CITY OF SYDNEY

BLACKWATTLE BAY CATCHMENT FLOOD STUDY

FINAL REPORT

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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 Federal Government may also provide subsidies in some circumstances.

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.

The Blackwattle Bay Catchment Flood Study presented herein constitutes the first stage in the Floodplain Risk Management Program for the catchment (see Figure 1 for catchment location and extent). WMAwater has been engaged by the City of Sydney to prepare the Flood Study under the guidance of Council's floodplain management committee. This study provides the basis for the future management of those parts of the Blackwattle Bay catchment which are flood liable and within the City of Sydney local government area.

EXECUTIVE SUMMARY

The NSW Government's Flood Policy provides for:

- *a framework to ensure the sustainable use of floodplain environments,*
- *solutions to flooding problems,*
- *a means of ensuring new development is compatible with the flood hazard.*

Implementation of the Policy requires a four stage approach, the first of which is preparation of a Flood Study to determine the nature and extent of the flood problem.

The Blackwattle Bay Flood Study was initiated as a result of substantial flooding of roads and residential areas, most recently in November 1984, January 1991 and February 2001. This report has been prepared by WMAwater on behalf of the City of Sydney (Council) and the Office of Environment and Heritage (OEH) under the guidance of Council's floodplain management committee.

The specific aims of the Blackwattle Bay Flood Study are to:

- *define flood behaviour in terms of flood levels, depths, velocities, flows and extents within the Blackwattle Bay catchment study area;*
- *prepare flood hazard and flood extent mapping;*
- *prepare suitable models of the catchment and floodplain for use in a subsequent Floodplain Risk Management Study;*
- *to consider the potential effects of a climate change induced increase in design rainfall intensities and sea level rise; and*
- *carry out a flood damages assessment using surveyed floor levels.*

Description of Study Area (Section 1.2 of report): The Blackwattle Bay catchment is located in Sydney's inner city suburbs. This region lies within the City of Sydney Local Government Area and has been extensively developed for urban usage. The catchment covers an area of approximately 315 hectares with some 50 hectares of land draining directly into Blackwattle Bay and the remaining portion draining to Sydney Water's major trunk drainage system used to route flows from the upper regions of the catchment.

A number of locations within the catchment are flood liable. This flood liability mainly relates to the nature of the topography within the study area as well as the capacity of service provided by drainage assets. The topography of the catchment is steep in the upper areas, steep and undulating in the middle sections, and then flat particularly in the lower regions close to Blackwattle Bay.

Urbanisation throughout the catchment occurred prior to the installation of road drainage systems in the 1900s and many buildings have been constructed on overland flow paths or in unrelieved sags. Due to these drainage restrictions, topographic depressions can cause localised flooding as excess flows have no opportunity to escape via overland flow paths. This creates a significant drainage/flooding problem in many areas throughout the catchment.

Past Flooding Problems (Sections 1.3 of report): Catchment development has caused significant increases in peak runoff rates and volumes as well as restrictions in the conveyance capacity of overland flow paths. Consequently rainfall intensities as low as 2 year ARI levels can cause flooding at many locations within the catchment. June 1949, November 1961, March 1975, November 1984, January 1991 and February 2001 are some of the most significant storm events that have caused extensive flooding throughout the catchment.

Available Data (Section 2 of report): The Sydney Water's Blackwattle Bay Flood Study and the Hughes Trueman and Perrens Consultant's Blackwattle Bay and Johnstons Creek Catchment Drainage Study were completed in 1995 and 2004 respectively. They provide pit, pipe and overland flow details within the catchment as well as a number of flood level readings for historical events used in calibration/validation of the current model. Note however that neither of these studies carried out modelling of local overland flow nor of overbank flood and storage routing, i.e. both studies focused on pits and pipes hydraulics only.

Airborne Laser Scanning (ALS) survey (provides a very accurate and detailed definition of the ground surface) was available for the entire study area and was used to determine catchment areas as well as to define the topography for the hydraulic models. Council provided details on the pit and pipe network within the catchment.

A community questionnaire survey was undertaken during June 2011 with a return rate of 1% (122 responses) which aided in identification of problem flood regions within the catchment.

Approach (Section 3 of report): In the absence of an extensive historical flood record, a flood frequency approach cannot be undertaken for the Blackwattle Bay catchment. Therefore, design rainfalls have been used in conjunction with the establishment of a hydrologic/hydraulic modelling system. A variation on the direct rainfall on grid approach has been used in the hydrodynamic modelling package TUFLOW which negates the need for a separate hydrologic model.

Calibration to Historical Flood Levels (Section 4.4 of report): Due to the lack of available data a rigorous calibration (matching of actual flood height data to that produced from the models and so verifying the accuracy of the models) of the TUFLOW model could not be undertaken. This situation is typical of all urban catchments where there are limited flood records available (no instruments measuring water level installed and residents may not actually see the floodwaters since the flood happens very quickly – thus has to rely on debris marks or such. Questionnaires were sent out as part of this study to allow residents to advise of past flood events and data). However, limited calibration (26th January 1991 flood event) and validation (17th February 1993) of the model were undertaken based on recorded flood levels obtained from the aforementioned reports. This exercise indicates that the results from TUFLOW are generally similar to historical data. However, both rainfall and flood level data should be collected immediately following the next major flood and used to further verify the results as per the 2005 NSW Floodplain Development Manual recommendations.

Determination of Design Flood Flows and Levels (Section 6.4 of report): Design rainfall data from the Bureau of Meteorology and design rainfall patterns from Australian Rainfall and Runoff (1987) were obtained and input to the modelling procedure to obtain the design flood data. Detailed mapping was undertaken for a range of design events (2 year ARI, 5 year ARI, 10%, 5%, 2%, 1% AEP events and the Probable Maximum Flood) with the results provided as maps showing:

- Peak flood depths for all design flood events, Figure 14 - Figure 20;
- Peak flood levels for the 1% AEP and PMF flood events, Figure 21 - Figure 22;
- Peak flood velocity for the 1% AEP and PMF flood events, Figure 23 - Figure 24;
- Flood profiles along main trunk for all design flood events, Figure 25;
- Provisional flood hazard categorisation for all design flood events, Figure 27 - Figure 33;
- Preliminary flood hydraulic categorisation for all design flood events, Figure 34 - Figure 40; and
- Climate change scenarios (rainfall increases and sea level rise), Figure 41 - Figure 45.

Accuracy of Design Flood Levels and Extents (Sections 6.8 and 6.9): Sensitivity analyses (to assess the effects of changing various model parameters) were undertaken on model results. Part of this analysis was to assess the effects of possible increases in design rainfall (10%, 20% and 30%) due to climate change. The results indicate that the average increase (based on a comparison of the peak flood level at selected review points) in the 1% AEP event is:

- low level rainfall increase of 10% = +0.1m,
- medium level rainfall increase of 20% = +0.1m,
- high level rainfall increase of 30% = +0.2m.

However the results do show some variation between locations. On the other hand, the impacts of sea level rise are largely confined to the low lying areas adjacent to Blackwattle Bay.

The model results are much less sensitive to changes of the model parameter values. The most sensitive parameter was the pipe/culvert blockage factor which resulted in a maximum change in peak flood level of ±0.1m.

Due to the limited quantity and quality of the calibration data available and in view of the sensitivity analyses, it is estimated that the order of accuracy of the design flood levels is up to ±0.2m, however in many places the order of accuracy will be ±0.1m. These orders of accuracy are typical of such studies and can only be improved upon with additional observed flood data to refine the model calibration and more detailed and accurate definition of the terrain.

Flood Damages Assessment (Section 6.10): A flood damages assessment was undertaken for existing development in accordance to the OEH guidelines (Reference 16). The assessment was based on detailed floor level survey carried out by Council's surveyors and flood levels produced from the modelling of design events herein. Only properties which have surveyed floor levels have been included in the flood damages assessment. Table i indicates the estimated number of building floors which are likely to be flooded for a range of event magnitudes and the corresponding tangible damages. Damages to public structures have not been assessed.

Table i: Estimated Combined Flood Damages for the Blackwattle Bay Catchment

Event	Number of Properties Flood Affected	No. of Properties Flooded Above Floor Level	Total Tangible Flood Damages	Average Tangible Damages Per Flood Affected Property
2 year ARI	202	94	$ 8,851,400	$ 43,900
5 year ARI	236	112	$ 11,010,900	$ 46,700
10% AEP	246	131	$ 12,258,600	$ 49,900
5% AEP	259	141	$ 13,526,500	$ 52,300
2% AEP	268	163	$ 14,627,600	$ 54,600
1% AEP	283	171	$ 16,229,800	$ 57,400
PMF	307	255	$ 25,050,200	$ 81,600
		Average Annual Damages (AAD)	**$ 7,783,100**	**$ 25,400**

* Excludes all damages to public assets but includes external damages that may or may not occur with building floor inundation.

Outcomes: The main outcomes of this study are:

- full documentation of the methodology and results,
- preparation of depth, velocity, hazard and extent maps for the study area,
- an assessment of the potential impacts of climate change on flooding, and
- a modelling platform that will form the basis for a subsequent Floodplain Risk Management Study and Plan.

Recommendations: This Flood Study should be adopted by Council before proceeding with the subsequent floodplain risk management Study and Plan. As part of these subsequent studies a risk analysis of the implications of climate change on flooding should be undertaken.

The key recommendation from this study is to highlight the importance of collecting and maintaining a database of historical rainfall and flood height data. It is vital that information from future flood events is collected within 24 hours and the magnitude and direction of flow paths through private property recorded. This information will significantly improve the accuracy of the design flood levels and extents and ensure that known flood areas are identified and assessed. Data collection can be undertaken by Council Officers digitally photographing flood marks etc (they can be levelled later based on the photograph) and possibly mailing out a resident questionnaire requesting information and photographs. Unfortunately if this process is not done quickly, information is lost forever.

1. INTRODUCTION

This Flood Study has been prepared by WMAwater (formerly Webb, McKeown & Associates) on behalf of the City of Sydney (Council). The main objective of this study is to define the flood behaviour in the Blackwattle Bay catchment (the catchment) under existing conditions. This study has examined past flood events in addition to undertaking a flood assessment for a range of design storms. The findings in this report provide information to inform Council with regards to managing existing and future flood risk within the catchment.

All levels provided in this report are to Australian Height Datum (AHD). A glossary of terms is provided as Appendix A.

1.1. Objectives

The information and results obtained from this Flood Study will define existing flood behaviour and provide a firm basis for the development of a subsequent Floodplain Risk Management Study and Plan.

In addition to defining the flood behaviour (2 year ARI, 5 year ARI, 10%, 5%, 2%, 1% AEP events and the Probable Maximum Flood (PMF)) in the Blackwattle Bay catchment, the study was developed to:

- Define flood behaviour in terms of flood levels, depths, velocities, flows and flood extents within the study area;
- Provide provisional flood hazard and flood extent mapping (for all design events modelled); and
- Consider the potential effects of a climate change induced increase in design rainfall intensities and sea level rise in accordance with the NSW Government guidelines[1].

1.2. Study Area

The Blackwattle Bay catchment is located in Sydney's inner city suburbs of Glebe, Chippendale, Ultimo, Darlington, Camperdown, Redfern, Pyrmont and Surry Hills (see Figure 1). This region lies within the City of Sydney Local Government Area (LGA) and has been extensively developed for urban usage. Land use is predominantly medium to high-density housing as well as commercial and industrial developments. In addition, there are pockets of open space sporadically positioned throughout the catchment, such as Wentworth, Victoria, and Prince Alfred Parks.

The catchment covers an area of approximately 315 hectares with some 50 hectares of land draining directly into Blackwattle Bay (the Bay) and the remaining portion draining to Sydney

[1] It should be noted, however that in September 2012 the NSW Government repealed mandatory compliance with the 0.4 m sea level rise by the year 2050 and 0.9 m sea level rise by the year 2100. Councils in NSW must now make their own decisions regarding the assessment of sea level rise.

Water's major trunk drainage system (known as SWC 17) to route flows from the upper regions of the catchment. The trunk drainage system is linked to Council's feeder drainage system consisting of covered channels, in-ground pipes, culverts and kerb inlet pits. Further information on the drainage system is presented in Section 1.4.

A number of locations within the catchment are flood liable. This flood liability mainly relates to the nature of the topography within the study area as well as the capacity of service provided by drainage assets. The topography of the catchment is steep in the upper areas, steep and undulating in the middle sections, and then flat particularly in the lower regions close to Blackwattle Bay (Figure 2). The upper regions of the catchment experience the greatest relief with a maximum elevation of approximately 60m AHD occurring in the vicinity of Surry Hills. Urbanisation throughout the catchment occurred prior to the installation of road drainage systems in the 1900s and many buildings have been constructed on overland flow paths or in unrelieved sags. Due to these drainage restrictions, topographic depressions can cause localised flooding as excess flows have no opportunity to escape via overland flow paths. This creates a significant drainage/flooding problem in many areas throughout the catchment.

Council has advised that there are no large developments proposed within the catchment. Any future development in this area is most likely to be in the form of urban consolidation, with aggregation of individual lots creating high density high rise residential developments.

1.3. Catchment History

Blackwattle Bay catchment was first settled in the early 1800s. The original natural drainage system comprised numerous rock gullies draining through small pockets of mangroves and into various coves that have now been consolidated into Blackwattle Bay. As development proceeded, the natural drainage lines were converted into a constructed drainage system of open channels and sub surface elements.

During the late 1800s and early 1900s, urbanisation in the area spread significantly. This development led to a widespread change in land usage from predominantly pervious to largely impervious uses, greatly increasing peak flows and the overall flow volume. By the late 1900s, the majority of the channel system was progressively covered over and piped, with much of this system forming the backbone of today's stormwater drainage system.

In summary, the effect of development was a significant increase in peak runoff rates and volumes combined with a restriction in the conveyance capacity of overland flow paths. The existing pattern and intensity of development would not permit restoration of natural conditions and sufficient land is not available to achieve this.

1.3.1. Flooding History

Historical records (photographs, reports) indicate that rainfall intensities as low as 2 to 5 year ARI levels can cause flooding at many locations within the catchment. Consequently there have been many instances of flooding in the past with June 1949, November 1961, March 1975, November 1984, January 1991 and February 2001 being some of the most significant storm events causing extensive flooding throughout the catchment. Section 2.5.1 provides details on a number of these past rainfall events responsible for the above mentioned floods.

