect of climate change induced rainfall increases and sea level rise.

Table 26 – Results of Climate Change Analyses – 100 Year ARI Event Flows (m^3/s)

ID	Location	100 Year ARI Peak Flood Flow (m^3/s)	Rainfall Increase 10%	Rainfall Increase 20%	Rainfall Increase 30%	Sea Level Rise 2050	Sea Level Rise 2100
			Difference with 100 Year ARI Base Case (m^3/s)				
1	Victoria Street U/S St Vincents Hospital	4.2	0.6	1.1	1.7	0.0	0.0
2	Barcom Street near Oxford St	0.1	0.0	0.0	0.1	0.0	0.0
		1.4	0.0	0.1	0.1	0.0	0.0
		0.6	0.0	0.0	0.1	0.0	0.0
3	Hopewell Street Near Oxford St	2.1	0.3	0.6	0.9	0.0	0.0
		0.2	0.0	0.0	0.0	0.0	0.0
4	Boundary Street below Burton St	12.6	1.5	3.0	4.5	0.0	0.0
		1.6	0.0	0.1	0.2	0.0	0.0
5	Womerah Avenue	0.4	0.0	0.1	0.1	0.0	0.0
6	Boundary Street near Dillan St	19.9	2.4	4.8	7.2	0.0	0.1
		0.0	0.0	0.0	0.0	0.0	0.0
		2.5	0.1	0.2	0.2	0.0	0.0
7	McLachlan Ave (West)	10.8	1.2	2.4	3.5	0.0	0.0
		3.3	0.1	0.2	0.3	0.0	0.0
8	McLachlan Ave (East)	9.4	0.9	1.7	2.6	0.0	0.0
		3.9	0.1	0.2	0.3	0.0	0.0
9	Neild Ave D/S of Boundary Street	16.5	2.4	4.6	6.9	0.0	0.0
		0.2	0.0	0.0	0.0	0.0	0.0
		0.6	0.0	0.0	0.0	0.0	0.0
		0.5	0.0	0.0	0.0	0.0	0.0
10	Roslyn Gardens	1.5	0.2	0.5	0.7	0.0	0.0
		0.2	0.0	0.0	0.0	0.0	0.0

Table 27 – Results of Climate Change Analyses – 100 Year ARI Event Depths (m)

ID	Location	100 Year ARI Peak Flood Depth (m)	Rainfall Increase 10%	Rainfall Increase 20%	Rainfall Increase 30%	Sea Level Rise 2050	Sea Level Rise 2100
			Difference with 100 Year ARI Base Case (m)				
1	Sims Street	1.1	0.01	0.03	0.06	-	-
2	Oxford Street (West)	1.0	0.10	0.16	0.21	-	-
3	Victoria Street	1.8	-	-	0.03	-	-
4	Taylor Street	0.9	0.02	0.04	0.05	-	-
5	Sturt Street	0.5	0.03	0.08	0.11	-	-
6	Victoria St adjacent St Vincents Hospital	1.7	0.02	0.05	0.07	-	-
7	Boundary Street	1.3	0.06	0.11	0.15	-	-
8	McLachlan Ave	0.6	0.03	0.06	0.09	-	-
9	Neild Ave and New South Head Rd	0.8	0.03	0.05	0.08	-	-
10	Kellett Place	0.3	0.02	0.04	0.05	-	-
11	Waratah Street	0.8	0.03	0.05	0.07	-	-
12	Sims Street	0.5	0.02	0.04	0.06	-	-

11. DAMAGES ASSESSMENT

The cost of flood damages and the extent of the disruption to the community depend upon many factors including:

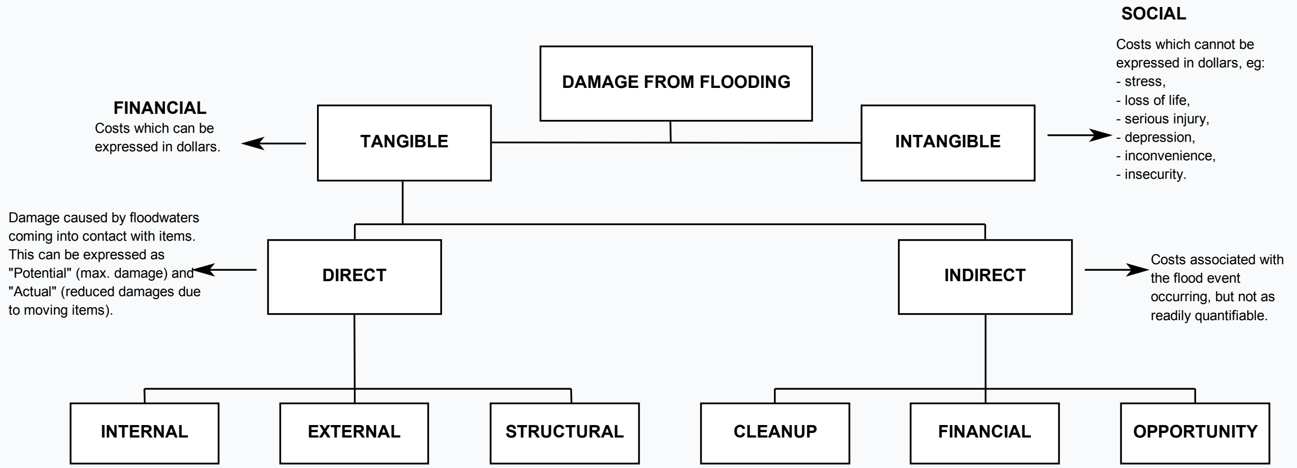
- the magnitude (depth, velocity and duration) of the flood,
- land usage and susceptibility to damage,
- awareness of the community to flooding,
- effective warning time,
- the availability of an evacuation plan or damage minimisation program,
- physical factors such as failure of services (pits and pipes), flood borne debris, sedimentation, and
- the types of asset and infrastructure affected.

The estimation of flood damages tends to focus on the physical impact of damages on the human environment but there is also a need to consider the ecological cost and benefits associated with flooding. Flood damages can be defined as being tangible or intangible. Tangible damages are those to which a monetary value cannot easily be attributed. Types of flood damages are shown on Table 28.

While the total likely damages in a given flood are useful to get a “feel” for the magnitude of the flood problem, it is of little value for absolute economic evaluation. When considering the economic effectiveness of a proposed mitigation measure, the key question is what are the total damages prevented over the life of the measure? This is a function not only of the high damages which occur in large floods but also of the lesser but more frequent damages which occur in small floods.

The standard way of expressing flood damages is in terms of average annual damages (AAD). AAD represents the equivalent average damages that would be experienced by the community on an annual basis, by taking into the account the probability of a flood occurrence. By this means, the smaller floods, which occur more frequently, are given a greater weighting than the rare catastrophic floods.

Table 28 – Breakdown of Flood Damages Categories



	INTERNAL	EXTERNAL	STRUCTURAL	CLEANUP	FINANCIAL	OPPORTUNITY
RESIDENTIAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Laundries, Caravans, Sheds, Tools, Gardens, Fences	Physical Damage to Buildings: Gyprock, Cupboards, Scour of Footings, Houses becoming buoyant (floating off footings)	Clean Carpets, Walls, Clothes; Re-instate Furniture; Remove Mud and Debris	Loss of wages, Living costs (temporary accomodation and food), Time to repair/replace damaged items	Not Applicable
RURAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Sheds (stables/barns), Machinery, Tools, Fences, Feed storage, Saddles, Crops &/or Stock, Irrigation Systems	Physical Damage to Structures: Damage to Homestead, Sheds, Access tracks, Protection levees	Clean Homestead and Out-buildings; Remove Debris; Dispose of affected crops &/or stock	Loss of Farm Production and Income, Re-instatement of Pastures, Supplementary feeding of stock (by hand or outside agistment), Stock movement/ transport, Living costs (temporary accomodation and food)	Sowing or harvesting of Crops, Sale of Stock (at depreciated value or dependent on market influences)
COMMERCIAL	Contents of Buildings: Products, Stock, Fittings, Tools, Machinery, Raw Materials	External Items: Vehicles, Machinery, Display, Raw Materials/Stockpiles, Fences	Physical Damage to Buildings	Dispose of damaged products, stock, materials; Cleaning and Re-instatement	Loss of Productivity and Income, Bank Interest Charges	Loss of existing &/or Potential Trade
PUBLIC AUTHORITIES	Contents of Public Buildings and Facilities	Public Property and Facilities: Parks, Signs, Machinery, Equipment	Physical Damage to Infrastructure: Electricity, Water, Telephone, Gas, Road & Rail Transport Links	Remove Mud & Debris from Facilities, Public & Private Property Repairs (temporary & permanent)	Disruption of Services, Community Service Relief Grants	Provision of Public Service

A flood damages assessment was undertaken for existing development for overland flooding within the Rushcutters Bay catchment. This was based on a detailed floor level survey which was undertaken for 138 properties (613 properties are flood affected in the PMF event). Only properties which have surveyed floor levels have been included in the flood damages assessment.

A number of properties within the study area have below ground floors or basement car parking. In the case of below ground floors it was assumed that 50% would be inhabited and the maximum depth of flooding would be 1m. For basement car parking, if water could access the car park damages were assumed to be $10,000 (assumed 50% have a car at a cost of $20,000 per car park).

Damages to public structures have not been assessed. A summary of flood damages for the catchment is provided in Table 29 and Table 30 and with the building floors inundated shown on Figure 34.

Table 29 – Summary of Properties Flooded Above Floor Level

Design Flood Event	Residential Properties Flooded Above Floor Level	Commercial Properties Flooded Above Floor Level	Total Properties Flooded Above Floor Level
2 Year ARI	20	21	41
5 Year ARI	28	24	52
10 Year ARI	30	25	55
20 Year ARI	32	29	61
50 Year ARI	32	30	62
100 Year ARI	33	31	64
PMF	59	46	105

Note: * Excludes all damages to public assets

Table 30 – Summary of Flood Damages

Design Flood Event	Residential Properties Tangible Flood Damages	Commercial Properties Tangible Flood Damages	Total Tangible Flood Damages*
2 Year ARI	$1,180,000	$1,290,000	$2,470,000
5 Year ARI	$1,480,000	$1,530,000	$3,010,000
10 Year ARI	$1,670,000	$1,680,000	$3,360,000
20 Year ARI	$1,870,000	$1,760,000	$3,630,000
50 Year ARI	$1,940,000	$1,990,000	$3,930,000
100 Year ARI	$2,080,000	$2,250,000	$4,330,000
PMF	$3,780,000	$3,840,000	$7,620,000
Average Annual Damages			$2,150,000

Note: * Excludes all damages to public assets

11.1. Limitations of Flood Damage Assessment in Rushcutters Bay

In most areas the extent of above floor inundation is difficult to accurately assess. The effect of buildings, sheds, fences and other structures can have a significant impact on the direction and depth of floodwaters. Also the exact location and level of all entry points to buildings is unknown.

It should be noted that the number of floors inundated in the smaller events (say up to the 10 year ARI) is probably over estimated compared to what has been observed in past events. It is unlikely that all above floor flooding during past events has been reported, and some properties may have localised features (such as solid brick walls) that prevent above-floor inundation from a certain direction. Additional inaccuracies may result from the estimation of flood levels which ultimately are based on the ALS ground survey (accuracy of approximately 0.2m or more on uneven surfaces).

12. ACKNOWLEDGEMENTS

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The assistance of the following in providing data and guidance to the study is gratefully acknowledged:

- City of Sydney;
- Office of Environment and Heritage;
- Residents of the City of Sydney within the study area; and
- Bureau of Meteorology.

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Figures

FIGURE 1
LOCALITY MAP

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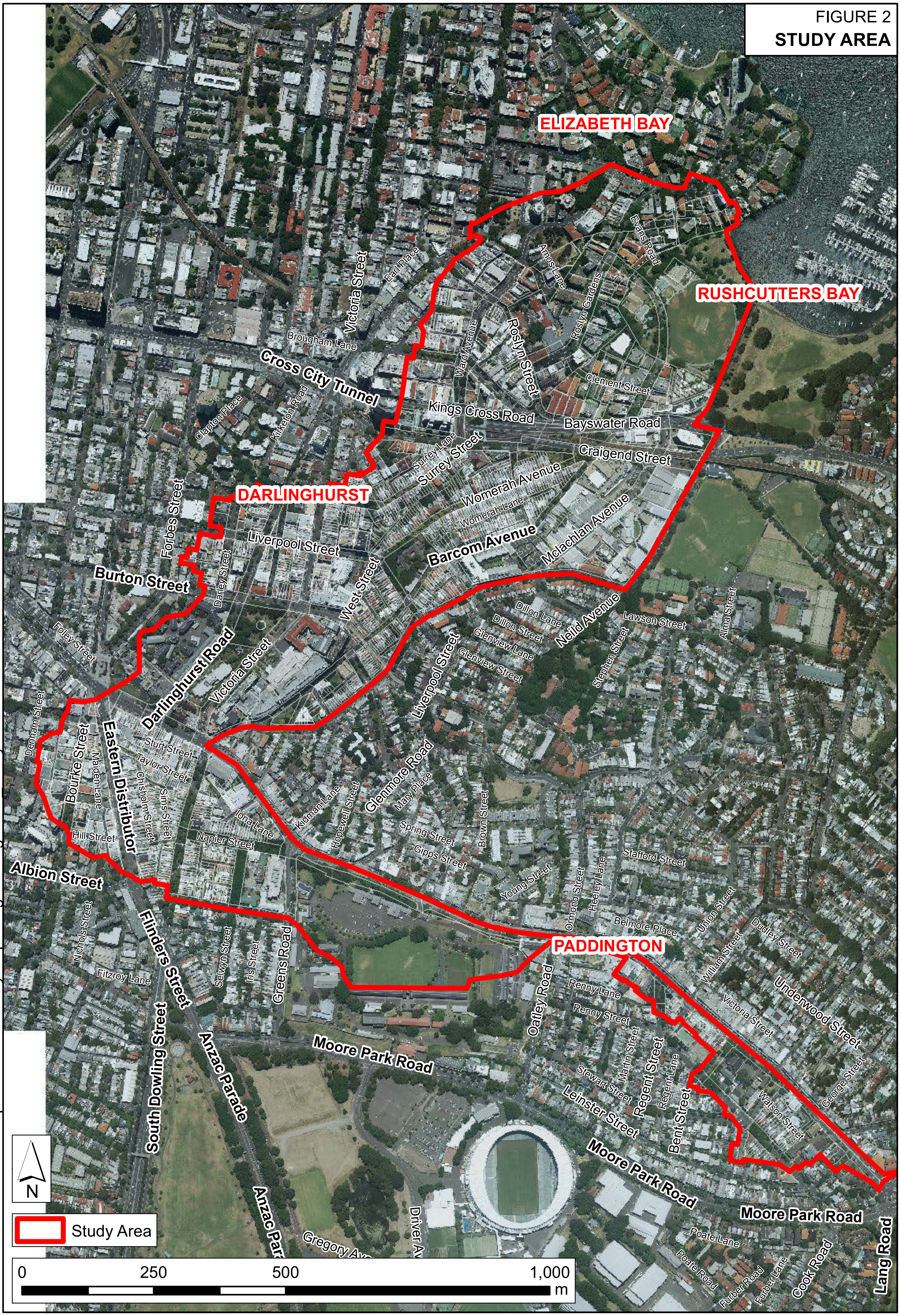
FIGURE 2
STUDY AREA



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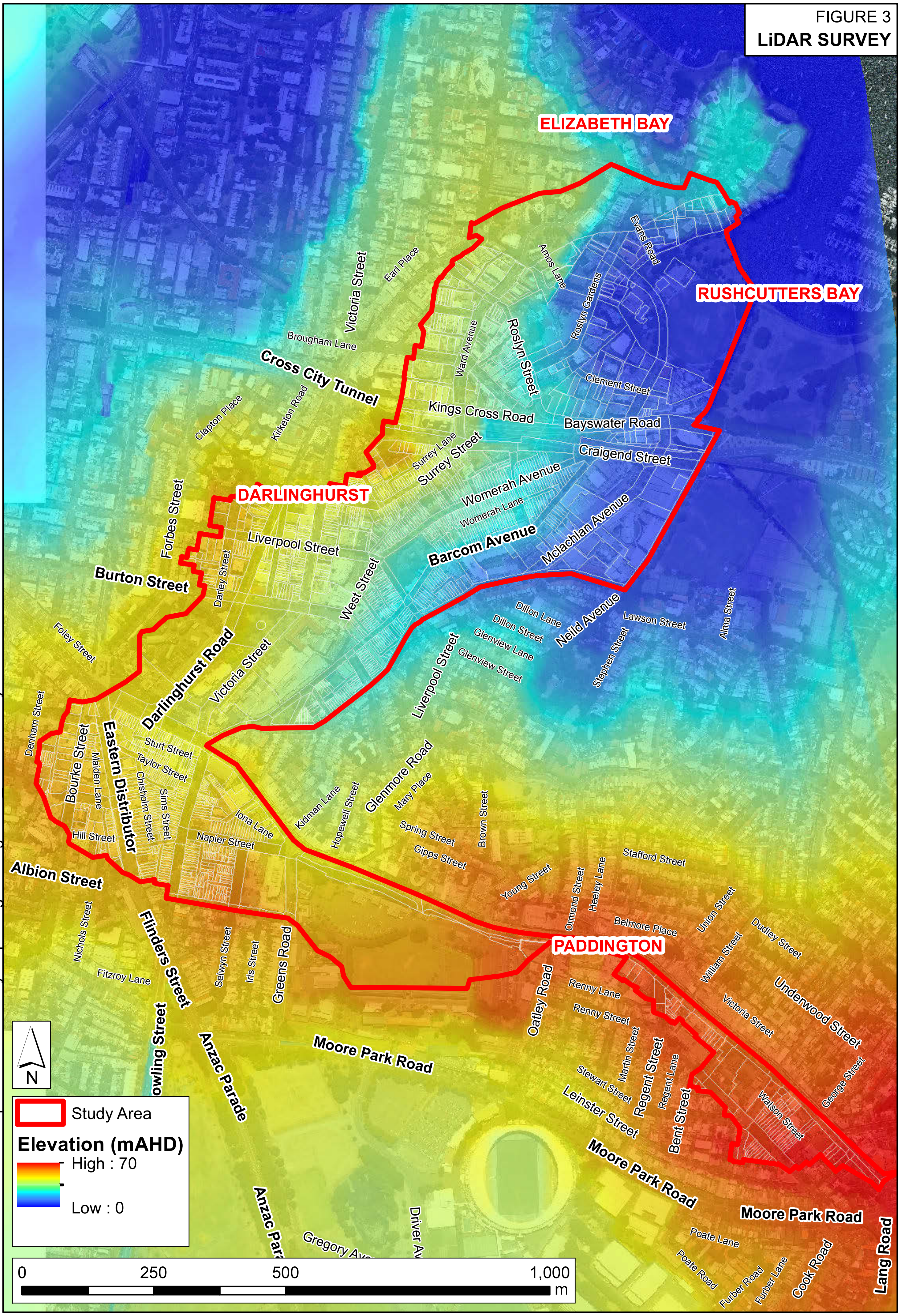