To further highlight the potential magnitude of flooding in the region, Council has provided photographs (Photo 1 & Photo 2) at Macarthur Street during the March 1975 flood event. It can be seen that water depths in excess of one metre covered large areas during this event.

Photo 1: Macarthur Street at junction of Mountain Street

Photo 2: Macarthur Street at junction of Mountain Street

1.4. Drainage System

The catchment is serviced by a major/minor drainage system. Property drainage is directed to the kerb/gutter system where it is then able to enter the Council owned minor street drainage network. Reference 1 determined that the minor drainage within the catchment delivers approximately a 5 year ARI flood. Flow is then routed into the Sydney Water Corporation (SWC) owned and maintained SWC17 trunk drainage system. This trunk drainage system is composed of eight large drains that run predominately south-north through the catchment. A list of these eight main branches is presented below and illustrated in Figure 3:

- Wattle Street (Council) Branch,
- Wattle Street (Old Council) Branch,
- Tooheys Brewery Sub-Branch,
    - Prince Alfred Park Sub-Branch,
- Blackwattle Creek Branch,
    - Mountain Street/Shepherd Street Branch,
- Bay Street Branch,
    - Victoria Park Sub-Branch.

The upper branches collect runoff from a wide area in the south of the catchment, before converging to a narrow strip of parallel branches immediately south of Wentworth Park which then discharge into Blackwattle Bay near the Fish Markets.

When the capacity of the drainage system is exceeded there is the potential for velocities and/or flow depths combining to generate high hazard flooding conditions. Past events indicate that events as small as the 5 year ARI rainfall event can cause these conditions in several locations throughout the catchment. Section 6.5 discusses the flood liability of some key locations within the catchment.

2. AVAILABLE DATA

2.1. Background

Various items of data as well as reports salient to the study have been collected and reviewed. Most reports and datasets were sourced from Council and supplemented by additional survey where required. Reports were reviewed particularly for topographic/hydrologic parameters as well as observations of historical flood events. The key focus of the exercise was to collect data suitable for the model calibration and validation process.

This section provides a summary of the reports as well as a description of the various forms of data utilised in the study.

2.2. Previous Reports

2.2.1. Blackwattle Bay (SWC 17) Flood Study, Sydney Water, September 1995 (Reference 1)

The aim of this flood study was to determine flooding behaviour for the 20% to 1% AEP design floods as well as the Probable Maximum Flood (PMF). The study used the hydrologic model ILSAX, which utilises the pit and pipe survey data and other parameters to generate runoff hydrographs. These inflows represented the upstream boundary conditions and were then input into a MIKE-11 UD model which was used to predict flood depths and velocities. Due to limited computer memory capacity, pits in the 1D network were aggregated in some cases. The downstream boundary (Blackwattle Bay) was represented as a 1m AHD level.

The study identified six major floodways:

- Wentworth Park Road;
- Blackwattle Lane;
- Wattle Street;
- Broadway between Mountain Street and Wattle Street;
- Buckland Street; and
- Abercrombie Street.

2.2.2. South Sydney Stormwater Quality and Quantity Study, Blackwattle Bay and Johnstons Creek Catchments, Hughes Trueman & Perrens Consultants, September 2004 (Reference 2)

This report was commissioned by South Sydney Council (now known as City of Sydney) on behalf of a Consortium comprising Council, Sydney Water Corporation and South Sydney Development Corporation to assess the performance of the trunk drainage systems in the Johnstons Creek and Blackwattle Bay catchments. The two trunk drainage systems SWC17 and SWC55 (Blackwattle Bay and Johnston Creek respectively) now lie within the City of Sydney LGA. The study aims were to provide stormwater management options. Key issues examined in the report are as follows:

- Analysis of the origin and causes of stormwater flows that contribute to stormwater flooding;
- Strategies for managing stormwater flooding;
- Options for reducing stormwater flooding;
- Water quantity and quality management opportunities; and
- Water quality improvement.

The study modelled the stormwater using the DRAINS modelling package. The DRAINS model was then used to produce a summary of pipe flows estimates, estimates of potential overland flow paths and estimates of flood depths in sag points.

2.3. Survey Data

Airborne Laser Scanning (ALS) data of the site was obtained from Council to define ground surface elevations. The provided ALS data was a combination of data collected in 2007 and 2008 with a 1.3 m average point separation. The ALS provides ground level spot heights from which a Digital Elevation Model (DEM) can be constructed. For well defined points mapped in areas of clear ground, the expected nominal point accuracies (based on a 68% confidence level) are ±0.15 metre (vertical accuracy). 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. This data formed the foundation of the 2D hydraulic model build process.

2.4. Pit and Pipe Data

Council provided a database of the pit and pipe network dated 21st March 2011, a summary of which is shown in Figure 3. The physical details included:

- coordinates of each pit;
- linkage between pits;
- pipe dimensions; and
- pit details (type of pit, inlet type and dimensions and depth to invert).

Where the pit and pipe information was not available from Council's database, estimates were made via StreetView in Google Maps, by site inspection or interpolation from the existing data. In these cases the pit inlet levels were obtained from the DEM. Table 1 contains a summary of the pit and pipe data used during modelling.

Table 1: Modelled Pit and Pipe Network

Pit Type	Number
Outlet	20
Kerb or Grate Inlets	711
Junctions	488

Pipe Diameter (mm)	Number	Total Length (m)
< 450	442	5315
450 - 750	444	9885
750 - 1000	120	2580
1000 - 2400	228	7578
2400 - 3800	48	1809

2.5. Rainfall Data

2.5.1. Historical Rainfall Data

There are no pluviometers (continuously collects rainfall data) or daily rainfall stations (collects only 24hour - daily rainfall) located within the study area. The closest pluviometer is located approximately 2 kilometres away (distance calculated from approximate study area centroid) at the SWC Annandale gauge (Gauge Number: 566065). Other proximate rainfall station locations are provided in Figure 4.

Rainfall events causing flooding in the catchment can be localised and as such will only be accurately "registered" by a proximate gauge. Gauges located even only a kilometre away in coastal areas such as the Blackwattle Bay catchment can show very different intensities and total rainfall depths than those experienced within the catchment itself.

Table 2 is a summary of the rainfall gauges used in this study (refer Figure 4 for locations). Whilst daily rainfall gauges have been included, these records are generally not suitable for calibration/validation of the modelling process as they are only 24 hour totals and thus do not define the short duration intensities that produce flooding in the region. Nevertheless, these gauges could be used to provide a reasonable spatial representation of historical rainfall within the catchment.

Table 2: Rainfall Data Sources

Station Number	Station Name	Ownership	Type	Record Period	Available Data
066062	Sydney (Observatory Hill)	BoM	Pluviometer	1858 – ongoing	03/01/1913 – 31/07/2009
066037	Sydney Airport AMO	BoM	Pluviometer	1962 - ongoing	06/07/1962 – 30/01/2009
566026	Marrickville SPS	SWC	Pluviometer	1904 - ongoing	31/12/1979 – 31/03/2011
566041	Crown St Reservoir	SWC	Daily Read	1882 –1960	-
566065	Annandale	SWC	Pluviometer	1988 - ongoing	01/01/1989 – 31/03/2011

BoM = Bureau of Meteorology SWC = Sydney Water Corporation

Figure 5 displays the rainfall burst intensity and frequency of various historical events at the Observatory Hill gauge. The largest recorded event with an ARI in excess of 100 years is the November 1984 rainfall event. Both the January 1991 and April 1998 events were also significant with rainfall intensities approximating the 20 year ARI event.

Furthermore, data from the two nearby Sydney Water owned pluviometer rainfall gauges (Annandale and Marrickville) have been analysed and numerous storm events identified as being significant are presented in Table 3 and Table 4. Not all historical rainfall events have been listed with preference being given to the more recent ones of reasonably large intensity. Events smaller than the 2 year ARI have not been displayed. Note that the available gauge data may not cover the entire period of record and there are non operational periods within the gauge record. Furthermore, only hourly data was available for analysis for the Marrickville gauge and as such calculated rainfall totals may be less than "event" rainfall depths, i.e. those calculated by use of 5 or 6 minute data.

Table 3: Events Identified from Annandale Gauge

Duration	Date	Time	Rainfall (mm/hr)	Approximate ARI
1 Hour	26/01/1991	15:00	53	5
	17/02/1993	8:36	71.5	20
	14/09/1993	1:00	47	3
	10/04/1998	8:00	47.5	4
	12/02/2010	22:00	45	3
2 Hour	26/01/1991	15:00	54	3
	17/02/1993	8:18	87	15
	14/09/1993	0:00	55	3
	10/04/1998	8:00	63.5	4

Table 4: Events Identified from Marrickville Gauge

Duration	Date	Time	Rainfall (mm/hr)	Approximate ARI
1 Hour	8/11/1984	9:00	42.5	3
	17/02/1993	10:00	44.5	3
	14/09/1993	0:00	53.5	6
	10/04/1998	8:00	48	4
	13/05/2003	10:00	64	15
2 Hour	17/02/1993	9:00	81	15
	10/04/1998	7:00	75.5	8
	13/05/2003	10:00	66	5

Note: Rainfall values have been calculated from hourly readings and may be less than true rainfall depths.

2.5.2. Design Rainfall Data

Design rainfalls were obtained from the Bureau of Meteorology (BoM) and temporal patterns were obtained from Australian Rainfall and Runoff (Reference 3). The Intensity-Frequency-Duration (IFD) data for the catchment is provided in Table 5.

Table 5: IFD Data for Blackwattle Bay Catchment

Location: 33.875S 151.200E NEAR.. Blackwattle Bay Issued: 25/10/2011

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

	Average Recurrence Interval						
Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	101	129	163	183	209	243	269
6Mins	94.3	121	153	172	196	228	252
10Mins	77.2	99.1	127	142	164	191	212
20Mins	56.6	73.1	94.8	107	124	146	163
30Mins	46.0	59.7	78.0	88.8	103	122	136
1Hr	31.2	40.5	53.5	61.2	71.2	84.4	94.5
2Hrs	20.3	26.4	34.9	39.9	46.5	55.1	61.8
3Hrs	15.6	20.3	26.7	30.6	35.6	42.2	47.3
6Hrs	9.90	12.8	16.9	19.2	22.3	26.4	29.6
12Hrs	6.34	8.21	10.7	12.2	14.2	16.8	18.7
24Hrs	4.12	5.33	6.98	7.95	9.22	10.9	12.2
48Hrs	2.64	3.42	4.49	5.12	5.94	7.02	7.85
72Hrs	1.97	2.55	3.33	3.80	4.41	5.21	5.82

(Raw data: 40.84, 8.2, 2.55, 85.87, 16.75, 5.22, skew=0.00, F2=4.29, F50=15.86) © Australian Government, Bureau of Meteorology

Probable Maximum Precipitation (PMP) rainfall depths used to determine the Probable Maximum Flood (PMF) were obtained from Reference 4 using the generalised short-duration method. The maximum duration for which the method is applicable in the region is 6 hours. The parameters used for estimating the PMP are:

- Terrain classification: rough;
- Adjustment for catchment elevation (EAF): 1;
- Moisture Adjustment Factor (MAF): 0.7, and;
- Ellipses enclosing the catchment: A and B (refer to Reference 4 for further explanation of ellipsoid selection).

Final rainfall depths used in the hydrological model are shown on Table 6.

Table 6: Probable Maximum Precipitation Depths (rounded to the nearest 10 mm)

Storm Duration (hours)	Ellipse A (mm)	Ellipse B (mm)
1	350	330
2	530	500
3	640	610
4	730	700
5	810	770
6	850	810

2.6. Downstream Boundary Water Levels

The downstream boundary of the study area is Blackwattle Bay. Blackwattle Bay is tidal so natural variability of water level is expected in the Bay from both tidal and catchment flows. For design flood estimation a level in Blackwattle Bay is required for calculation of water levels and pipe discharges in the lower parts of the catchment. There is no definitive approach for determining the coincidence of flooding in the catchment with a water level in the Bay. Flooding could occur on a low or high tide and be coincident with stormwater runoff from other parts of the catchment or not. An ocean anomaly could also occur (elevation of ocean level above the astronomical tide) as a result of the same meteorologic condition (low pressure system) that produces the intense rainfall. The largest recorded such event in Sydney Harbour was in May 1974 where the ocean level reached just over 1.4 metres AHD. However this event was not associated with very intense rainfall intensities. The highest astronomical tide (HAT) in a year is approximately 1 metre AHD.

A joint probability analysis is required to fully assess the situation and such a study would be limited by the limited amount of flood event data. Best practice at the time of writing in regard to the setting of downstream boundary conditions is to refer to NSW Government guidelines (Reference 14). The guidelines indicate that the local 1% AEP flood should be run in conjunction with a 5% AEP ocean water level (approximately 1.38 mAHD in Sydney Harbour) and vice versa (i.e. a local 5% AEP rainfall event with an ocean water level of 1% AEP, depending on sensitivity of the study area to elevated ocean levels). This approach was therefore adopted in this study. Adjustment of this tailwater level has been modelled with increases of +0.4 metres and +0.9 metres for the climate change analysis. This also provides a sensitivity analysis of the impact on flood levels in the lower catchment due to the water level in the Bay.

2.7. Community Consultation

A community questionnaire survey was undertaken during June 2011. 14,400 surveys were distributed to residents within the Blackwattle Bay study area and 122 responses were received. This equates to a return rate of 0.8% and as such the views expressed by this sample may not accurately reflect that of the total population. However it is normal that responses predominately come from residents that have been affected by flooding and 122 responses are in the order of magnitude of those impacted by flooding issues in the catchment.

The locations of the community consultation respondents are shown in Figure 6 along with regions identified by respondents as problem flood areas. Unfortunately no flood levels or depths were provided although the reported flood marks were able to be used as a means of model verification (for further details see Section 4.4).

Following the community consultation, it was found that three historic events in particular resonated with residents:

- April 1998;
- February 2001; and
- June 2007.