FIGURE 3
LiDAR SURVEY
ELIZABETH BAY
RUSHCUTTERS BAY
DARLINGHURST
PADDINGTON
Cross City Tunnel
Kings Cross Road
Bayswater Road
Craigend Street
Womerah Avenue
Barcom Avenue
Mclachlan Avenue
Liverpool Street
Burton Street
Darlinghurst Road
Victoria Street
Eastern Distributor
Albion Street
Flinders Street
Moore Park Road
Anzac Parade
Oatley Road
Underwood Street
Lang Road
Study Area
Elevation (mAHD)
High : 70
Low : 0
0
250
500
1,000
m
N
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FIGURE 4
RAINFALL GAUGES

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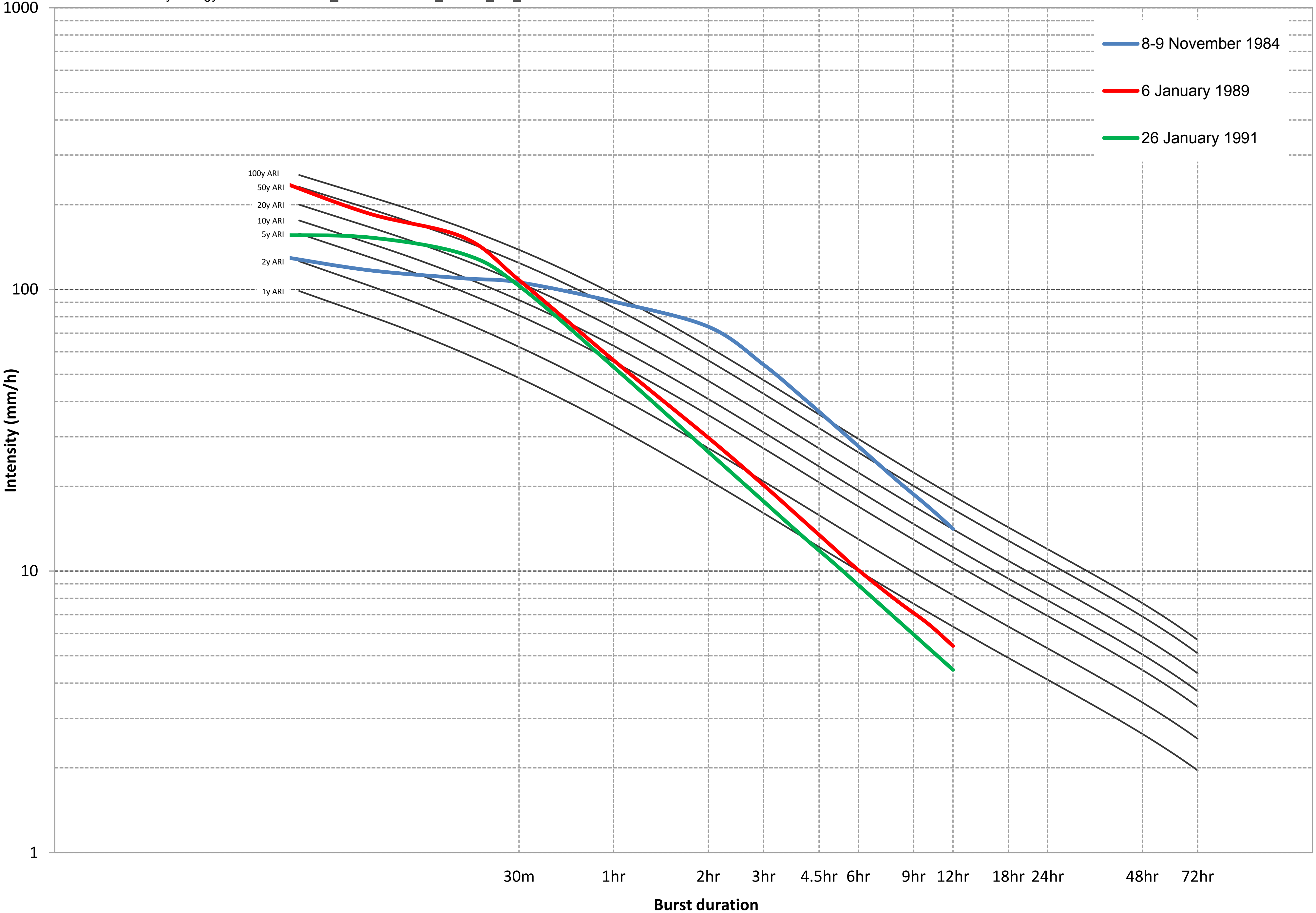
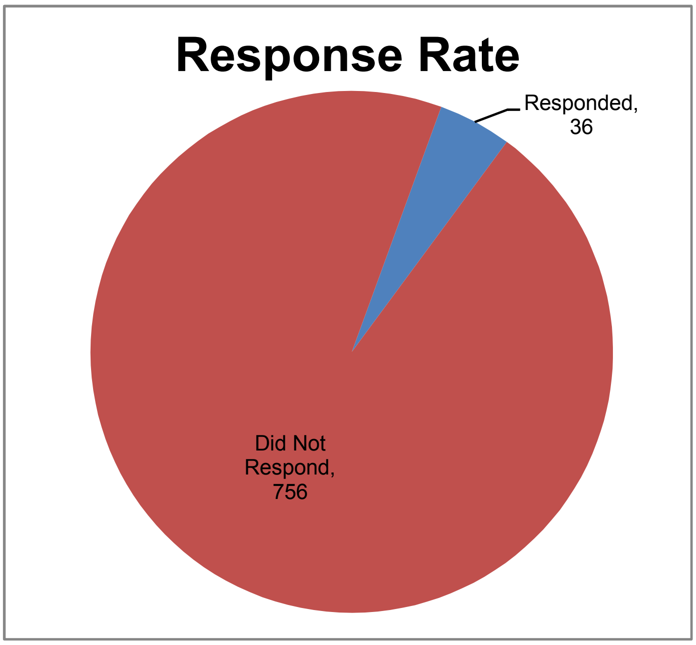
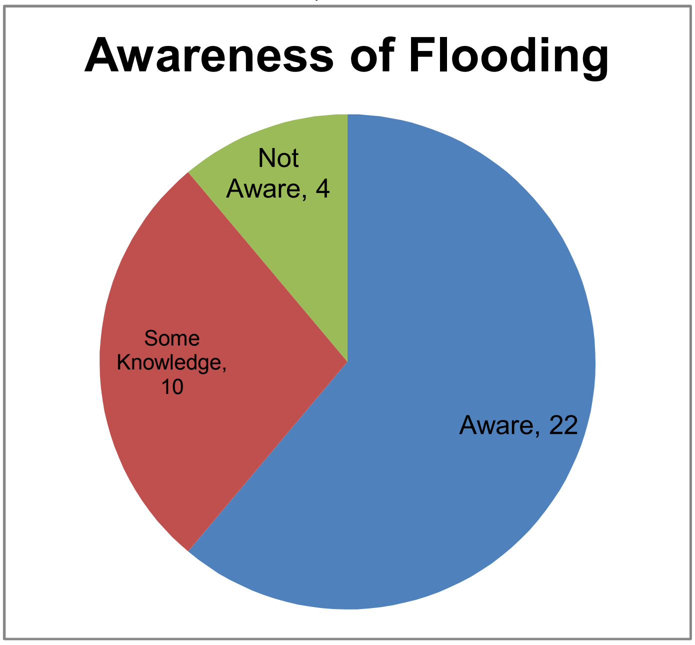
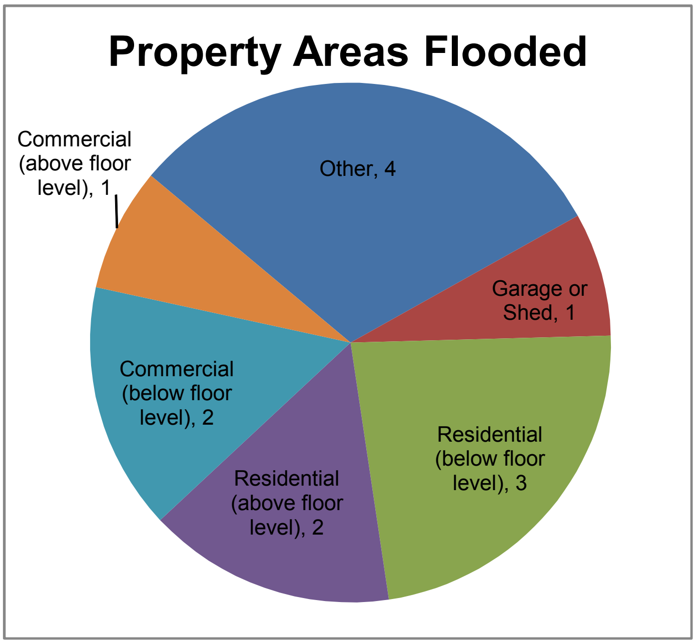