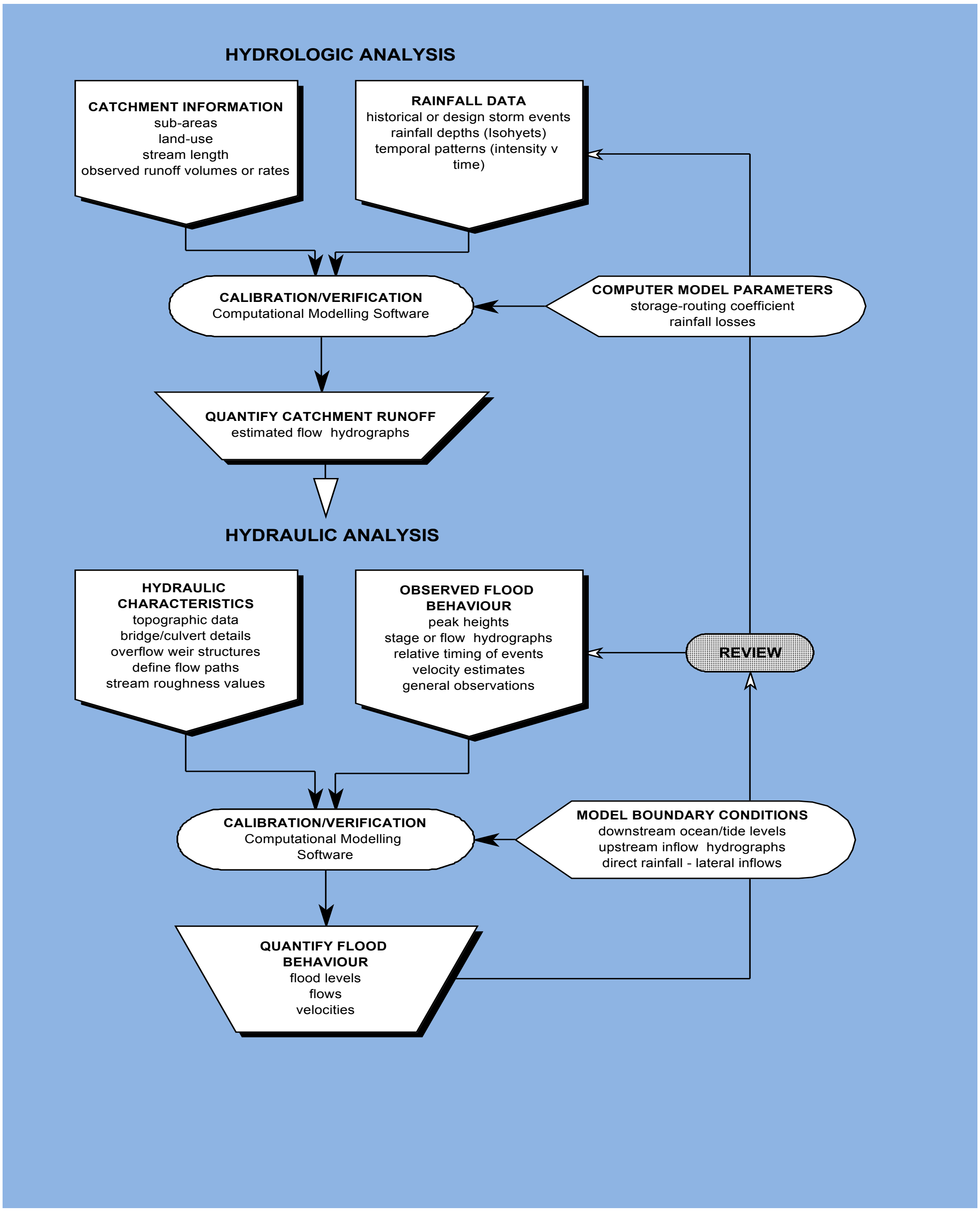
Interestingly, only the April 1998 event was prominent in the rain gauge analysis (see Section 2.5.1).

It should also be noted that over 70% of respondents (out of the 122 who replied) are aware of flooding or have some knowledge of flooding in the study area. Further, almost half of the respondents reported flooding on roads, which serve as formalised overland flow paths in this catchment as the sub-surface drainage system is overwhelmed by the runoff volume associated with more extreme events. The full set of results from the community consultation questionnaire are summarised in Figure 7A to 7C.

3. 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.). In the absence of an extensive historical flood record a flood frequency approach cannot be undertaken for the Blackwattle Bay catchment and must rely on the use of design rainfalls and establishment of a hydrologic/hydraulic modelling system. A diagrammatic representation of the flood study process is shown below.

Diagram 1: Approach Adopted in a Flood Study



4. MODEL BUILD

4.1. Overview

The hydrodynamic modelling program TUFLOW (Reference 5) has been used to model both the hydrologic and hydraulic processes in the catchment. TUFLOW is a finite difference grid based 1D/2D hydrodynamic model which uses the St Venant equations in order to route flow according to gravity, momentum and roughness. Furthermore, TUFLOW's rainfall on grid functions allows seamless merging of the hydrologic and hydraulic models. This negates the need to use an independent hydrologic model to determine inflow hydrographs for subsequent input to the hydraulic model.

TUFLOW is ideally suited to this study because it facilitates the identification of the potential overland flow paths and flood problem areas as well as inherently representing the available floodplain storage within the 2D model geometry. In addition to this, TUFLOW allows for the utilisation of breaklines at differing resolution to the main grid. Breaklines are used to ensure the correct representation of features which may affect flooding (features such as roads, embankments, kerbs, etc) which is especially important in an urban environment.

The incorporation of 1D elements into the 2D domain is another beneficial factor of TUFLOW. This allows such elements as open channels represented in 1D to function dynamically within the 2D grid. This suits the study as it facilitates the inclusion of channel flow within the context of a medium resolution 2D approach as well as facilitating the inclusion of the pit and pipe network.

4.2. Hydrology

The hydrologic model boundary covers the entire 315 hectare catchment area shown in Figure 2. As the TUFLOW model is utilising the direct rainfall method, rainfall for particular events was generated from IFD data obtained from the BoM (see Section 2.5.2) and input directly onto the 2D grid. To remove spurious losses associated with a DEMs tendency to exaggerate surface depressions (potentially causing a significant portion of rainfall to be retained within the catchment), rainfall is applied only to regions which are likely to collect and distribute flow such as kerbs and gutters. To achieve this, the catchment has been divided into 720 sub-catchments (See Figure 8) each of which contains a region to which excess rainfall from the entire sub-catchment is applied.

4.2.1. Rainfall on Grid Considerations

Given that the direct rainfall approach is a relatively new approach in hydrologic modelling studies in Australia, some discussion of the method is presented herein, along with a consideration of the advantages and disadvantages of the method.

Many studies undertaken by specialised consultants for both private and government clients, both in Australia and overseas, have been conducted using a direct rainfall approach. Also, within the literature on hydrological/hydraulic modelling there are examples of research which demonstrate the ability of this approach to emulate more established lumped conceptual hydrological models as well as more importantly to match observed data.

The main advantages of the approach are that:

- flows can be applied to the drainage system avoiding non-conservative over attenuation of flows due to the non-inclusion of sub grid features;
- routing is based on relatively high resolution topography and the full St Venant equations and so parameterisation of storage/routing processes is not necessary;
- no double routing of flows occurs such as will likely be a result in a joint modelling system; and
- the approach lends itself to the final product which is mapped flood levels to inform planning decisions.

4.2.2. Check Integrity of Rainfall on Grid Methodology

Whilst direct rainfall can be used to great advantage it is a relatively new method and as such it is best to corroborate the flows derived from the method against alternative methods (i.e. calibration/validation and comparison to other methods used to estimate design peak flow).

4.2.2.1. Mass Balance Check

As a partial check of the integrity of the employed rainfall on grid method, a mass balance assessment has been undertaken. This assessment utilises continuity equations to ensure that excess water mass is not lost during modelling. The mass balance assessment was performed for the 1% AEP design rainfall event and was undertaken in two steps:

1. The volume of the total applied rainfall was compared against the rainfall volume that actually entered the model via the rainfall on grid method. The difference was calculated to be effectively zero; and
2. The total volume of water entering the model was verified against the volume of water that exited the model or remained in the model after the model run had finished. The calculated residual was determined to be -1.1% which was slightly more than the -0.6% automatically calculated by TUFLOW. These residuals were found to be comparable indicating good model stability.

Typically, mass balance residuals for the rainfall on grid method within ±5% are considered acceptable (albeit as long as mass generation/loss is not overly localised). The residual calculated for this model is well within these limits and the model is therefore considered stable.

4.2.2.2. Comparable Catchment Hydrologic Model Check

To further test the reliability of the applied rainfall on grid method a number of flow comparisons have been made to peak flows obtained through more conventional methods. Flow results from the Rose Bay hydrologic model (DRAINS) were compared to those used in the current study. The Rose Bay catchment is located only 6km from the Blackwattle Bay catchment and they share many similar characteristics.

To removes the effects that differences in catchment delineation can have on peak discharge a number of sub-catchments specific yields were determined. Specific yield is calculated by dividing the peak discharge by the area of the upstream catchment. This removes the obvious effects that differences in sub-catchment size have on peak discharge. Table 7 displays the model comparisons for three random sub-catchments from both models.

Table 7: Comparable Catchment Hydrologic Model Check

Sub-catchment	TUFLOW			DRAINS			% difference
	Area (ha)	Peak Discharge (m^3/s)	Specific yield ($m^3/s/ha$)	Area (ha)	Peak Discharge (m^3/s)	Specific yield ($m^3/s/ha$)	
1	1.2	1.0	0.8	1.0	0.6	0.7	15
2	0.6	0.4	0.7	0.4	0.2	0.6	22
3	0.6	0.4	0.7	0.6	0.4	0.6	13

Discrepancies between the compared specific yields can be attributed to a number of reasons such as variance in loss parameters and design rainfall values, changes in land use and difference in the applied routing method. In this case it was noted that the DRAINS model used Horton Infiltration Curves and the applied losses were approximately double of those applied in the current study (for current study losses see Section 4.2.3). This is the most likely reason as to why the specific yields for the current study were greater.

It was found that the flows produced by the two models are comparable and thus the rainfall on grid method employed is robust.

4.2.3. Loss Parameters

The Australian Rainfall and Runoff 1987 (Reference 3) suggests a range of initial losses are possible (10 to 30+ mm) and a continuing loss of 2.5mm/hr for the pervious regions of catchments within NSW and more specifically Sydney. The conservative lower value of 10 mm initial loss has been adopted for this study. 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) and as such an initial loss of 1.5 mm has been applied with no continuing losses. A summary of losses applied to the TUFLOW model is displayed in Table 8.

Table 8: Adopted Design Rainfall Loss Parameters

Parameter	Pervious	Impervious
Initial Loss	10 mm	1.5 mm
Continuing Loss	2.5 mm/hr	0 mm/hr

The above losses are the same as adopted in the nearby Leichhardt Flood Study (Reference 6).

4.2.4. Percentage Impervious

The average perviousness of a catchment plays a significant role in determining the structure of the runoff hydrograph. It has implications for the peak flow, the total runoff volume as well as the time of concentration. Urban regions with large areas of impervious surfaces lose less rainfall to losses and flow reaches the downstream end of the catchment quicker due to the generally smoother surfaces associated with urbanisation[2] (and less initial loss). Thus it is important to determine the average imperviousness throughout the catchment for any hydrologic model.

For each of the sub-catchments mentioned previously an estimate of the percent imperviousness was calculated. Council provided the Local Environmental Plan (LEP) for the region which was used to distinguish areas of various zoning types. LEP zone types generally have fairly homogenous land uses and therefore a correlation is assumed between zoning and imperviousness. The mean imperviousness was calculated by detailed inspection of representative zones. Table 9 indicates the average perviousness and imperviousness for the various land uses in the catchment. Additionally, the spatial distribution of land usages can be seen in Figure 9.

Table 9: Percent Imperviousness for Various Land Uses

Land Use Description	% Pervious	% Impervious
Infrastructure (roads, train tracks etc)	10	90
General Residential	30	70
Mixed Use	0	100
Public Recreation (parks, ovals etc)	100	0
University of Sydney	80	20
Light Industrial	0	100
Harold Park	80	20
Local Centre	0	100
Neighbourhood Centre	30	70

4.3. Hydraulic Modelling

The hydraulic model converts applied flow (discharge generated by a hydrological model) into flood levels and velocities. In the approach used herein, where the hydraulic model also converts rainfall excess into runoff (i.e. the traditional work carried out by hydrological models), the hydraulic model is the only model run. The hydraulic model in this study takes an applied rainfall depth (net of losses) and routes it to create flood extent, level and velocity information.

More importantly, TUFLOW model can clearly define spatial variations in flood behaviour across the study area. Information such as flow velocity, flood levels and hydraulic hazard can be readily mapped in detail across the model extent. This information can then be integrated easily into a GIS based environment enabling outcomes to be efficiently incorporated into Council's planning activities.

[2] Note: A time of concentration of a catchment is also affected by a number of other factors including the catchment slope, shape and size.

4.3.1. Model Build Process

Model construction begins with the DEM (Digital Elevation Model) which defines a catchment's topographical characteristics at high resolution. Finer features (such as kerbs and gutters) that have significant impacts on flows may then be incorporated via additional spatial layers of information. Also, via the inclusion of dynamically linked 1D elements, drainage pits and pipes are also incorporated. Numerous spatial layers are applied to the model with the aim of closely replicating the catchment's true topographic conditions.

4.3.2. Model Domain and Grid

A two metre 2D grid was generated from the ALS as mentioned in Section 2.3. A computational time step of 0.5 seconds was adopted. Buildings have been excluded from the model as it is assumed that there is very little flow through the structures and minimal temporary floodplain storage.

4.3.3. Roughness Values

The Manning's "n" values for each grid cell were estimated based on established references (e.g. Reference 17) and engineering experience. Values were applied to the 2D overland area based on land use as shown in Table 10. For 1D Manning's roughness values see Section 4.3.4.

Table 10: Adopted Manning's 'n' values

Land Use	Manning's 'n'
Roads	0.015
Parks	0.06
Parking areas	0.02
Ponds and lakes	0.01
Dense Vegetation	0.08
Residential and mixed use*	0.05

*Buildings were nulled out in the hydraulic model

There is no definitive approach for representing buildings and fences in 2D hydraulic models. The approach to be adopted depends on a number of factors including: the nature of the development; the model extent/grid definition; and the likely impacts of the approach on flood levels and velocities.

For this study it is considered that properties adjacent to the overland flow-path boundary would not be part of the effective flow path due to the presence of fences and buildings. This was achieved by nulling grid cells based on digitised building outlines which effectively constricted the available flow path.

The "loss" of temporary floodplain storage by nulling the building outlines is a slightly conservative assumption as in reality some floodwaters may enter these buildings under some flooding scenarios. However this approach was adopted as it was considered that the impact of such an assumption would be negligible relative to the overall flood runoff volume.