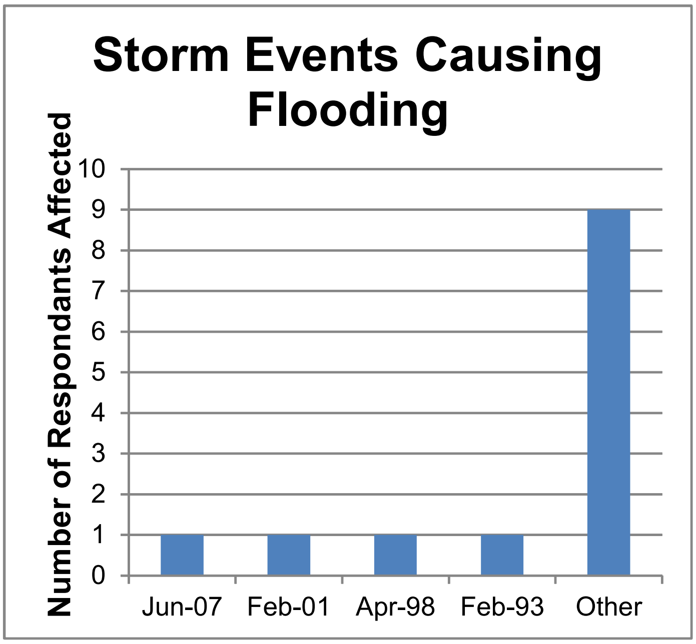
FIGURE 5
IFD DATA AND RAINFALL COMPARISON
PADDINGTON GAUGE

FIGURE 6
QUESTIONNAIRE RESULTS









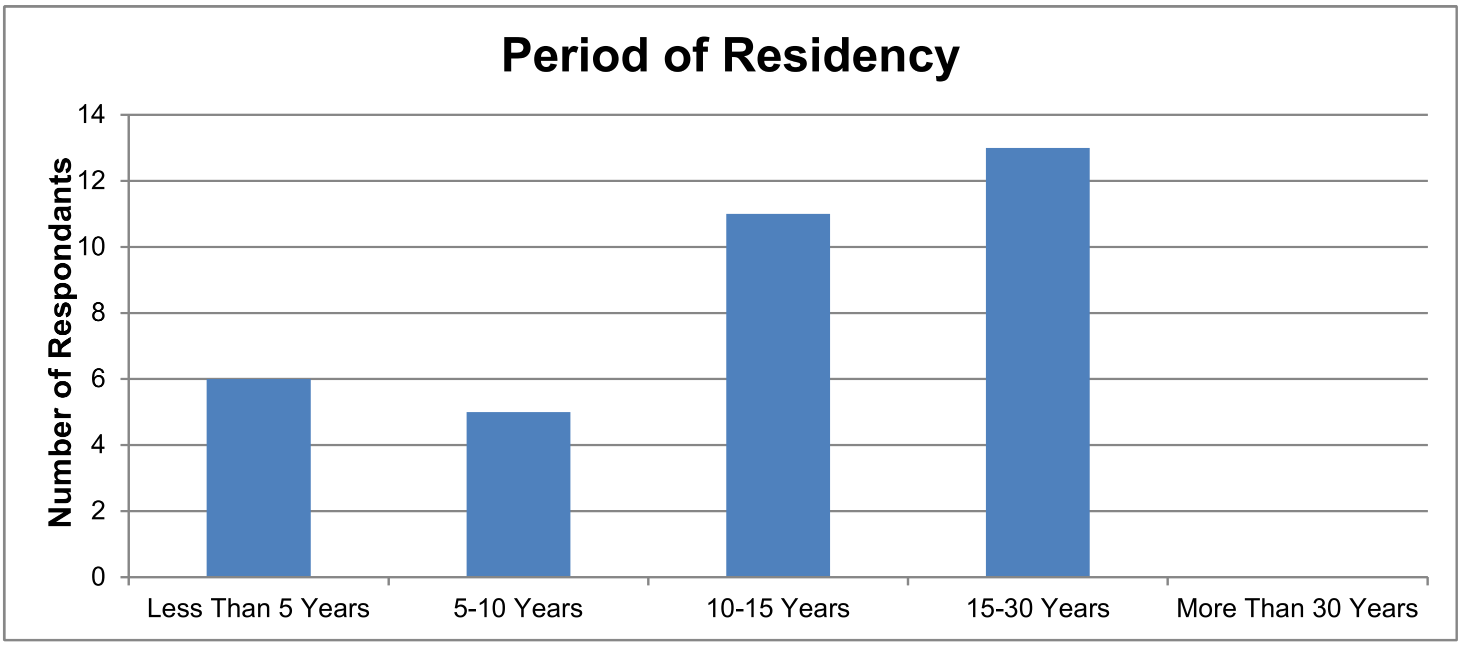


FIGURE 7

COMMUNITY CONSULTATION RESPONSE LOCATIONS

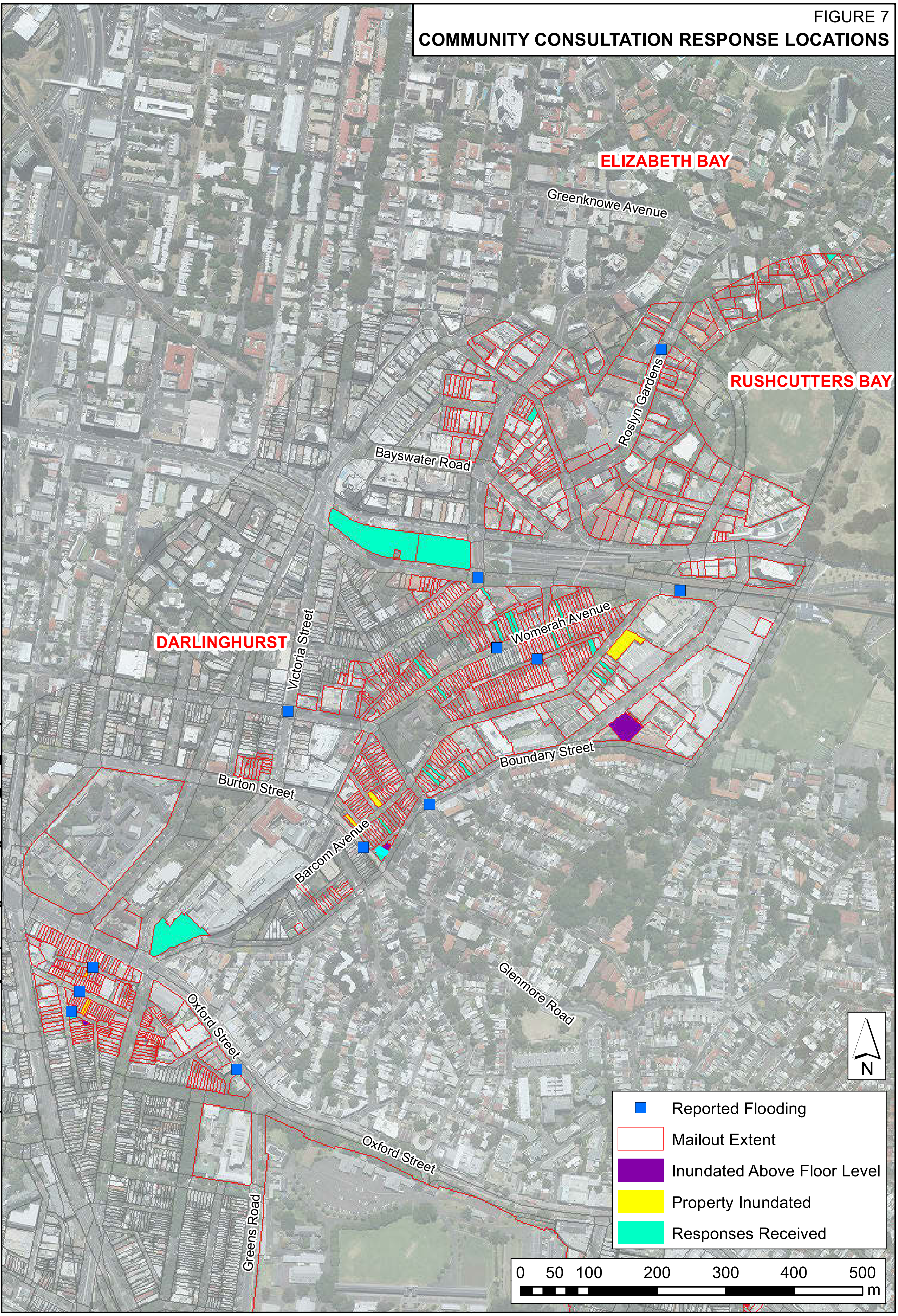


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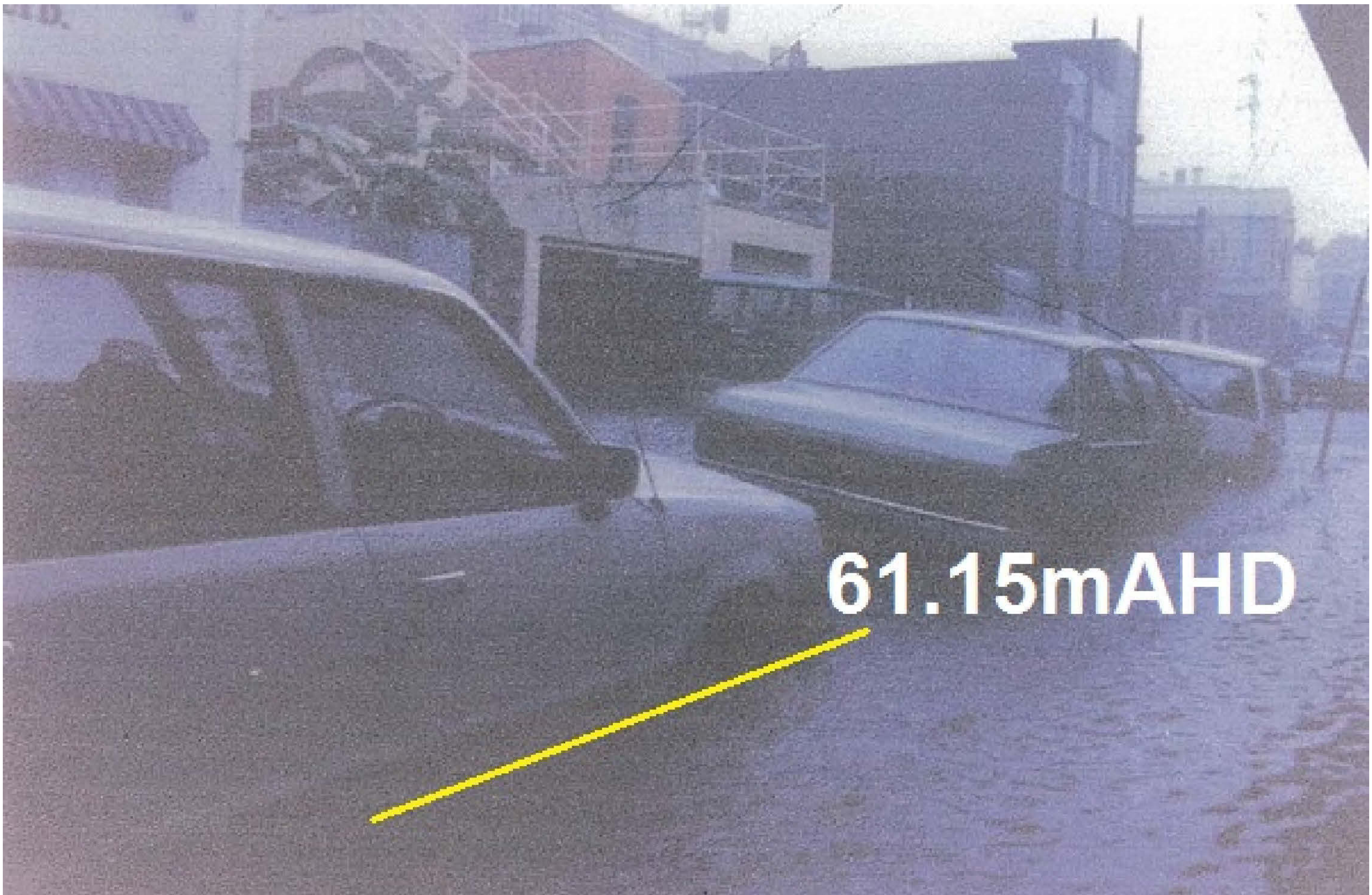
FIGURE 8
FLOODING PICTURES