4.3.4. Pit and Pipe Network

Pit and pipe data (see Section 2.4 for details) provided by Council was used to create a 1D drainage network in TUFLOW. As agreed in the Brief, pipes of diameter smaller than 450 mm were not included in the TUFLOW model as it was assumed that pipes of this size would suffer from blockage during storms due to leaves and debris. This is a “conservative” assumption as some of these smaller pipes may not be blocked. All pipes and culverts were allocated a Manning’s roughness of 0.013 (Reference 17).

4.3.5. Trunk Drainage Blockage

The effect of blockage in urban drainage systems (pipes and open channels) has become a significant factor in design flood estimation following the post flood observations from the North Wollongong August 1998 and Newcastle June 2007 events. However, recent reviews of how blockage should be included in design flood analysis are inconclusive, as it appears that the incidence of blockage is not consistent across all catchments or even within the same catchment. Thus there is no consensus regarding the design approach that should be adopted.

In this study the approach adopted for all pipes and culverts of diameter larger than 450mm (or area proportional to a 450mm pipe) has been to assume 25% blockage. This approach has been adopted to take into account blockage caused by larger debris (such as cars, fencing, vegetation etc.) being swept into drainage structures. All blockages have been assumed to occur at the culvert/pipe level with all pit inlets at 100% capacity. Sensitivity to the selected pipe blockage values has been considered in Section 6.8.

4.3.6. Boundary Conditions

As the direct rainfall on grid method has been applied to the model there is no need for upstream flow boundaries. The main boundary condition is at the downstream end of the model at Blackwattle Bay. This is a tidal boundary in which a static tailwater level of 1.38 mAHD (see Section 2.6) has been adopted. This approach has been commonly adopted in similar studies throughout Sydney and is recommended by OEH (Reference 14).

4.4. Model Calibration/Validation Events

Generally calibration/validation is a process whereby historical events are used to test a model’s ability to accurately replicate observed behaviour (i.e. match historical flood levels). This process requires rainfall data (pluviometer and daily read) and then observations such as:

- streamflow velocities;
- gauged water levels;
- peak flood level at specific locations; and
- peak flood level extent at a specific location at a specific time.

No stream gauges exist within the catchment and as such no gauged water levels or flows are available for historical events. A review of historical records was carried out to identify dates of historical events in the hope of obtaining other calibration/verification data. More recent events (since 1980) were the main focus of this review as rainfall data collected prior to this date would generally be of insufficient resolution to be used in model calibration.

A review of the reports mentioned in Section 2.2 (References 1 & 2) provided peak flood levels at key points within the Blackwattle Bay catchment for the post 1980 period. Rainfall data for these events was also examined to determine if suitable data was available for modelling. Table 11 shows the historical data available for the most recent significant flood events.

Table 11: Available Data for Recent Flood Events

Dates with water level data	Peak Flood Levels (Reference 1 & 2)	Gauge 066062	Gauge 066037	Gauge 566065	Gauge 566026
26th January 1991	✓	✓		✓	✓
17th February 1993	✓		✓	✓	✓
9-10th April 1998		✓	✓	✓	✓
13th April 1994				✓	✓

After consideration of the available flood level and rainfall data, it was determined that the 26th January 1991 flood event would be the most suitable for model calibration, with the 17th February 1993 event to be used for model verification. The results of the model calibration and verification are provided in Section 6.2.

5. CLIMATE CHANGE

The 2005 Floodplain Development Manual (Reference 7) requires that Flood Studies and Floodplain Risk Management Studies consider the impacts of climate change on flood behaviour.

The current best practice for considering the impacts of climate change (sea level rise and rainfall increase) has been evolving rapidly. Key developments in the last four years have included:

- release of the Fourth Assessment Report by the Inter-governmental Panel on Climate Change (IPCC) in February 2007 (Reference 8), which updated the Third IPCC Assessment Report of 2001 (Reference 9);
- preparation of Climate Change Adaptation Actions for Local Government by SMEC Australia for the Australian Greenhouse Office in mid 2007 (Reference 10);
- preparation of Climate Change in Australia by CSIRO in late 2007 (Reference 11), which provides an Australian focus on Reference 10;
- release of the Floodplain Risk Management Guideline Practical Consideration of Climate Change by the NSW Department of Environment and Climate Change in October 2007 (Reference 12 - referred to as the DECC Guideline 2007).

In August 2010, the former NSW Department of Environment, Climate Change and Water (DECCW) issued the Flood Risk Management Guide (Reference 14) which required incorporation of sea level rise benchmarks in flood risk assessments.

In October 2012 the NSW Government repealed mandatory compliance with its 2009 Sea Level Rise Policy (Reference 13) which states that:

"*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 the 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";

Hence, Councils must now make their own decisions regarding the assessment of sea level rise. City of Sydney has made no formal statement that it is adopting a sea level rise assessment different to the Policy Statement (Reference 13) previously issued by the NSW Government.

As a result of the information provided in the documents mention previously, and to keep up-to-date with current best practice, this study incorporates an assessment of climate change. 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 most recent guideline (Reference 14) indicates a 0.9 metre sea level rise by the year 2100 and a 0.4 metre rise by the year 2050. These changes in sea level have been modelled as part of the sensitivity analysis for this study. It should be noted that climate change and the associated rise in sea levels will continue beyond 2100.

The climate change scenarios in the earlier DECC Guideline 2007 (Reference 12) suggested for undertaking rainfall sensitivity analysis in flood studies are indicated below.

Increase in peak rainfall and storm volume:

low level rainfall increase	=	10%,
medium level rainfall increase	=	20%,
high level rainfall increase	=	30%.

A high level rainfall increase of up to 30% is recommended for consideration due to the uncertainties associated with this aspect of climate change and to apply the “precautionary principle”. A 30% rainfall increase is probably overly conservative. However, as part of the rainfall sensitivity analysis used in this study all changes to rainfall intensities mention above have been modelled. The DECC Guideline 2007 (Reference 12) is currently the only NSW reference providing guidelines for rainfall increases for design flood analysis due to climate change.

Results for the climate change analysis are contained in Section 6.9.

6. HYDRAULIC MODELLING

6.1. Approach

Limited model calibration/verification was undertaken initially to ensure that the model could replicate historical events and the model was subsequently used to determine design flood flows, levels, velocities and extents. Sensitivity analysis was then undertaken to assess the effect of changing various model parameters.

6.2. Calibration/Verification Results

The TUFLOW model was calibrated using the 26th January 1991 flood event. A comparison of modelled peak flood levels compared to the observed levels is presented in Table 12. Figure 10 displays the flood depth and extent map along with the model calibration points.

Table 12: Calibration Results for the 26th January 1991 Flood Event – Peak Flood Levels

Location	Observed (m AHD)	Modelled (m AHD)	Difference (m)
Glenmore Wholesale Meats (Wentworth Park Road)	2.8	2.8	0.0
Kauri Hotel (Bridge Road)	2.5	2.3	-0.2
WC Penfold Stationers*	3.9*	3.9	0.0
145 Broadway Newsagent	8.3	8.3	0.0
Century Motors, 3 Owen St**	5.7	-	n/a

*This flood point (from Reference 1) details a flood level at 4.41mAHD. It also notes that depth relative to the gutter is 0.30m and the depth relative to floor level as 0.93m. From the ALS the ground level has been measured at approximately 3.6mAHD. From this it can be concluded that the accurate flood level would be a combination of the gutter depth and the ground elevation.
** There has been new development in this area since the 1991 event in which ground levels have been elevated. It is now unlikely that flooding could occur at this point.

The 17th February 1993 verification event was used to validate the calibrated model. The results of the verification event are contained in Table 13 and the flood depths and extent map along with the calibration points are displayed in Figure 11.

Table 13: Verification Results for the 17th February 1993 Flood Event – Peak Flood Levels

Location	Observed (m AHD)	Modelled (m AHD)	Difference (m)
44 Wattle Street	2.5	2.6	0.2
2 Wentworth Park Road	2.9	2.8	-0.1
Glenmore Wholesale Meats (Wentworth Park Road)	2.5	2.6	0.1
145 The Broadway, Ultimo	8.3	8.2	-0.0
70 Wentworth Park Road	2.6	2.4	-0.2

The calibration and verification results show high levels of correlation for the majority of recorded flood marks. Discrepancies may occur for any number of reasons such as the blockage of a pit/culvert by debris or diversion of flow by an upstream obstacle such as a parked car or newly built fence.

A review of these results indicates that the model has been suitably calibrated/validated and can therefore be used in the modelling of design events.

6.3. Critical Duration

Critical storm duration analysis is undertaken to determine the storm duration that produces the greatest flood levels for the given design event. A range of storm durations were modelled for the 1% AEP event and it was found that the critical duration varied (ranging from 15 to 720 minutes) spatially (see Figure 12). However, the variance in peak flood levels for the tested durations was not considered to be markedly different with all peak flood levels within ±0.1m. Therefore, for all design events, excluding the PMF, the 2 hour duration was used to determine the peak flood levels.

A similar process was undertaken for the PMF with various PMP durations (1 to 6 hours) modelled so that peak flood levels and associated rainfall durations could be identified. The 1 hour duration PMP was determined to be the critical duration in all regions of the catchment and was thus used to determine peak flood levels.

6.4. Overview of Results

A number of maps have been produced to display the flood affected regions for the various design events. It should be noted that inundation patterns and/or peak flood levels shown for design events are based on best available estimates of flood behaviour within the catchment. Inundation from local overland flow may vary depending on the actual rainfall event and local influences (parked cars, change in topography, road works etc.). Tabulated results (Table 14 - Table 18) are also provided in the following sections for ease of comparison between flood events. Further, peak flood levels have been recorded at regions of interest throughout the catchment and the locations of these readings are displayed in Figure 13.

A summary of the results is provided as follows:

- Peak flood depths for all design flood events, Figure 14 - Figure 20;
- Peak flood levels for the 1% AEP and PMF flood events, Figure 21 - Figure 22;
- Peak flood velocity for the 1% AEP and PMF flood events, Figure 23 - Figure 24;
- Flood profiles along main trunk for all design flood events, Figure 25;
- Provisional flood hazard categorisation for all design flood events, Figure 27 - Figure 33;
- Preliminary flood hydraulic categorisation for all design flood events, Figure 34 - Figure 40;
- Climate change scenarios (rainfall increases and sea level rise), Figure 41 - Figure 45; and
- Properties inundated above flood levels for all design flood events, Figure 46.

6.5. Results at Key Locations

The results for peak flood depths and velocities at key locations are shown in Table 14 while the peak flood levels are provided in Table 15 (refer to Figure 13 for locations). The performance of the stormwater drainage system within the study area is governed by the complex interaction between:

- Conveyance within the formal drainage system (pipes and box culverts), and
- Ponding and overland flow along streets and through private land.

A large range of depths (see Figure 14 - Figure 20) and velocities (see Figure 23 - Figure 24) can be observed throughout the catchment for the design flood events. One feature of flooding within the study area is that the sub-surface drainage system generally flows at capacity (refer to Table 16) even for smaller events (i.e. 2 year ARI) and the majority of the flows traverse through the catchment via overland flow paths. The intersection of Parramatta Road and Buckland Street (point 8) experiences the greatest flood depths with approximately 0.9 metres during the 2 year ARI event up to 1.6 metres in the 1% AEP event and 2.4 metres in the PMF. On the other hand, flow velocities are predominantly low throughout the catchment. Exceptions to this occur at the Beaumount Street and Cleveland Street intersection (point 6), at the William Henry Street and Wattle Street intersection (point 13), as well as along Bridge Road going downstream towards the tramline (point 21) where velocities of over 2 m/s can occur during the 1% AEP event. A closer inspection of these locations found that the local topography is characterised by steep gradients.

Table 14: Peak Flood Depths (m) and Velocities (m/s) at Key Locations (refer to Figure 13)

ID	Location	2 year ARI		5 year ARI		10% AEP		5% AEP		2% AEP		1% AEP		PMF	
		D	V	D	V	D	V	D	V	D	V	D	V	D	V
1	Chalmers St	0.1	0.2	0.2	0.1	0.3	0.1	0.3	0.1	0.3	0.1	0.4	0.2	0.6	0.4
2	Cleveland St between Chalmers & Pit	0.2	0.2	0.2	0.3	0.2	0.3	0.3	0.3	0.3	0.4	0.3	0.6	0.3	0.5
3	Regent St between Cleveland & Lawson	0.3	0.1	0.3	0.1	0.4	0.1	0.4	0.3	0.4	0.4	0.5	0.5	0.7	0.6
4	Regent St	0.6	0.2	0.7	0.5	0.7	0.5	0.7	0.6	0.8	0.7	0.8	0.8	1.0	0.4
5	Boundary St / Ivy St	0.3	0.7	0.3	0.7	0.3	0.7	0.4	0.7	0.4	0.7	0.5	0.8	1.1	1.5
6	Beaumount St / Cleveland St	0.5	1.1	0.6	1.4	0.7	1.6	0.7	1.7	0.8	1.8	0.8	2.0	1.5	3.0
7	Abercrombie St near Blackfriars St	0.0	0.0	0.0	0.0	0.0	0.2	0.0	0.2	0.0	0.4	0.0	0.3	0.1	0.8
8	Parramatta Rd / Buckland St	0.9	0.6	1.2	0.6	1.3	0.6	1.4	0.6	1.5	0.6	1.6	0.7	2.4	1.1
9	Blackwattle Ln near Small St	0.7	1.1	0.8	1.3	0.9	1.4	1.0	1.4	1.0	1.5	1.1	1.5	1.8	1.7
10	Blackwattle Ln / Kelly St	0.4	0.9	0.6	0.9	0.7	0.9	0.8	0.9	0.9	0.9	0.9	0.9	1.9	0.9
11	Blackwattle Ln / Macarthur St	0.8	0.4	1.0	0.4	1.1	0.5	1.2	0.5	1.3	0.9	1.4	1.0	2.4	0.6
12	Bay St / Wentworth Park Rd	0.4	0.5	0.6	0.6	0.7	0.6	0.8	0.6	0.9	0.6	1.0	0.6	1.9	2.4
13	William Henry / Wattle St	0.1	0.2	0.1	0.3	0.1	1.4	0.2	1.9	0.2	2.0	0.2	2.3	0.9	4.2
14	Wattle St / Fig St	0.4	0.4	0.5	0.4	0.5	0.6	0.6	0.6	0.6	0.6	0.7	0.5	1.7	2.9
15	Wattle St (near tramline)	0.7	0.2	0.7	0.2	0.8	0.2	0.8	0.1	0.9	0.2	0.9	0.2	1.9	2.1
16	Campbell St	0.5	0.4	0.6	0.3	0.6	0.3	0.7	0.3	0.7	0.4	0.8	0.4	1.2	0.3
17	Darghan Ln / Mitchell St East	0.4	0.7	0.4	1.1	0.5	1.2	0.5	1.2	0.6	1.2	0.6	1.2	1.0	1.5
18	Wentworth Park Rd / Mitchell St	0.3	1.1	0.4	1.3	0.5	1.3	0.6	1.4	0.7	1.4	0.8	1.4	1.8	1.6
19	Wentworth Park Rd / Lyndhurst St	0.5	0.6	0.6	0.8	0.7	0.9	0.8	1.1	0.9	1.3	1.0	1.5	1.8	2.2
20	Talfourd St	1.0	0.1	1.1	0.1	1.1	0.1	1.1	0.1	1.2	0.1	1.2	0.1	1.3	0.3
21	Bridge Rd (underneath tramline)	0.1	4.0	0.2	5.0	0.2	5.6	0.2	6.4	0.3	6.8	0.3	6.7	0.4	6.7
22	Bridge Rd / Wentworth Park Rd	0.4	0.3	0.5	0.5	0.6	0.8	0.6	0.9	0.7	1.0	0.8	1.1	1.5	1.9

Note: “D” and “V” in the second row represent the Peak Flood Depth (measured in m) and Velocity (measured in m/s) respectively.