Flooding in Victoria Street, Paddington on January 6, 1989. This location is immediately outside the study area, though indicates the downstream flooding within the same catchment.



Flooding in Victoria Street, Paddington on January 6, 1989, showing the approximate elevation of flood waters.

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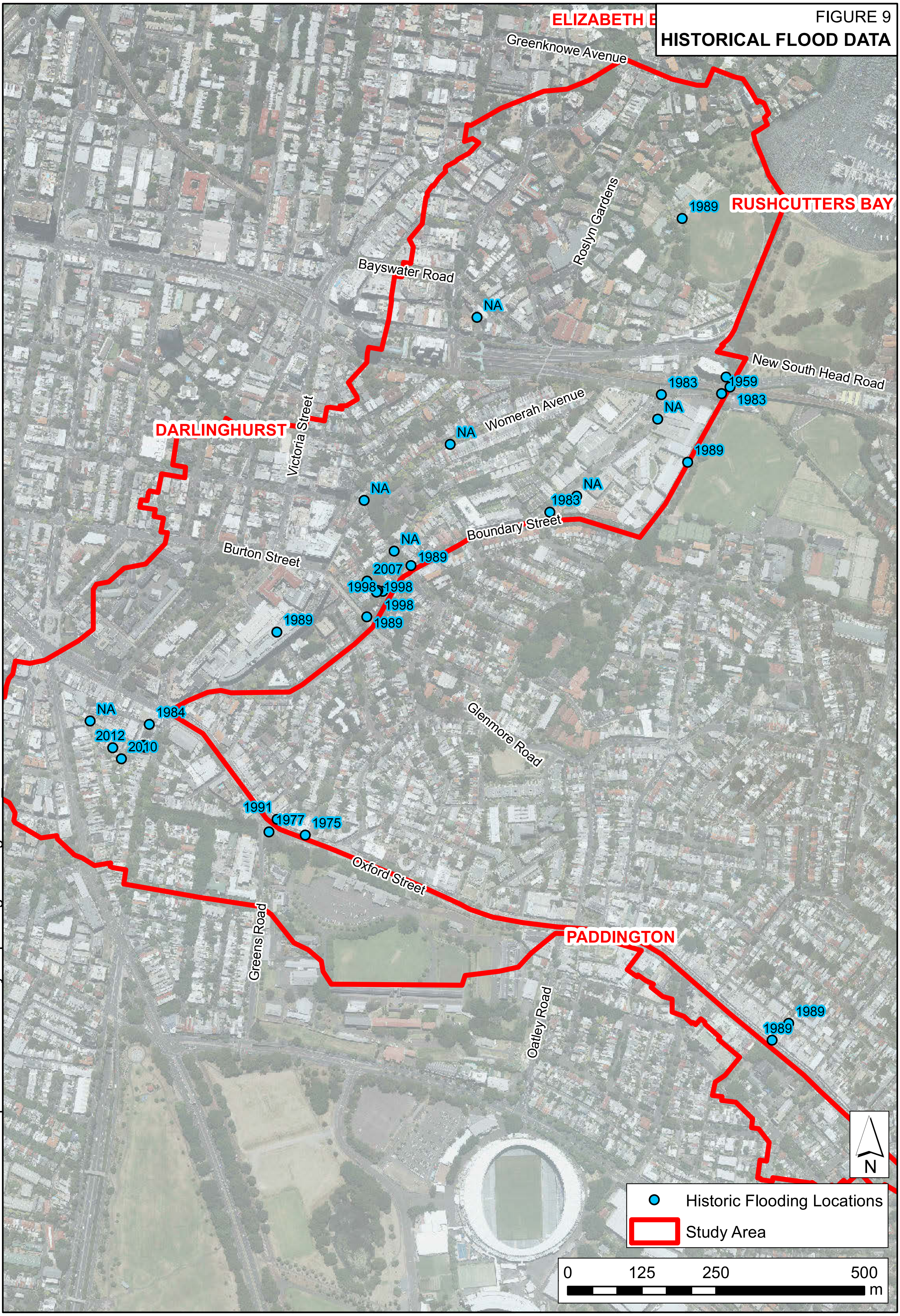

FIGURE 9
HISTORICAL FLOOD DATA

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FIGURE 10
HYDROLOGIC MODEL CATCHMENT LAYOUT

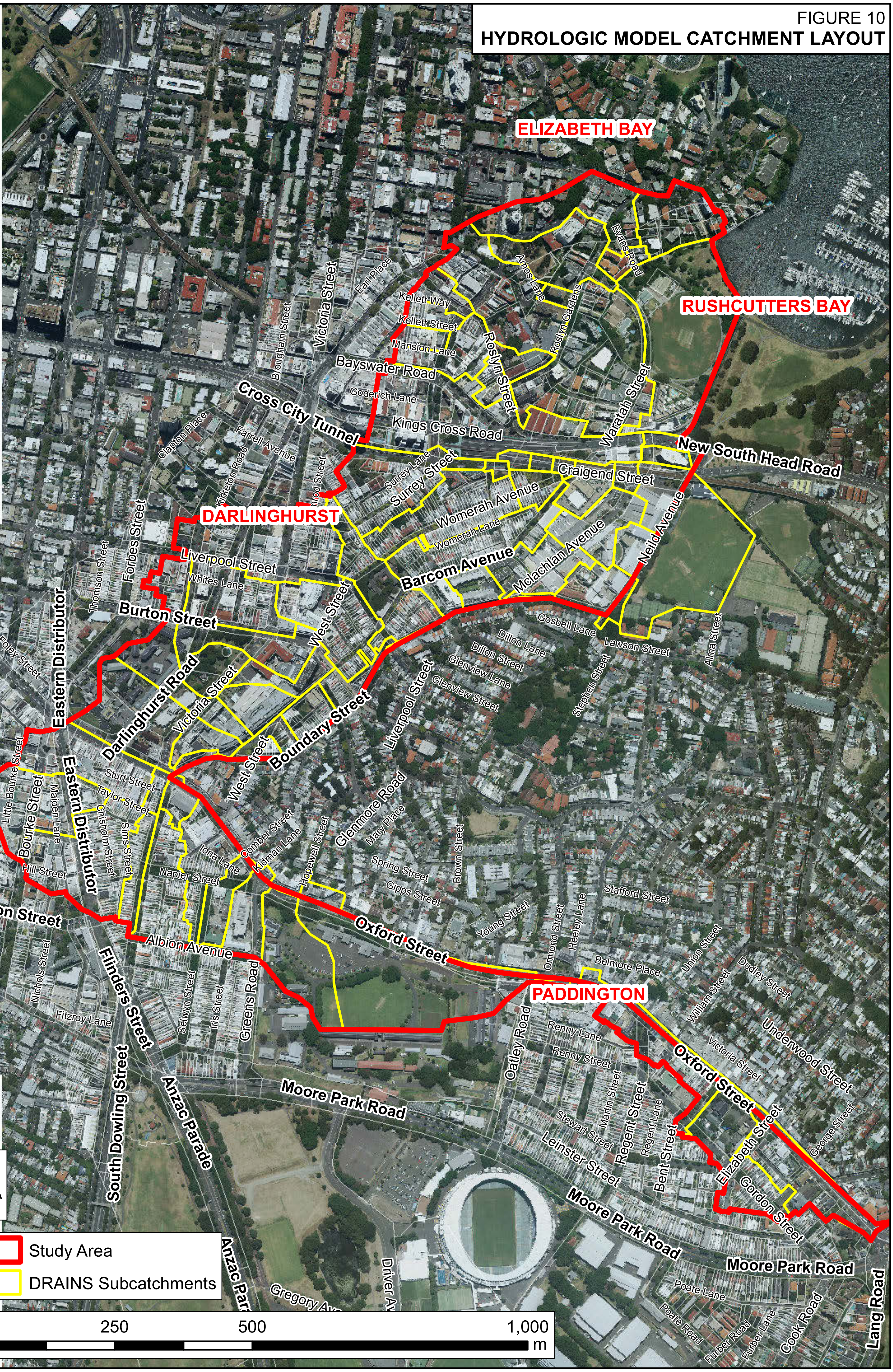

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FIGURE 11
HYDRAULIC MODEL CATCHMENT LAYOUT

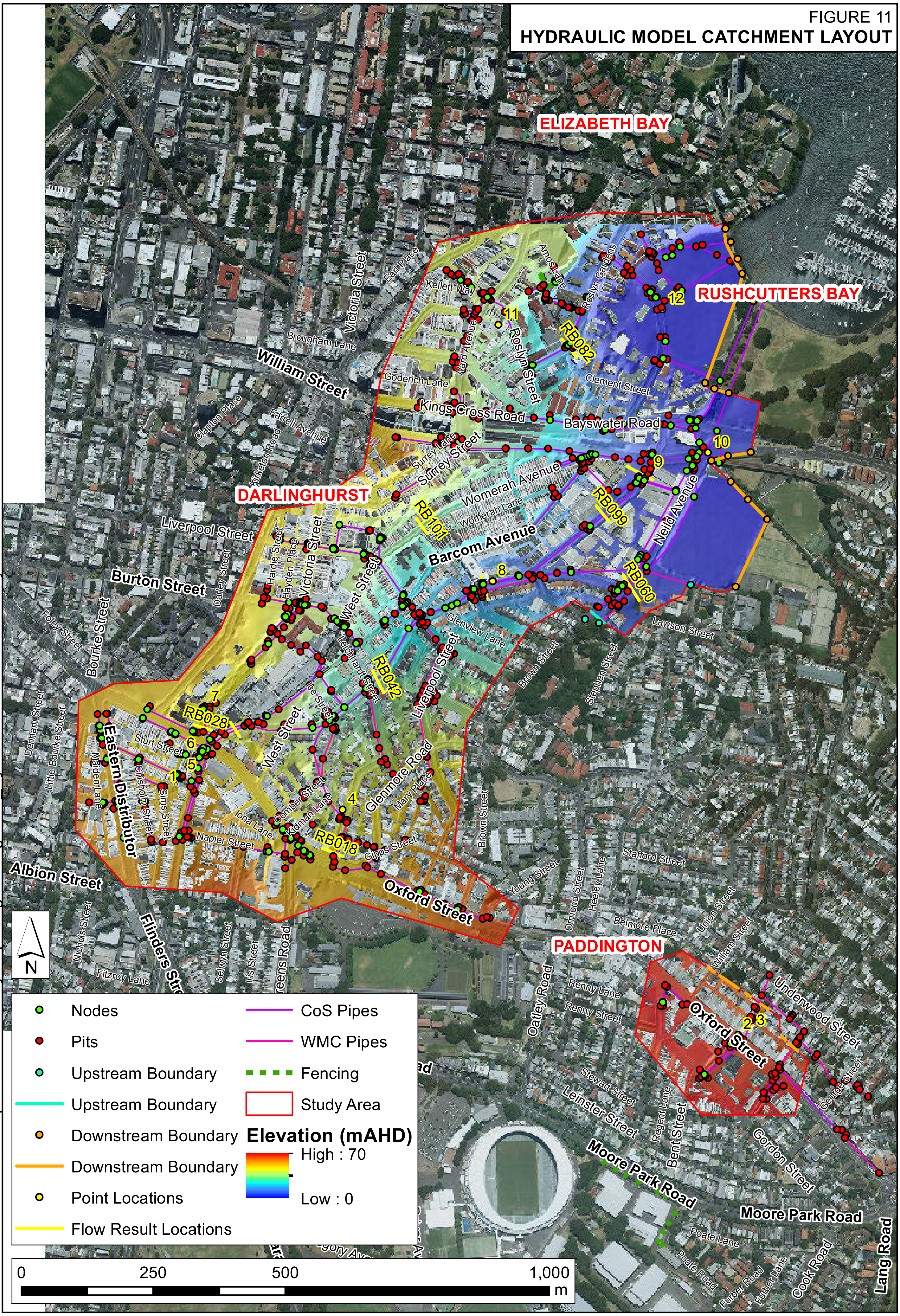

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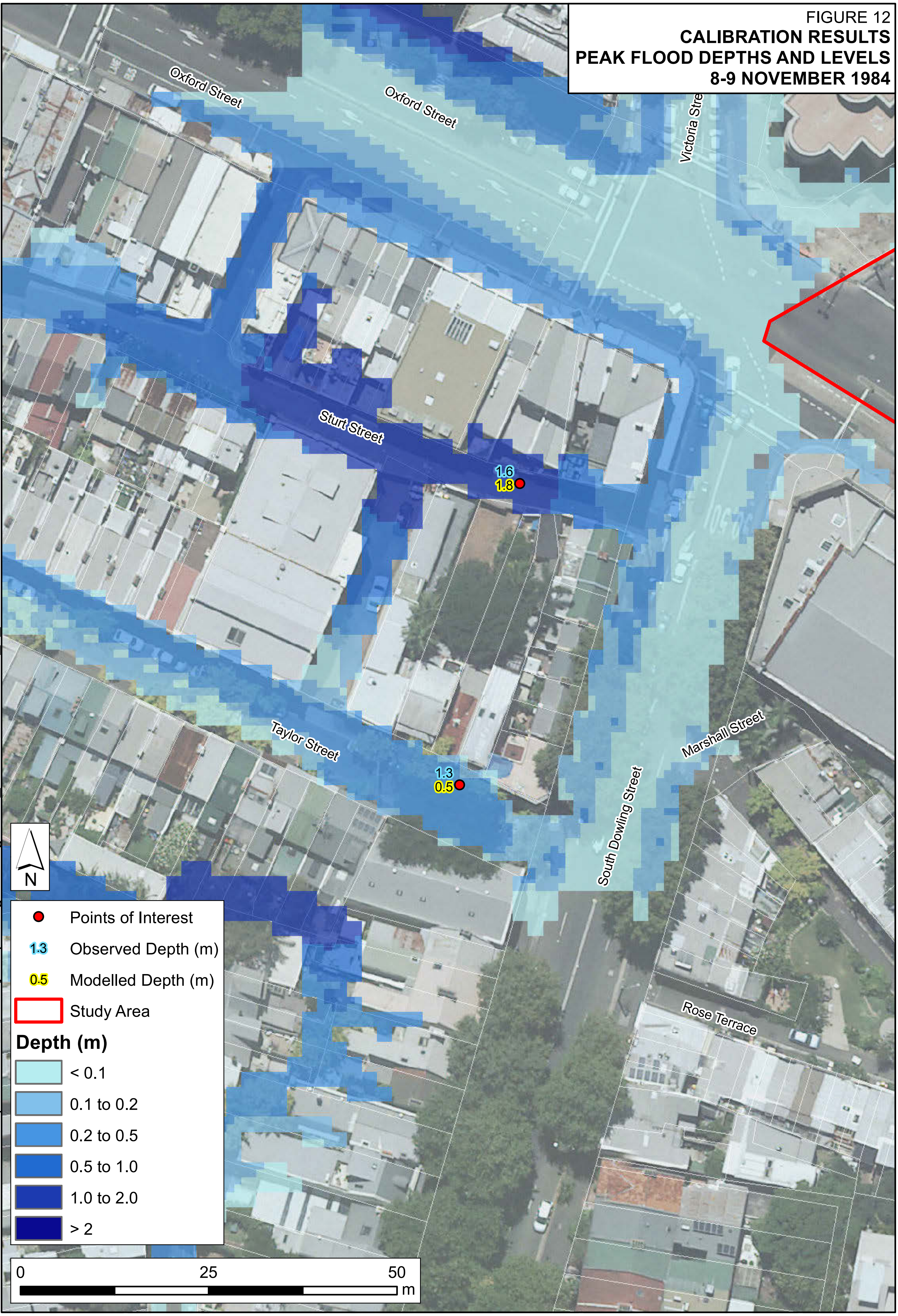
FIGURE 12
CALIBRATION RESULTS PEAK FLOOD DEPTHS AND LEVELS 8-9 NOVEMBER 1984