Table 15: Peak Flood Levels (mAHD) at Key Locations (refer to Figure 13)

ID	Location	2 year ARI	5 year ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
1	Chalmers St	29.9	30.0	30.1	30.1	30.2	30.2	30.4
2	Cleveland St between Chalmers & Pit	31.2	31.2	31.2	31.2	31.2	31.2	31.3
3	Regent St between Cleveland & Lawson	23.9	23.9	23.9	24.0	24.0	24.0	24.3
4	Regent St	18.4	18.4	18.5	18.5	18.5	18.5	18.8
5	Boundary St / Ivy St	14.2	14.3	14.3	14.4	14.4	14.4	15.0
6	Beaumount St / Cleveland St	11.6	11.7	11.8	11.8	11.9	11.9	12.6
7	Abercrombie St near Blackfriars St	11.5	11.5	11.5	11.6	11.6	11.6	11.7
8	Parramatta Rd / Buckland St	8.4	8.7	8.8	8.9	9.0	9.0	9.9
9	Blackwattle Ln near Small St	5.4	5.5	5.6	5.7	5.7	5.8	6.5
10	Blackwattle Ln / Kelly St	4.6	4.7	4.8	4.9	5.0	5.0	6.1
11	Blackwattle Ln / Macarthur St	4.4	4.5	4.6	4.7	4.8	4.9	6.0
12	Bay St / Wentworth Park Rd	2.9	3.1	3.2	3.3	3.4	3.4	4.3
13	William Henry / Wattle St	3.6	3.6	3.6	3.7	3.7	3.7	4.4
14	Wattle St / Fig St	2.6	2.7	2.7	2.8	2.8	2.9	3.9
15	Wattle St (near tramline)	2.6	2.7	2.7	2.8	2.8	2.9	3.9
16	Campbell St	13.5	13.6	13.6	13.7	13.8	13.8	14.2
17	Darghan Ln / Mitchell St East	3.7	3.8	3.8	3.9	3.9	4.0	4.4
18	Wentworth Park Rd / Mitchell St	2.7	2.8	2.9	3.0	3.1	3.2	4.2
19	Wentworth Park Rd / Lyndhurst St	2.5	2.6	2.7	2.8	2.9	2.9	3.8
20	Talfourd St	16.9	17.0	17.0	17.0	17.0	17.1	17.2
21	Bridge Rd (underneath tramline)	3.1	3.2	3.2	3.2	3.3	3.3	3.4
22	Bridge Rd / Wentworth Park Rd	2.2	2.3	2.3	2.4	2.5	2.5	3.3

Table 16: Peak Flow Distribution (m^3/s) across Key Locations in the Blackwattle Bay Catchment (refer to Figure 13)

ID	Location	2 year ARI	5 year ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
	Overland Flows							
OF1	Abercrombie St	3.5	4.9	5.7	6.9	7.8	8.8	20.5
OF2	Buckland St	6.4	10.5	13.5	17.3	20.9	24.5	78.8
OF3	City Rd	0.3	1.4	0.5	0.6	1.9	2.2	4.0
OF4	Wattle St / Parramatta Rd	1.9	5.7	8.6	12.9	16.1	19.6	84.0
OF5	Broadway Shops	7.0	8.8	9.6	9.7	10.3	10.8	17.0
OF6	Mountain St / Parramatta Rd	0.4	0.5	0.8	1.6	2.6	3.7	29.9
OF7	Bay St / Parramatta Rd	0.6	0.6	0.6	0.6	0.6	0.6	4.4
OF8	Wattle St / William Henry St	0.6	2.9	4.4	6.8	9.7	12.2	67.5
OF9	Bay St / William Henry St	0.8	1.8	3.0	5.6	8.4	10.8	64.1
OF10	Cowper St	0.8	1.9	2.7	3.4	4.4	5.2	18.0
OF11	Mitchell Ln East	1.8	2.8	3.1	3.5	3.8	4.1	7.7
OF12	Mitchell St	0.3	0.4	0.5	0.6	0.7	0.8	9.0
OF13	Wattle St (underneath tramline)	0.3	0.9	1.4	2.4	3.8	5.2	78.8
OF14	Wentworth Park Rd (underneath tramline)	3.7	7.6	10.6	14.6	18.5	21.9	78.6
OF15	Bridge Rd (underneath tramline)	1.5	2.5	3.2	4.2	5.3	5.7	15.1
	Pipe Flows							
P1	Wattle St Branch @ Parramatta Rd	0.7	0.6	0.7	0.8	0.9	1.1	0.8
P2	Blackwattle Ck Branch @ Parramatta Rd	2.3	2.3	2.3	2.5	2.5	2.5	2.6
P3	Mountain/Shepherd St Branch @ Parramatta Rd	2.7	3.4	3.6	3.8	3.9	4.0	4.4
P4	Wattle St Branch @ Wentworth Park Inflows	0.3	0.4	0.4	0.4	0.4	0.5	0.5
P5	Old Wattle St Branch @ Wentworth Park Inflows	0.2	0.7	1.1	1.2	1.2	1.2	1.3
P6	Blackwattle Ck Branch @ Wentworth Park Inflows	10.0	9.9	9.9	9.9	9.8	9.8	10.1
P7	Bay St Branch @ Wentworth Park Inflows	1.3	1.4	1.5	1.6	1.6	1.6	1.7
P8	Mitchell St @ Wentworth Park Inflows	1.4	1.4	1.4	1.4	1.4	1.4	1.7
P9	Wattle St Branch @ Wentworth Park Outflows	0.9	0.9	0.9	0.9	0.9	1.0	1.2
P10	Old Wattle St Branch @ Wentworth Park Outflows	0.2	0.7	1.1	1.2	1.2	1.2	1.3
P11	Blackwattle Ck Sub-Branch 1 @ Wentworth Park Outflows	3.0	3.3	3.4	3.6	3.7	3.8	4.5
P12	Blackwattle Ck Sub-Branch 2 @ Wentworth Park Outflows	3.1	3.4	3.5	3.7	3.8	3.9	4.6
P13	Blackwattle Ck Branch @ Wentworth Park Outflows	5.1	5.8	6.1	6.5	6.6	6.7	7.8
P14	Bay St Branch @ Wentworth Park Outflows	2.8	2.9	3.0	3.1	3.1	3.1	3.5

The peak water level profiles for the catchment for all design flood events are shown in Figure 25. A 5% AEP ocean water level of 1.38 mAHD was adopted for all events and the premise of this has been discussed in Section 2.6. As can be seen from the profiles, the overland flows are predominantly shallow at the upstream sections of the catchment. In the presence of a flow path restriction (i.e. buildings), the flows experience ponding upstream of the obstructions and these areas which retard flows perform as an informal detention basin. Flood storage areas are found downstream of the catchment, i.e. at Wentworth Park and its surrounds, where lower flow velocity and higher flood depths can be expected.

A review of results, Table 14 in particular, reveals that flood depths can vary greatly for different design events and locations throughout the catchment. To determine the flood liability of individual properties floor level survey has been undertaken by Council so that modelled design flood levels can be compared to property floor levels. The survey was performed on close to 240 properties throughout the catchment. The selected property locations and details on the selection criteria are provided in Appendix B. A flood damages assessment was carried out and the results are presented in Section 6.10.

Referring to Table 17, it was found that 171 properties are liable to over floor inundation in the 1% AEP event. In smaller events such as the 2 year ARI event this figure drops to 94 properties although this estimate is conservative given the prudent blockage assumption. This number is approximately half of the total number of properties that are flood affected, which includes those properties that are inundated in the yard but not above the building floor level. The proportion of residential properties which are flood affected is significantly higher than the commercial/industrial lots. Whilst overall flood liability numbers are not high (compared against a total number of homes of circa 14,400 inclusive of apartments), those that are flood liable are persistently so. The properties that are over floor flood liable are impacted by overland flows and located in unrelieved sags. As a result many tend to be flooded in smaller events (i.e. 2 year ARI event), as well as the larger events (i.e. 1% AEP event).

Table 17: Over-floor Flood Liability for Blackwattle Bay Catchment

	Properties Flood Affected			No. of Properties Flooded Above Floor Level		
Event	**Residential**	**Commercial/Industrial**	**Total**	**Residential**	**Commercial/Industrial**	**Total**
2 year ARI	162	40	202	71	23	94
5 year ARI	189	47	236	82	30	112
10% AEP	197	49	246	96	35	131
5% AEP	206	53	259	102	39	141
2% AEP	212	56	268	120	43	163
1% AEP	226	57	283	127	44	171
PMF	248	59	307	202	53	255

The locations of these flood liable properties are mapped in Figure 46. It can be observed that they are quite distributed across the catchment and primarily located along the major overland flow paths.

Due to the combination of high flood depths and velocities, many regions of the catchment are affected by high hazard flows. Figure 27 to Figure 33 show the flow hazard classification throughout the catchment for various design flood events. It can be seen that during the 1% AEP flood event many roads form significant flow paths with high hazard flows, with the situation worsening for the PMF.

Although the NSW Floodplain Development Manual (Reference 7) provides guidelines on determining hydraulic categories it does not explicitly define each category. Consultants and authorities use different approaches for this. For the purpose of this study the preliminary hydraulic categories have been adopted based on previous experience and review of literature (e.g. Reference 15):

- *Floodway = Velocity * Depth > 0.25 m^2/s AND Velocity > 0.25m/s OR Velocity > 1m/s*
- *Flood Storage = Depth > 0.2m (provided that NOT categorised as Floodway)*
- *Flood Fringe = Depth < 0.2m (provided that NOT categorised as Floodway or Storage)*

Figure 34 to Figure 40 display the preliminary hydraulic categorisation for the various design flood events.

6.5.1. Major Access Road Flooding

Two major arterial roads in the catchment are subject to flooding from events as small as the 2 year ARI event. Parramatta Road and Cleveland Street form one of the main road linkages from the Eastern Suburbs through to the city and into the Western Suburbs. Excessive flooding of these roads could potentially inhibit traffic and result in significant impacts on traffic flows throughout the region. During a significant flood event it is likely that emergency service vehicles would be required in the affected area, though access may be severely hindered by the possibility of major road closures. A summary of flood depths on these two major access roads is provided in Table 18.

Table 18: Major Road Peak Flood Depths (m) for Various Events

ID	Location	2 year ARI	5 year ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
6	Beaumount St / Cleveland St	0.5	0.6	0.7	0.7	0.8	0.8	1.5
8	Parramatta Rd / Buckland St	0.9	1.2	1.3	1.4	1.5	1.6	2.4

6.6. Flood Hazard Classification

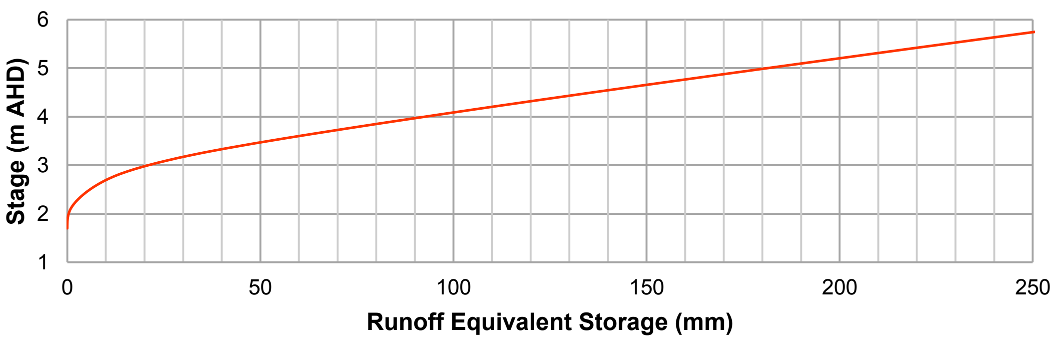
The 2005 NSW Government's Floodplain Development Manual (Reference 7) determines the *provisional flood hazard* categorisation of an area based on the combination of the depth and velocity of floodwaters on the land. The classification is a qualitative assessment based on a number of factors as listed in Table 19 which will be assessed in the Floodplain Risk Management Study and Plan.