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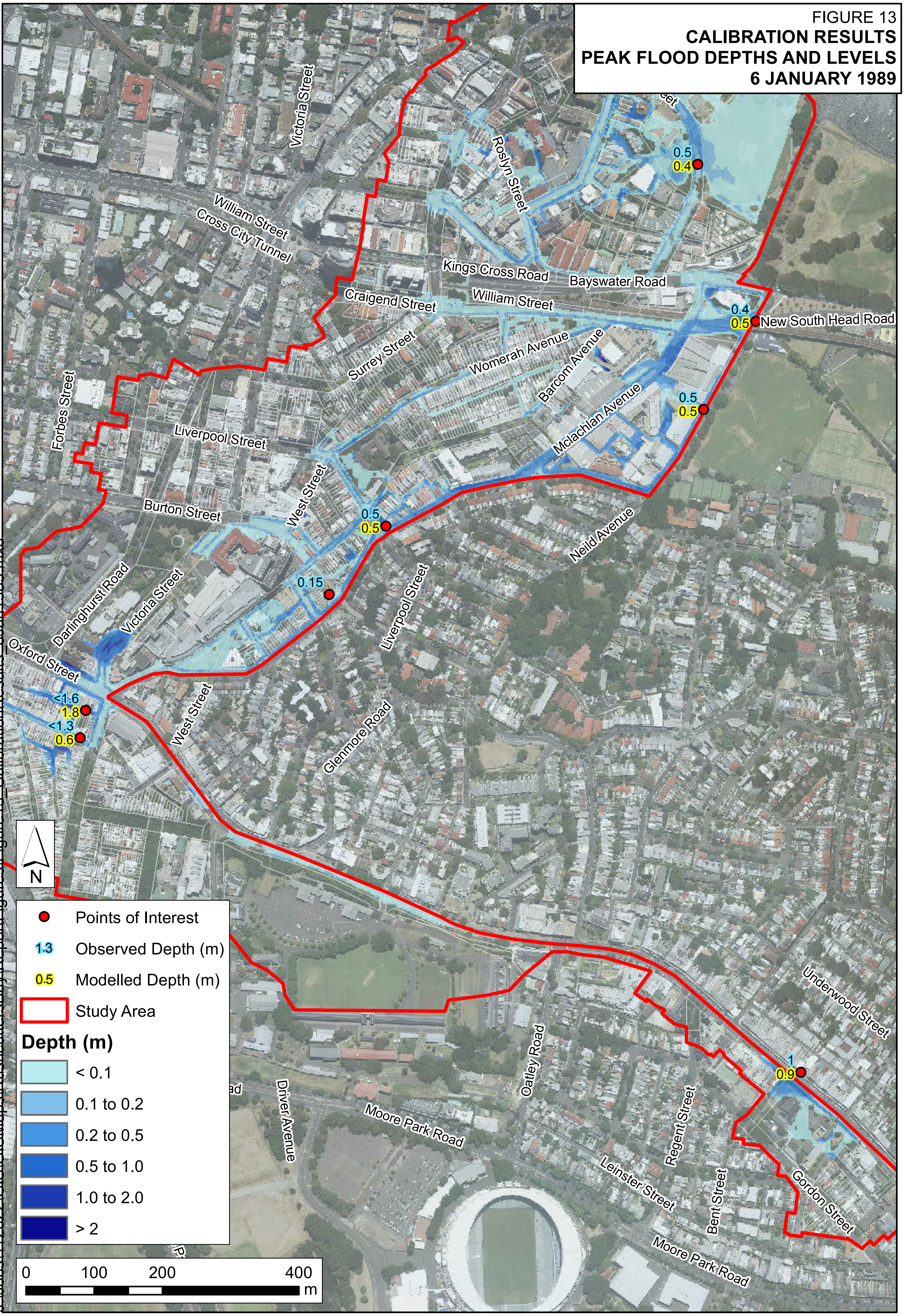

FIGURE 13
CALIBRATION RESULTS
PEAK FLOOD DEPTHS AND LEVELS
6 JANUARY 1989

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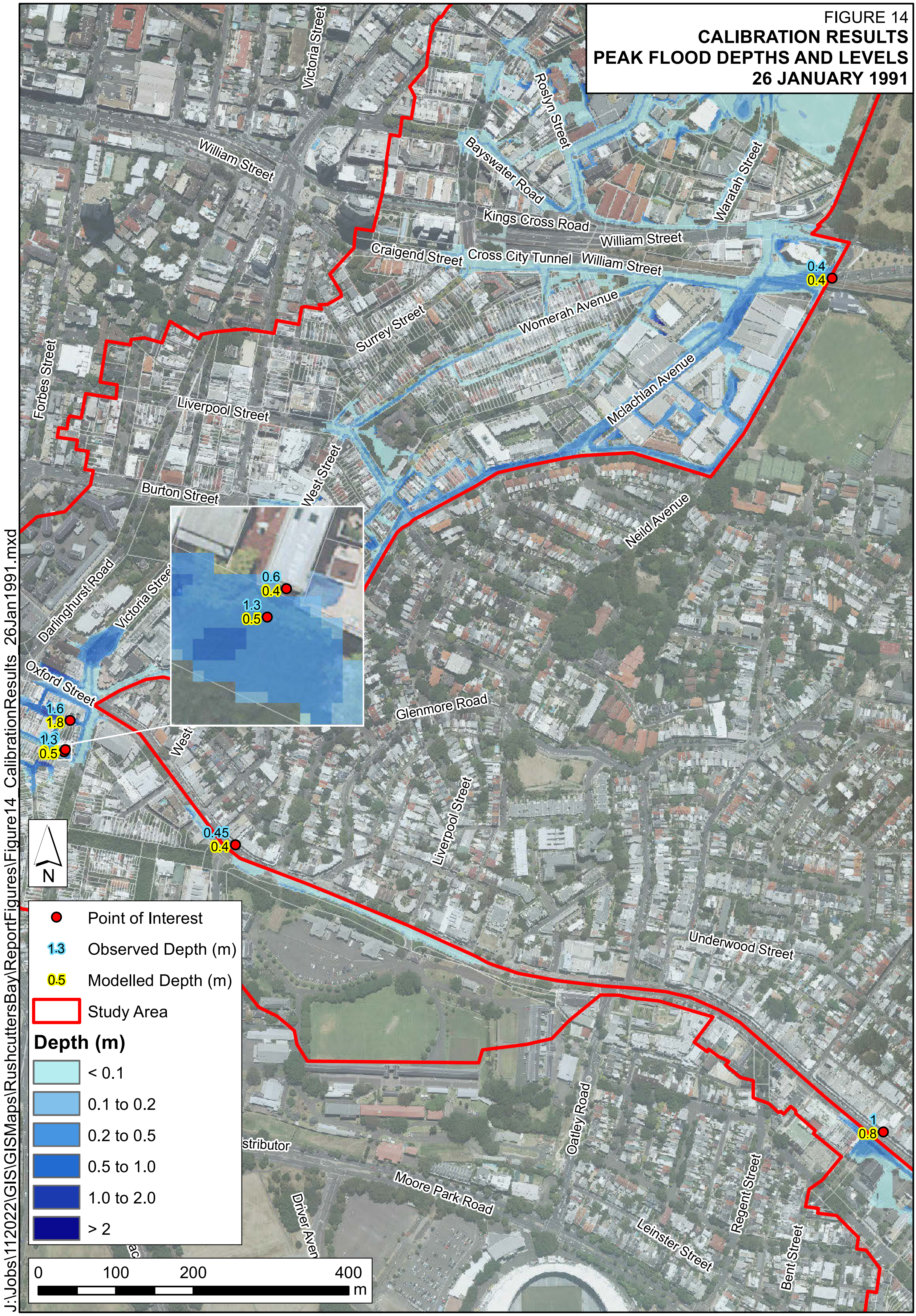

FIGURE 14
CALIBRATION RESULTS
PEAK FLOOD DEPTHS AND LEVELS
26 JANUARY 1991
Victoria Street
William Street
Roslyn Street
Bayswater Road
Waratah Street
Kings Cross Road
William Street
Craigend Street
Cross City Tunnel
William Street
Surrey Street
Womerah Avenue
Forbes Street
Liverpool Street
Mclachlan Avenue
West Street
Burton Street
Neild Avenue
Darlinghurst Road
Victoria Street
Oxford Street
Glenmore Road
Liverpool Street
Underwood Street
Oatley Road
Moore Park Road
Driver Aven
Regent Street
Leinster Street
Bent Street
0.4
0.4
0.6
0.4
1.3
0.5
1.6
1.8
1.3
0.5
0.45
0.4
1
0.8
N
Point of Interest
1.3 Observed Depth (m)
0.5 Modelled Depth (m)
Study Area
Depth (m)
< 0.1
0.1 to 0.2
0.2 to 0.5
0.5 to 1.0
1.0 to 2.0
> 2
0 100 200 400
m
J:\Jobs\112022\GIS\GISMaps\RushcuttersBay\ReportFigures\Figure14_CalibrationResults_26Jan1991.mxd

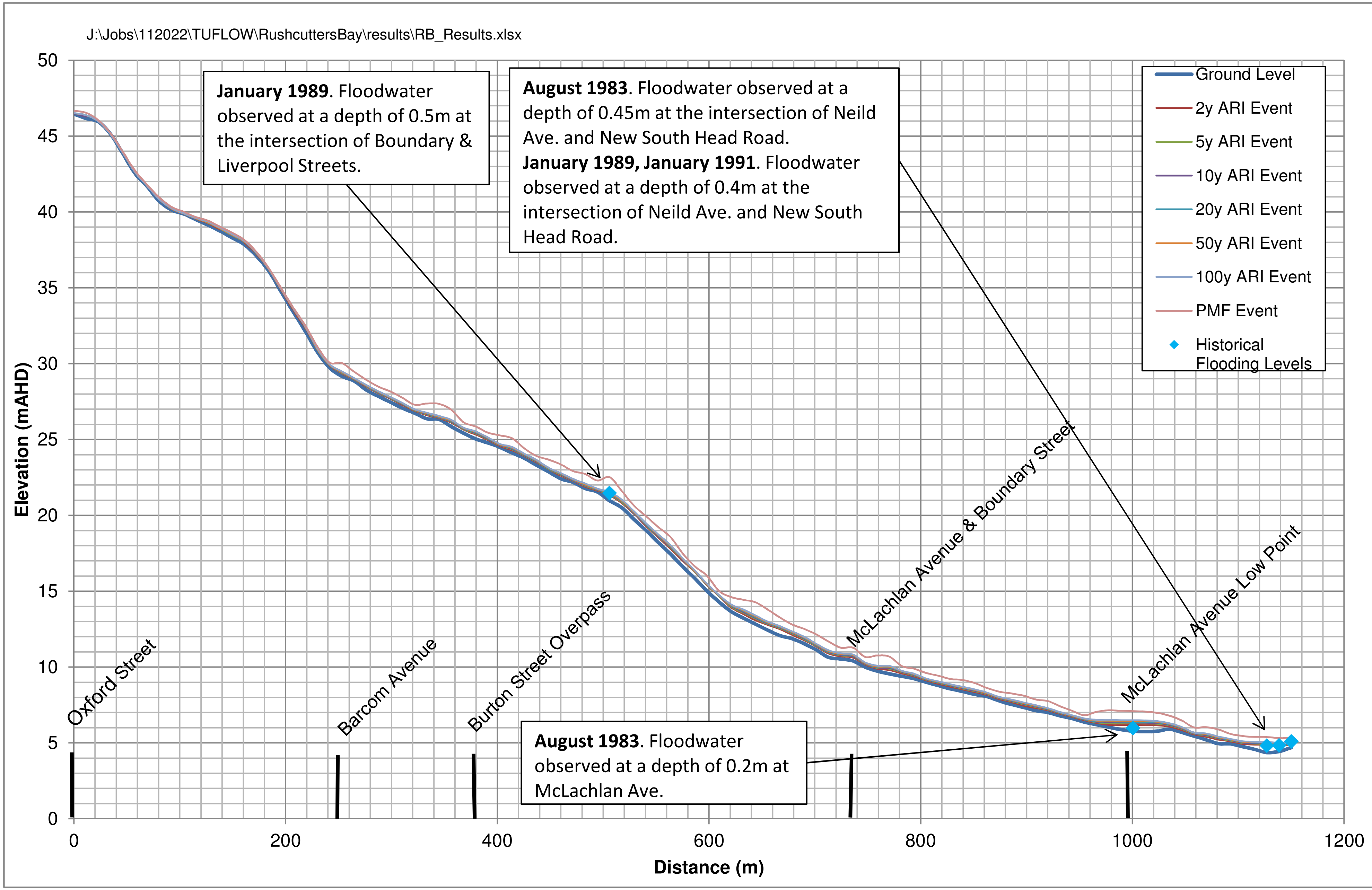

FIGURE 15
DESIGN FLOOD PROFILES
BOUNDARY STREET & MCLACHLAN AVENUE
OXFORD STREET TO NEW SOUTH HEAD ROAD

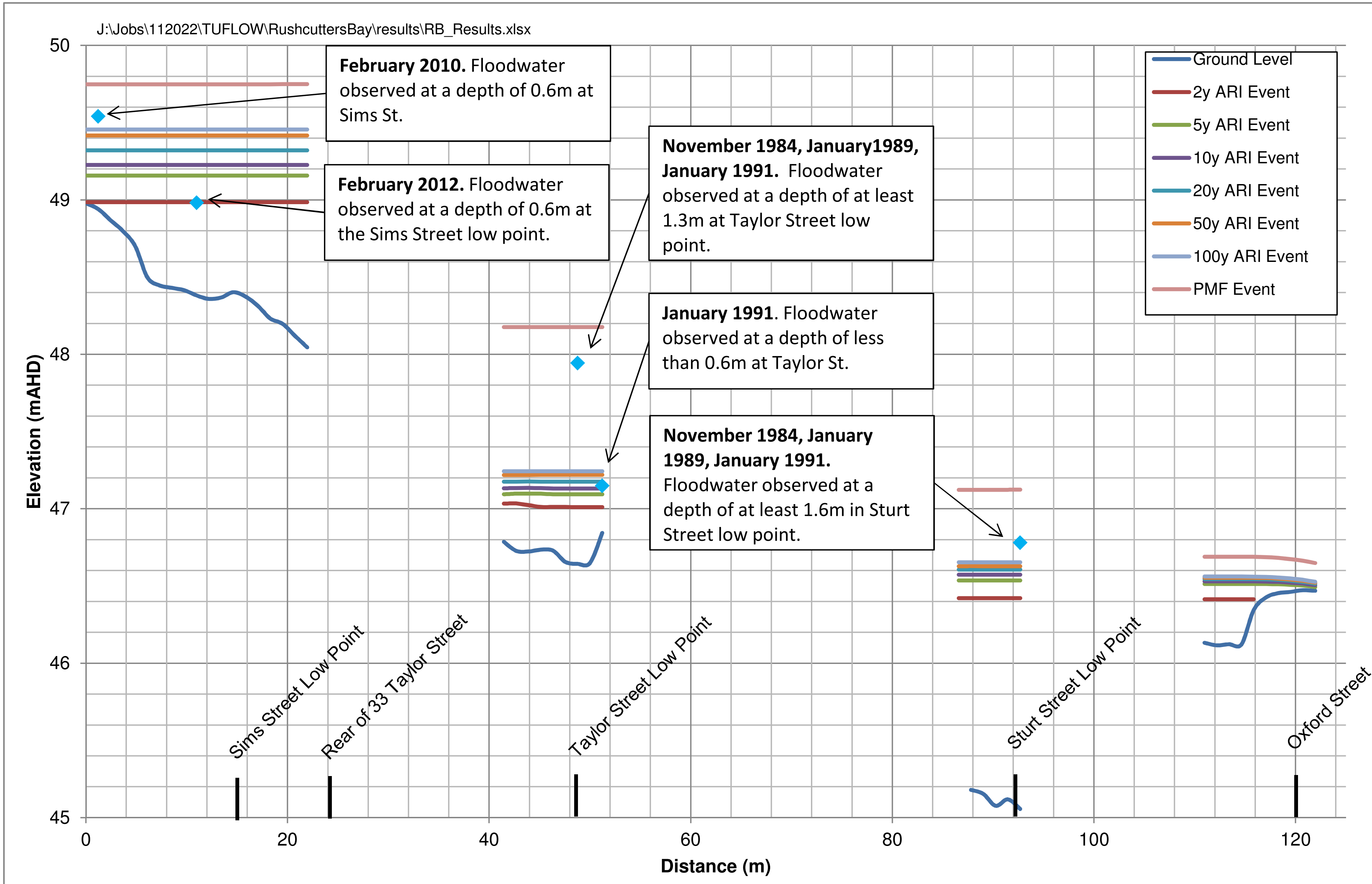

FIGURE 16
DESIGN FLOOD PROFILES
SIMS STREET, TAYLOR STREET & STURT STREET LOW POINTS
SIMS STREET TO OXFORD STREET

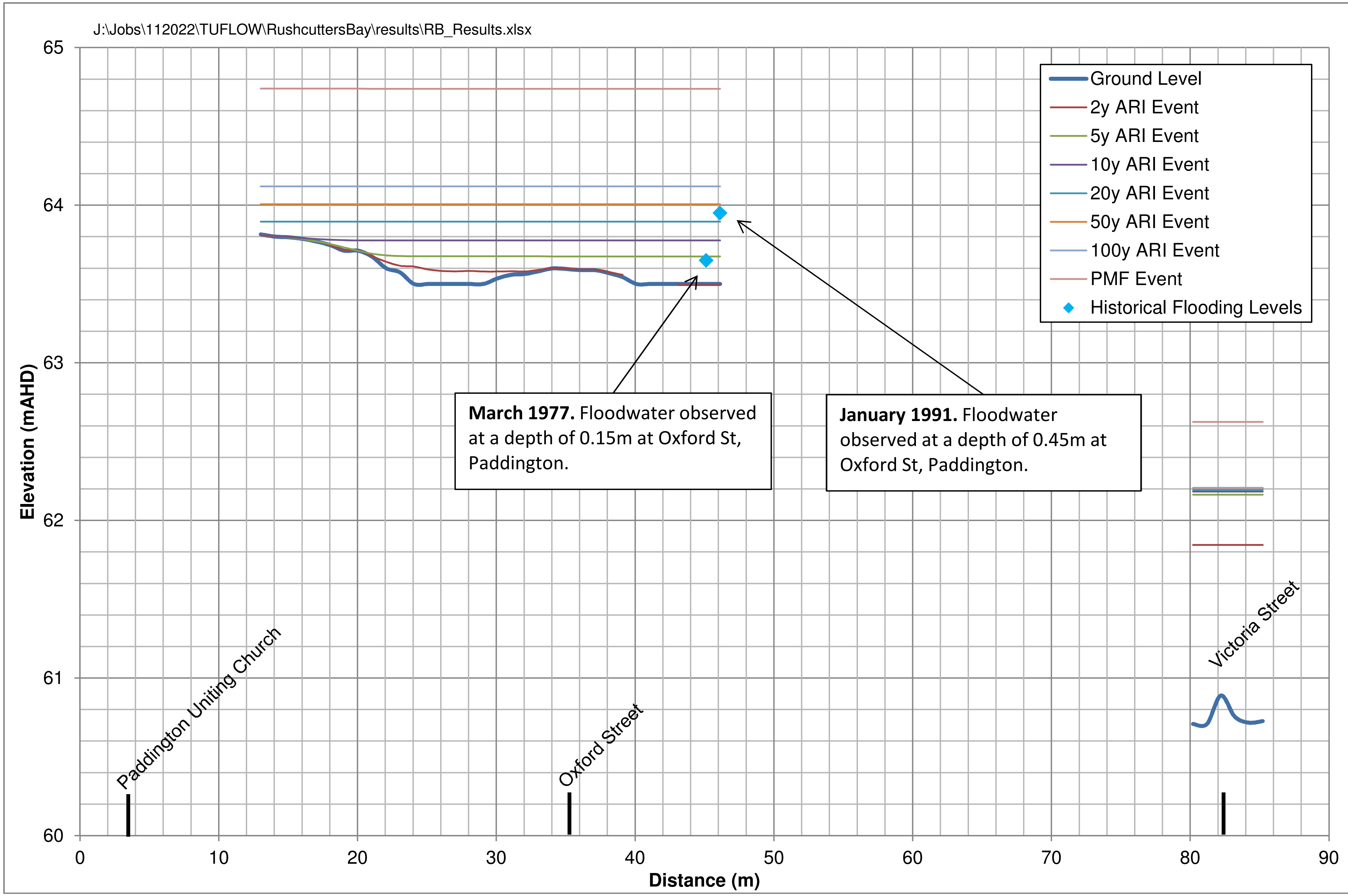

FIGURE 17
DESIGN FLOOD PROFILES
OXFORD STREET PADDINGTON
PADDINGTON UNITING CHURCH TO VICTORIA STREET