Table 19: Hazard Classification

Criteria	Weight	Comment
Size of the Flood	TBA	To be assessed in the Flood Risk Management Study and Plan
Flood Awareness of the Community	TBA	To be assessed in the Flood Risk Management Study and Plan
Depth and Velocity of Floodwaters	TBA	To be assessed in the Flood Risk Management Study and Plan
Effective Warning and Evacuation Times	TBA	To be assessed in the Flood Risk Management Study and Plan
Evacuation Difficulties	TBA	To be assessed in the Flood Risk Management Study and Plan
Rate of Rise of Floodwaters	TBA	To be assessed in the Flood Risk Management Study and Plan
Duration of Flooding	TBA	To be assessed in the Flood Risk Management Study and Plan
Effective Flood Access	TBA	To be assessed in the Flood Risk Management Study and Plan
Additional Concerns such as Bank Erosion, Debris, Wind Wave Action, Sewage overflows	TBA	To be assessed in the Flood Risk Management Study and Plan
Provision of Services	TBA	To be assessed in the Flood Risk Management Study and Plan

6.7. Wentworth Park Storage

The majority of flow generated in the study catchment must first flow around Wentworth Park before it can reach the catchment outlet (Blackwattle Bay). The Park is slightly raised from its surrounds, forcing flows to travel along bordering roads and not through the Park itself. The minor relief in the region, coupled with flow path restriction, causes the area to retard flows and perform as an informal detention basin. The stage/storage (displayed as rainfall runoff equivalent) relationship is shown in the chart as follows:



The attenuation of flow in the region can be seen in Figure 26. It shows the modelled 1% AEP hydrograph immediately upstream of Wentworth Park as well as the trunk drainage hydrographs for the major culverts that drain the region (mentioned in Section 1.4). All culverts peak at full capacity and it can be seen that culvert flows remain near capacity for some time after the arrival of the main peak. This is due to the volume of storage in the region.

6.8. Sensitivity Analysis

Sensitivity analysis was carried out in order to assess the affect that adjusting model parameters had on model results. A comparison was carried out using peak flood levels and flows for the 1% AEP design event. The following scenarios were modelled:

- An increase in rainfall losses of 10% (both initial and continuing losses);
- A decrease in rainfall losses of 10% (both initial and continuing losses);
- An increase in bed resistance (Manning's 'n') of 20%;
- A decrease in bed resistance (Manning's 'n') of 20%;
- Pipe/culvert blockage at 0%; and
- Pipe/culvert blockage at 50%.

A summary of the results obtained are shown in Table 20 and Table 21. The tables show the differences between the results for each tested run and the 1% AEP design flood event (base case).

Table 20: Sensitivity Analysis of Flows

ID	Location	Base Case (m^3/s)	Impact (%)					
			Losses +10%	Losses -10%	Manning's 'n' +20%	Manning's 'n' -20%	Blockage 50%	Blockage 0%
Overland Flow								
OF4	Wattle St / Parramatta Rd	19.6	-1.5%	1.1%	-4.6%	8.0%	6.3%	-2.3%
OF6	Mountain St / Parramatta Rd	3.7	-1.0%	1.0%	-14.8%	21.7%	13.9%	-3.7%
OF8	Wattle St / William Henry St	12.2	0.0%	0.6%	-6.5%	9.8%	14.2%	-15.2%
OF9	Bay St / William Henry St	10.8	-0.8%	0.0%	-3.1%	4.6%	15.0%	-7.0%
OF13	Wattle St (underneath tramline)	5.2	-0.2%	4.0%	-4.8%	5.8%	34.2%	-23.8%
OF14	Wentworth Park Rd (underneath tramline)	21.9	-0.3%	0.2%	-6.2%	5.3%	14.7%	-15.1%
Pipe Flow								
P2	Blackwattle Ck Branch @ Parramatta Rd	2.5	0.0%	0.0%	-0.1%	-0.2%	-32.5%	25.5%
P3	Mountain/Shepherd St Branch @ Parramatta Rd	4.0	-0.1%	-0.3%	1.1%	-2.3%	-32.7%	17.7%
P6	Blackwattle Ck Branch @ Wentworth Park Inflows	9.8	-0.7%	-0.9%	-0.3%	-0.8%	-35.7%	35.9%
P7	Bay St Branch @ Wentworth Park Inflows	1.6	0.0%	0.2%	0.8%	-0.4%	-36.3%	31.6%
P13	Blackwattle Ck Branch @ Wentworth Park Outflows	6.7	0.0%	0.0%	0.1%	-0.1%	-34.1%	29.2%

Table 21: Sensitivity Analysis of Flood Levels

ID	Location	Base Case (mAHD)	Impact (m)					
			Losses +10%	Losses -10%	Manning's 'n' +20%	Manning's 'n' -20%	Blockage 50%	Blockage 0%
1	Chalmers St	30.2	0.0	0.0	0.0	0.0	0.0	0.0
2	Cleveland St between Chalmers & Pit	31.2	0.0	0.0	0.0	0.0	0.0	0.0
3	Regent St between Cleveland & Lawson	24.0	0.0	0.0	0.0	0.0	0.0	0.0
4	Regent St	18.5	0.0	0.0	0.0	0.0	0.0	0.0
5	Boundary St / Ivy St	14.4	0.0	0.0	0.0	0.0	0.0	0.0
6	Beaumount St / Cleveland St	11.9	0.0	0.0	0.0	0.0	0.0	0.0
7	Abercrombie St near Blackfriars St	11.6	0.0	0.0	0.0	0.0	0.0	0.0
8	Parramatta Rd / Buckland St	9.0	0.0	0.0	0.0	0.0	0.0	0.0
9	Blackwattle Ln near Small St	5.8	0.0	0.0	0.0	0.0	0.0	0.0
10	Blackwattle Ln / Kelly St	5.0	0.0	0.0	0.0	0.0	0.0	0.0
11	Blackwattle Ln / Macarthur St	4.9	0.0	0.0	0.0	0.0	0.0	-0.1
12	Bay St / Wentworth Park Rd	3.4	0.0	0.0	0.0	0.0	0.1	-0.1
13	William Henry / Wattle St	3.7	0.0	0.0	0.0	0.1	0.0	0.0
14	Wattle St / Fig St	2.9	0.0	0.0	0.0	0.0	0.1	0.0
15	Wattle St (near tramline)	2.9	0.0	0.0	0.0	0.0	0.1	0.0
16	Campbell St	13.8	0.0	0.0	0.0	0.0	0.0	0.0
17	Darghan Ln / Mitchell St East	4.0	0.0	0.0	0.0	0.0	0.0	0.0
18	Wentworth Park Rd / Mitchell St	3.2	0.0	0.0	0.0	0.0	0.1	-0.1
19	Wentworth Park Rd / Lyndhurst St	2.9	0.0	0.0	0.0	0.0	0.1	-0.1
20	Talfourd St	17.1	0.0	0.0	0.0	0.0	0.0	0.0
21	Bridge Rd (underneath tramline)	3.3	0.0	0.0	0.0	0.1	0.0	0.0
22	Bridge Rd / Wentworth Park Rd	2.5	0.0	0.0	0.0	0.0	0.0	0.0
		Mean:	0.0	0.0	0.0	0.0	0.0	0.0

Overall, results were shown to be insensitive to the tested variables with a maximum of ±0.1 metres variation to peak flood levels at the tested locations. This can generally be accommodated within the 0.5 m freeboard (if adopted) applied to the 1% AEP results to determine the Flood Planning Levels (FPLs).

In general flood levels and flows were most sensitive to adjusting the pipe/culvert blockage factor. An increase in pipe blockage from 25% (as per the base case) to 50% blockage caused an increase in overland flow of up to 14% at Wattle Street (indicated as OF8 in Figure 13, with decrease flow in the pipes) and minor increases in flood levels in the lower catchment (approximately 0.1 metres).

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

6.9. Climate Change Results

As part of the study the following climate change scenarios have been analysed for the 1% AEP event in accordance with the DECC Guideline 2007 (Reference 12):

- 10% increase in design rainfall intensity,
- 20% increase in design rainfall intensity,
- 30% increase in design rainfall intensity,
- 0.4 m rise in tailwater level in Blackwattle Bay, and
- 0.9 m rise in tailwater level in Blackwattle Bay.

6.9.1. Rainfall Increase

The results for the rainfall increase scenarios are tabulated in Table 22. Overall, an increase in the 1% AEP design rainfalls result in generally an increase in flood levels across the study catchment predominately in the main flow paths and lower regions of the catchment. A 10% increase in design rainfall intensity results in approximately 0.2 m maximum increase in peak flood levels, a 20% rainfall increase results in approximately 0.4 m maximum increase in peak flood levels and a 30% rainfall increase results in approximately 0.5m maximum increase in peak flood levels (generally around the Wentworth Park area, which can also be seen in Figure 41 to Figure 43). Regions located in the steeper areas bordering the catchment boundary are not affected as much by these increases in flood levels as the flatter regions in the centre of the catchment.

Table 22: Results for Rainfall Increase Scenarios

ID	Location	Base Case (mAHD)	Impact (m)		
			10% Increase in Rainfall	20% Increase in Rainfall	30% Increase in Rainfall
1	Chalmers St	30.19	0.02	0.05	0.07
2	Cleveland St between Chalmers & Pit	31.24	0.01	0.01	0.01
3	Regent St between Cleveland & Lawson	24.05	0.03	0.04	0.06
4	Regent St	18.55	0.04	0.06	0.08
5	Boundary St / Ivy St	14.43	0.07	0.12	0.19
6	Beaumount St / Cleveland St	11.92	0.04	0.07	0.13
7	Abercrombie St near Blackfriars St	11.56	0.00	0.00	0.01
8	Parramatta Rd / Buckland St	9.02	0.06	0.11	0.17
9	Blackwattle Ln near Small St	5.77	0.04	0.07	0.11
10	Blackwattle Ln / Kelly St	5.05	0.06	0.12	0.18
11	Blackwattle Ln / Macarthur St	4.91	0.07	0.13	0.19
12	Bay St / Wentworth Park Rd	3.44	0.07	0.13	0.19
13	William Henry / Wattle St	3.74	0.04	0.08	0.13
14	Wattle St / Fig St	2.86	0.05	0.11	0.17
15	Wattle St (near tramline)	2.86	0.05	0.11	0.17
16	Campbell St	13.80	0.06	0.10	0.13
17	Darghan Ln / Mitchell St East	3.96	0.03	0.07	0.10
18	Wentworth Park Rd / Mitchell St	3.23	0.07	0.13	0.20
19	Wentworth Park Rd / Lyndhurst St	2.94	0.04	0.08	0.12
20	Talfourd St	17.07	0.02	0.02	0.04
21	Bridge Rd (underneath tramline)	3.31	0.01	0.03	0.05
22	Bridge Rd / Wentworth Park Rd	2.54	0.05	0.09	0.12
		Mean:	+0.04	+0.08	+0.12

6.9.2. Sea Level Rise

The results for the sea level rise scenarios are tabulated in Table 23. The impacts of increasing downstream water levels are largely confined to the low lying areas adjacent to Blackwattle Bay, as illustrated in Figure 44 and Figure 45. A 0.4 metre sea level rise is unlikely to have any effects on the peak flood levels whereas a 0.9 metre sea level rise may have minor implications on peak flood levels in the vicinity of the seawalls as well as Wentworth Park but is unlikely to be of concern in other regions of the catchment.

Table 23: Results for Sea Level Rise Scenario

ID	Location	Base Case (mAHD)	Impact (m)	
			Sea Level Rise of 0.4m	Sea Level Rise of 0.9m
1	Chalmers St	30.19	0.00	0.00
2	Cleveland St between Chalmers & Pit	31.24	0.00	0.00
3	Regent St between Cleveland & Lawson	24.05	0.00	0.00
4	Regent St	18.55	0.00	0.00
5	Boundary St / Ivy St	14.43	0.00	0.01
6	Beaumount St / Cleveland St	11.92	0.00	0.00
7	Abercrombie St near Blackfriars St	11.56	0.00	0.00
8	Parramatta Rd / Buckland St	9.02	0.00	0.00
9	Blackwattle Ln near Small St	5.77	0.00	0.00
10	Blackwattle Ln / Kelly St	5.05	0.00	0.01
11	Blackwattle Ln / Macarthur St	4.91	0.00	0.01
12	Bay St / Wentworth Park Rd	3.44	0.03	0.05
13	William Henry / Wattle St	3.74	0.00	0.00
14	Wattle St / Fig St	2.86	0.01	0.04
15	Wattle St (near tramline)	2.86	0.01	0.04
16	Campbell St	13.80	0.01	0.00
17	Darghan Ln / Mitchell St East	3.96	0.00	0.00
18	Wentworth Park Rd / Mitchell St	3.23	0.02	0.06
19	Wentworth Park Rd / Lyndhurst St	2.94	0.02	0.04
20	Talfourd St	17.07	-0.01	0.00
21	Bridge Rd (underneath tramline)	3.31	-0.01	0.00
22	Bridge Rd / Wentworth Park Rd	2.54	0.02	0.04
		Mean:	+0.01	+0.01

6.10. Flood Damages Assessment

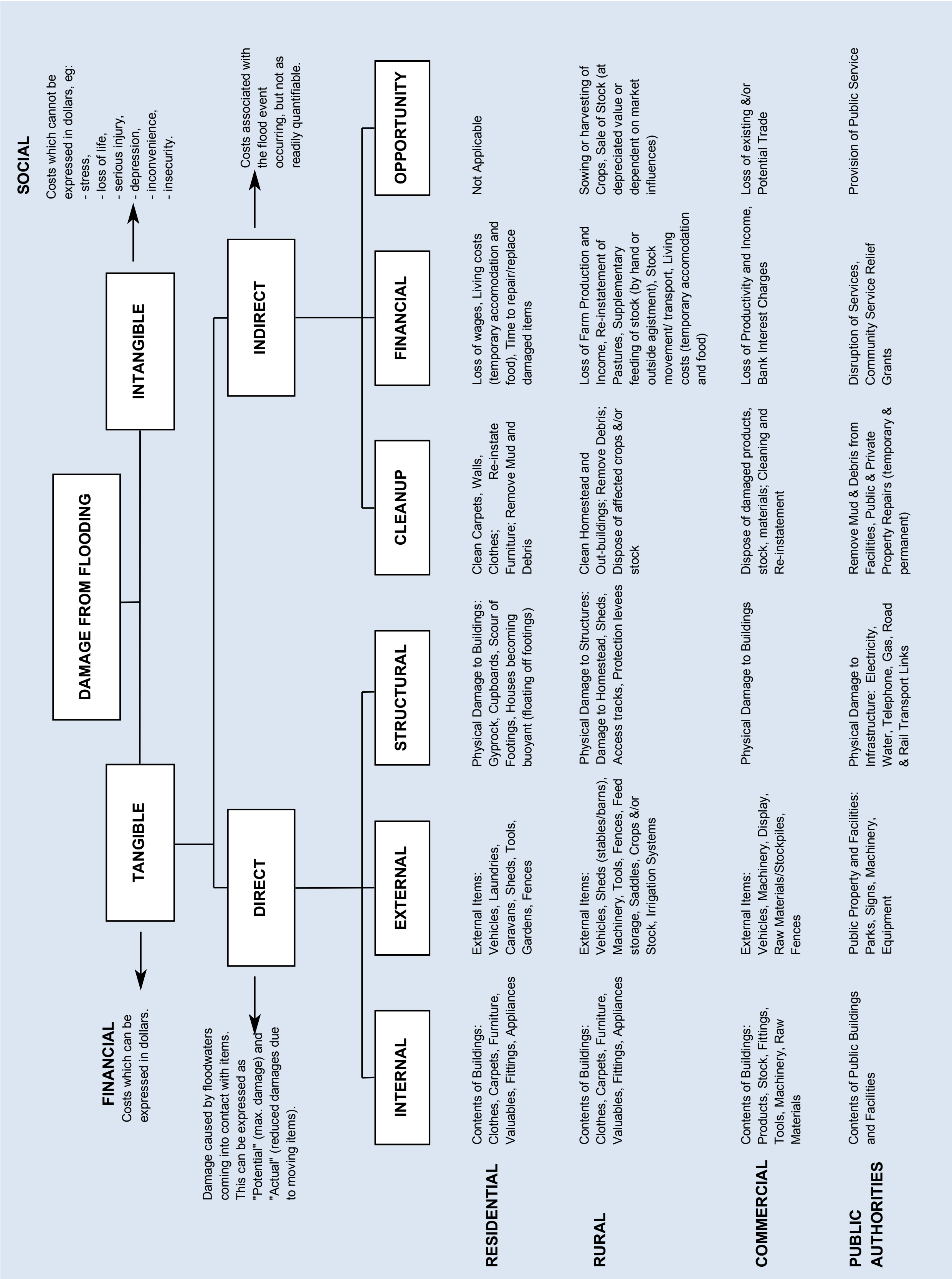
Flood impact can be quantified in the calculation of flood damages. Flood damage calculations do not include all impacts associated with flooding. They do, however, provide a basis for assessing the economic loss of flooding and also a non-subjective means of assessing the merit of flood mitigation works such as retarding basins, levees, drainage enhancement etc. The quantification of flood damages is an important part of the floodplain risk management process. By quantifying flood damage for a range of design events, appropriate cost effective management measures can be analysed in terms of their benefits (reduction in damages) versus the cost of implementation. The cost of damage and the degree of disruption to the community caused by flooding depends upon many factors including:

- The magnitude (depth, velocity and duration) of the flood;
- Land use and susceptibility to damages;
- Awareness of the community to flooding;
- Effective warning time;
- The availability of an evacuation plan or damage minimisation program;
- Physical factors such failure of services (sewerage), 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 for which a monetary value can be easily assigned, while intangible damages are those to which a monetary value cannot easily be attributed. Types of flood damages are shown in Diagram 2.

The assessment of flood damages not only looks at potential costs due to flooding but also identifies when properties are likely to become flood affected by either flooding on the property or by over floor flooding as shown on Figure 46.

Diagram 2: Flood Damages Categories (including damage and losses from permanent inundation)



6.10.1. Tangible Flood Damages

Tangible flood damages are comprised of two basic categories; direct and indirect damages (refer Diagram 2). Direct damages are caused by floodwaters wetting goods and possessions thereby damaging them and resulting in either costs to replace or repair or in a reduction to their value. Direct damages are further classified as either internal (damage to the contents of a building including carpets, furniture), structural (referring to the structural fabric of a building such as foundations, walls, floors, windows) or external (damage to all items outside the building such as cars, garages). Indirect damages are the additional financial losses caused by the flood for example the cost of temporary accommodation, loss of wages by employees etc.

Given the variability of flooding and property and content values, the total likely damages figure in any given flood event is useful to get a feel for the magnitude of the flood problem, however it is of little value for absolute economic evaluation. However, damages estimates are useful when studying the economic effectiveness of proposed mitigation options. Understanding the total damages prevented over the life of the option in relation to current damages, or to an alternative option, can assist in the decision making process.

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 account the probability of a flood occurrence. This means the smaller floods, which occur more frequently, are given a greater weighting than the rare catastrophic floods.

Floor level survey was undertaken to quantify the damages caused by inundation for existing development. As part of this floor level survey work an indicative ground level was recorded for use in the damages assessment. This was used in conjunction with the flood level information for design events as established in this study. Damages calculations were carried out for all properties which floor level was surveyed, with the selection criteria as described in Appendix B. It should be noted that by including only a selection of properties primarily in the 1% AEP extent, properties that are inundated in rarer events have not been accounted for. Therefore damage calculations for the PMF event are likely to be underestimated. It was not considered viable to survey all properties within the PMF extent for the purpose of damage calculations.

A flood damages assessment was undertaken herein for existing development in accordance with current OEH guidelines (Reference 16) and the Floodplain Development Manual (Reference 7). The damages were calculated using a number of height-damage curves which relate the depth of water above the floor with tangible damages. Each component of tangible damages is allocated a maximum value and a maximum depth at which this value occurs. Any flood depths greater than this allocated value do not incur additional damages as it is assumed that, by this level, all potential damages have already occurred.

Damages were calculated for residential and commercial\industrial properties separately and the process and results are described in the following sections. The combined results are provided as Table 24. This flood damages estimate does not include the cost of restoring or maintaining public services and infrastructure. It should be noted that damages calculations do not take into account flood damages to any basements or cellars, hence where properties have basements damages can be under estimated.

Table 24: Estimated Combined Flood Damages for Blackwattle Bay Catchment

Event	Number of Properties Flood Affected	No. of Properties Flooded Above Floor Level	Total Tangible Flood Damages	Average Tangible Damages Per Flood Affected Property
2 year ARI	202	94	$ 8,851,400	$ 43,900
5 year ARI	236	112	$ 11,010,900	$ 46,700
10% AEP	246	131	$ 12,258,600	$ 49,900
5% AEP	259	141	$ 13,526,500	$ 52,300
2% AEP	268	163	$ 14,627,600	$ 54,600
1% AEP	283	171	$ 16,229,800	$ 57,400
PMF	307	255	$ 25,050,200	$ 81,600
		Average Annual Damages (AAD)	**$ 7,783,100**	**$ 25,400**

* Excludes all damages to public assets but includes external damages that may or may not occur with building floor inundation.

6.10.1.1.Residential Properties

Flood damages assessment for residential development was undertaken in accordance with OEH guidelines (Reference 16). For residential properties, external damages (damages caused by flooding below the floor level) were set at $6,700 and additional costs for clean-up as $4,000. For additional accommodation costs or loss of rent a value of $220 per week was allowed assuming that the property would have to be unoccupied for up to three weeks. Internal (contents) damages were allocated a maximum value of $37,500 occurring at a depth of 2 m above the building floor level (and linearly proportioned between the depths of 0 to 2 m). Structural damages vary on whether the property is slab/low set or high set. For the purpose of this study, any property with a floor level of 0.5 m or more above ground level was assumed to be high set. For two storey properties, damages (apart from external damages) are reduced by a factor of 70% where only the ground floor is flooded as it is assumed some contents will be on the upper floor and unaffected and that structural damage costs will be less. In some instances external damage may occur even where the property is not inundated above floor level and therefore tangible damages include external damages which may occur with or without house floor inundation.

A summary of the residential flood damages for the Blackwattle Bay catchment is provided in Table 25. Overall, for residential properties in the catchment there is little difference in the average tangible damages per property for all the design events analysis up to the 1% AEP event. This is reflective of the relatively small differences in flood levels between the design flood events. Average damage per property increases at events larger than the 1% AEP when more properties become flooded above floor level. Note that the terminology used refers to a property or lot being the land within the ownership boundary. Flooding of a property does not necessarily mean flooding above floor level of a building on that property/lot.

Table 25: Estimated Residential Flood Damages for Blackwattle Bay Catchment

Event	Number of Properties Flood Affected	No. of Properties Flooded Above Floor Level	Total Tangible Flood Damages	Average Tangible Damages Per Flood Affected Property
2 year ARI	162	71	$ 3,460,100	$ 21,400
5 year ARI	189	82	$ 4,085,500	$ 21,700
10% AEP	197	96	$ 4,563,800	$ 23,200
5% AEP	206	102	$ 4,907,700	$ 23,900
2% AEP	212	120	$ 5,393,700	$ 25,500
1% AEP	226	127	$ 5,954,700	$ 26,400
PMF	248	202	$ 9,731,200	$ 39,300
Average Annual Damages (AAD)			**$ 2,955,700**	**$ 12,000**

6.10.1.2. Commercial and Industrial Properties

The tangible flood damage to commercial and industrial properties is more difficult to assess. Commercial and industrial damage estimates are more uncertain and larger than residential damages. Commercial and industrial damage estimates can vary significantly depending on:

- Type of business – stock based or not;
- Duration of flooding – affects how long a business may be closed for not just whether the business itself if closed but when access to it becomes available;
- Ability to move stock or assets before onset of flooding - some large machinery will not be able to moved and in other instances there may be no sufficient warning time to move stock to dry locations; and
- Ability to transfer business to a temporary location.

Costs to business can occur for a range of reasons, some of which will affect some businesses more than others dependent on the magnitude of flooding and the type of businesses. Common flood costs to businesses are:

- Removal and storage of stock before a flood if warning is given;
- Loss of production – caused by damaged stock, assets and availability of staff;
- Loss of stock and/or assets;
- Reduced stock through reduced or no supplies;
- Trade loss – by customers not being able to access the business or through business closure;
- Cost of replacing damages or lost stock or assets; and
- Clean-up costs.

No specific guidance is available for assessing flood damages to non-residential properties. Therefore for this study, commercial and industrial damages were calculated using the methodology for residential properties but with the costs/damages increased to a value which is consistent with commercial/industrial development. For example, the maximum value of internal (contents) damages was increased to $250,000 since the building contents are of higher value whilst loss of rent was set at $3,000 per week to account for the loss of business through having to close for a period. Flooding below floor level uses the same damages curve as the residential properties.

Though the original OEH guidelines for flood damages calculations are not applicable to non-residential properties, they can still be used to create comparable damage figures. The damages value figure should not be taken as an actual likely cost rather it is useful when comparing potential management options and for benefit-cost analysis.

A summary of the commercial/industrial flood damages for the Blackwattle Bay catchment is provided in Table 26. AAD for the surveyed commercial/industrial properties is nearly twice than that for residential properties despite the number of flood affected properties for the latter being 4 times more than that of the former. This reflects the higher costs that businesses would incur compared to residential dwellings when flooded above floor level. On a per property basis the AAD is approximately 6.8 times higher when comparing the commercial/industrial properties against the residential properties.

Table 26: Estimated Commercial and Industrial Flood Damages for Blackwattle Bay Catchment

Event	Number of Properties Flood Affected	No. of Properties Flooded Above Floor Level	Total Tangible Flood Damages	Average Tangible Damages Per Flood Affected Property
2 year ARI	40	23	$ 5,391,300	$ 134,800
5 year ARI	47	30	$ 6,925,500	$ 147,400
10% AEP	49	35	$ 7,694,900	$ 157,100
5% AEP	53	39	$ 8,618,900	$ 162,700
2% AEP	56	43	$ 9,234,000	$ 164,900
1% AEP	57	44	$ 10,275,200	$ 180,300
PMF	59	53	$ 15,319,000	$ 259,700
		Average Annual Damages (AAD)	**$ 4,827,400**	**$ 81,900**

6.10.2. Intangible Flood Damages

The intangible damages associated with flooding, by their nature, are inherently more difficult to estimate in monetary terms. In addition to the tangible damages discussed previously, additional costs/damages are incurred by residents affected by flooding, such as stress, risk/loss to life, injury, loss of sentimental items etc. It is not possible to put a monetary value on the intangible damages as they are likely to vary dramatically between each flood (from a negligible amount to several hundred times greater than the tangible damages) and depend on a range of factors such as the size of flood, the individuals affected, and community preparedness. However, it is still important that the consideration of intangible damages is included when considering the impacts of flooding on a community.

Post flood damages surveys have linked flooding to stress, ill-health and trauma for the residents. For example the loss of memorabilia, pets, insurance papers and other items without fixed costs and of sentimental value may cause stress and subsequent ill-health. In addition flooding may affect personal relationships and lead to stress in domestic and work situations. In addition to the stress caused during an event (from concern over property damage, risk to life for the individuals or their family, clean up etc.) many residents who have experienced a major flood are fearful of the occurrence of another flood event and the associated damage. The extent of the stress depends on the individual and although the majority of flood victims recover, these effects can lead to a reduction in quality of life for the flood victims.

During any flood event there is the potential for injury as well as loss of life due to causes such as drowning, floating debris or illness from polluted water. Generally, the higher the flood velocities and depths the higher the risk. Within the Blackwattle Bay catchment area, the high hazard areas mostly consist of the major roads which serve as overland flow paths for floodwaters to discharge downstream. However, there will always be local high risk (high hazard) areas where flows may be concentrated around buildings or other structures within low hazard areas.

7. CONCLUSIONS

A flood study, reported upon herein, has been undertaken for the Blackwattle Bay catchment. Mechanisms of flooding addressed include local overland flow (runoff in excess of pit/pipe drainage systems) as well as backwater flooding from receiving waters. The flood study has defined flood behaviour for a range of floods from the 2Y ARI to the PMF event and the results are presented herein.

The work carried out for this study has been verified by a limited calibration exercise to historical data (26th January 1991 calibration, 17th February 1993 verification) and is based on best practice and produces results for inundation and flows which are in line with previous investigations and expectations (References 1 and 2) as well as the information provided from resident surveys.

There is an extensive flood liability throughout the study area as a result of extensive development (filling of the floodplain and blocking of flow paths) in conjunction with pervious surfaces converted to impervious surfaces. The restricted overland flow paths running south to north in the centre of the catchment exacerbate the flood liability of the area. In addition the minor and major drainage systems are of limited capacity.

Through the course of the study a number of areas were identified which were prone to inundation of property and/or house habitable floor levels. Subsequent floor level survey and damages assessment identified 171 properties that are liable to over floor inundation in the 1% AEP event. Notably 94 properties are also flood liable (over floor inundation) in smaller events such as the 2 year ARI event although this estimate is conservative given the prudent blockage assumption.

A number of hotspots were also identified in the study, i.e. locations where a number of properties are flood liable due to a shared flooding mechanism. These include:

1. Intersection of Cleveland St and Beaumount St – the intersection is located on a depression that collects water from Cleveland St and the upstream areas of Boundary St. The adjacent commercial properties are inundated above floor levels for events smaller than the 10% AEP event;
2. Intersection of Parramatta Rd and Buckland St – the shops located along Parramatta Rd pose as a major obstruction for the overland flows coming from Buckland St and flooding here is worsened by the limited capacity of the local trunk drainage system. The majority of the commercial properties are subject to over floor inundation for the smallest event modelled, i.e. the 2Y ARI event;
3. Wentworth Park Rd/William Henry St - majority of the catchment runoff must first flow around Wentworth Park before it can reach Blackwattle Bay. As the Park is slightly raised from its surrounds, flows tend to travel along the bordering roads and not through the Park itself;
4. Wattle St – this road serves as a major overland flow path for upstream waters to reach the outlet (the other being Wentworth Park Rd). Note however that inundation above floor levels

of adjoining properties only occurs for the PMF event;

5. Properties off Mitchell St and Talfourd St – a large numbers of residential properties here are subject to over floor inundation for various flood events due to their location on a local depression where a trunk drainage system is also located; and
6. Bridge Rd – like Wattle St, this road serves as a major overland flow path for upstream waters to reach the outlet and the relatively steep gradient of the road means that high flow velocities can be expected at this location.

The subsequent Floodplain Risk Management Study and Plan will seek to address the flood liability of properties identified within the course of the study.

8. ACKNOWLEDGEMENTS

WMAwater wish to acknowledge the assistance of the City of Sydney staff and Floodplain Management Committee in carrying out this study as well as the NSW Government (Office of Environment and Heritage) and the residents of the Blackwattle Bay catchment. This study was jointly funded by the City of Sydney and the NSW Government.

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Figures

FIGURE 1
LOCALITY MAP

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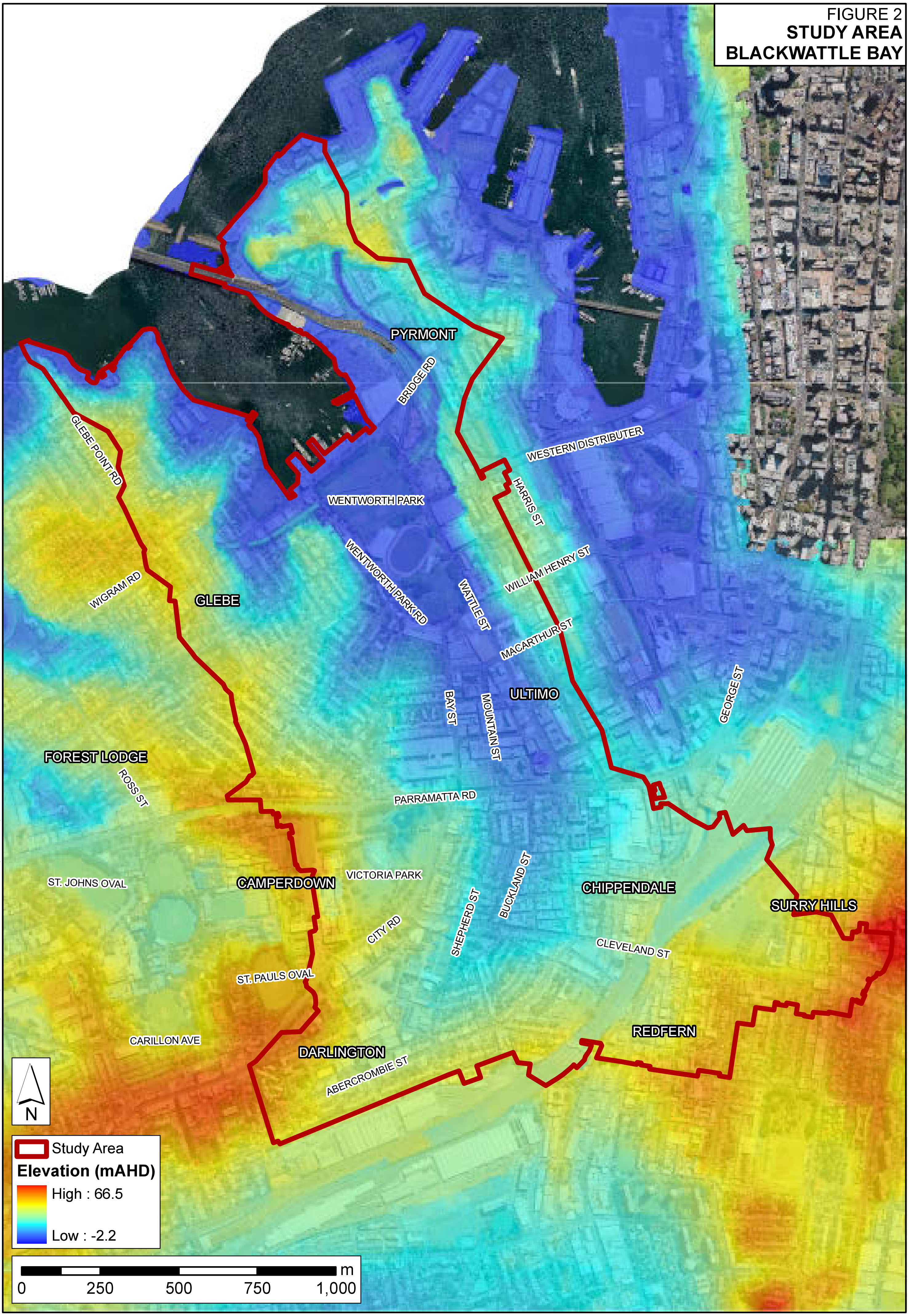

FIGURE 2
STUDY AREA
BLACKWATTLE BAY
PYRMONT
BRIDGE RD
GLEBE POINT RD
WESTERN DISTRIBUTER
WENTWORTH PARK
HARRIS ST
WENTWORTH PARK RD
WILLIAM HENRY ST
WIGRAM RD
GLEBE
WATTLE ST
MACARTHUR ST
ULTIMO
GEORGE ST
BAY ST
MOUNTAIN ST
FOREST LODGE
ROSS ST
PARRAMATTA RD
ST. JOHNS OVAL
CAMPERDOWN
VICTORIA PARK
BUCKLAND ST
CHIPPENDALE
SURRY HILLS
SHEPHERD ST
CITY RD
CLEVELAND ST
ST. PAULS OVAL
CARILLON AVE
REDFERN
DARLINGTON
ABERCROMBIE ST
N
Study Area
Elevation (mAHD)
High : 66.5
Low : -2.2
m
0 250 500 750 1,000
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FIGURE 3
DRAINAGE SYSTEM & HYDRAULIC MODEL LAYOUT
BLACKWATTLE BAY

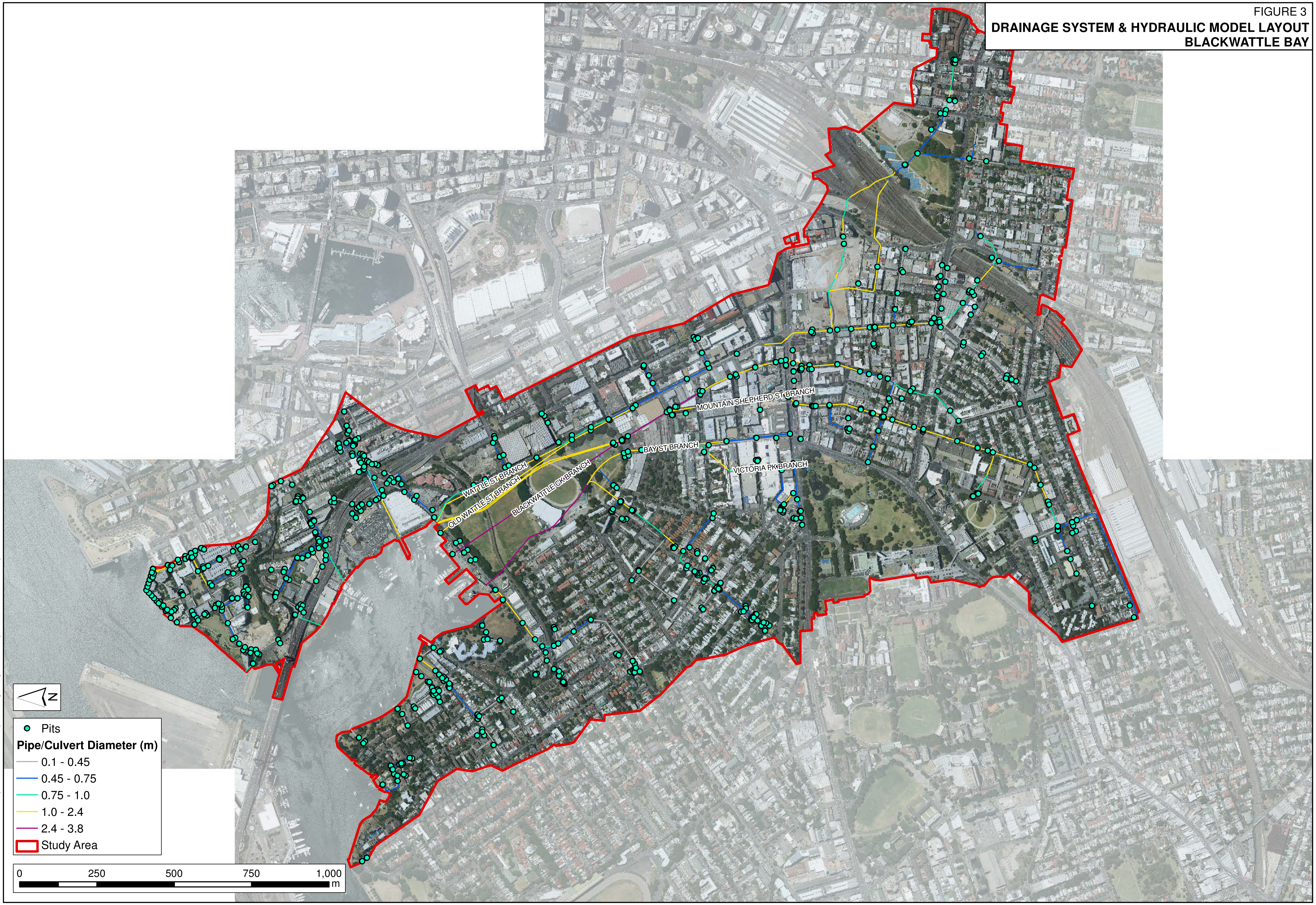

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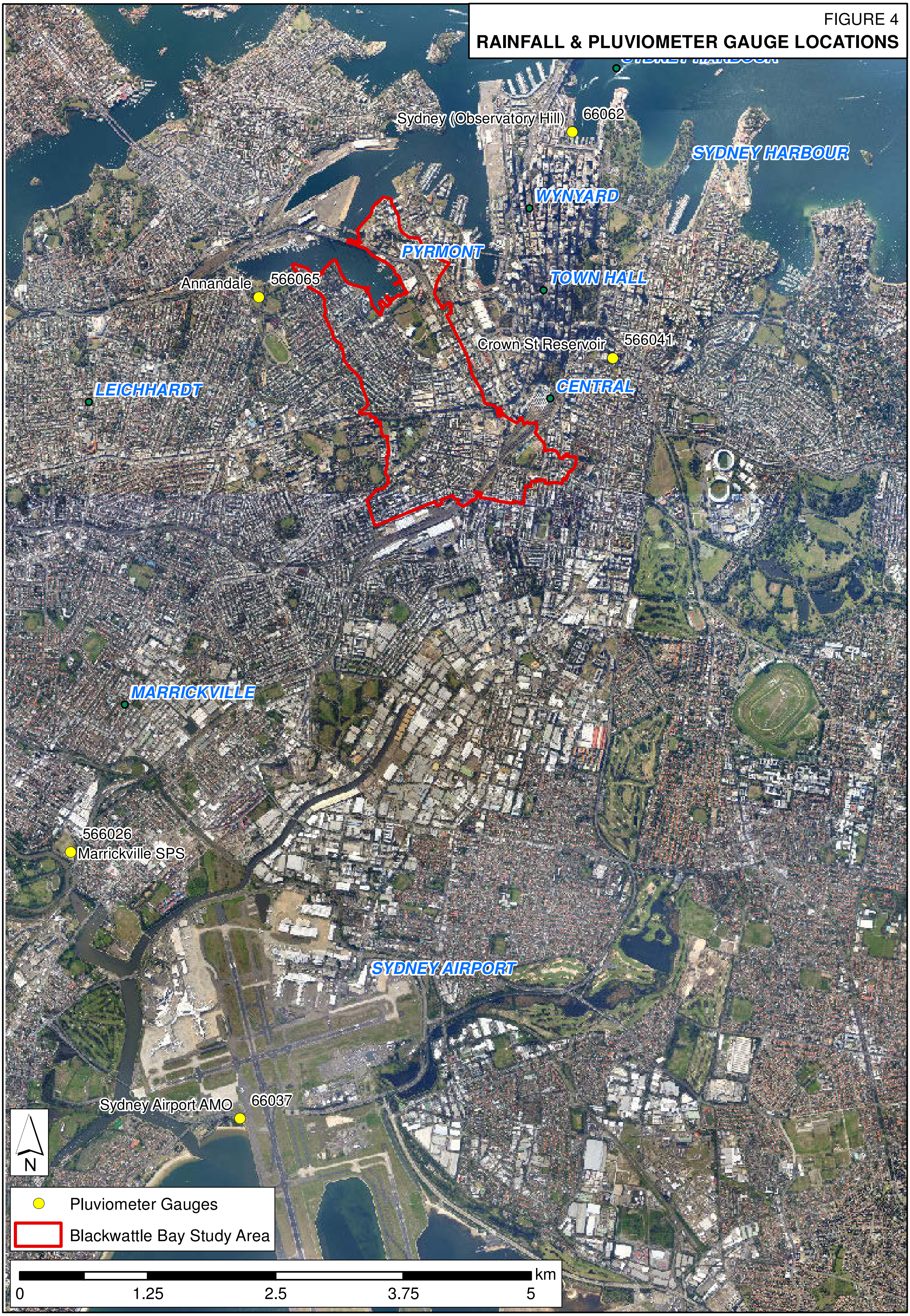
FIGURE 4
RAINFALL & PLUVIOMETER GAUGE LOCATIONS



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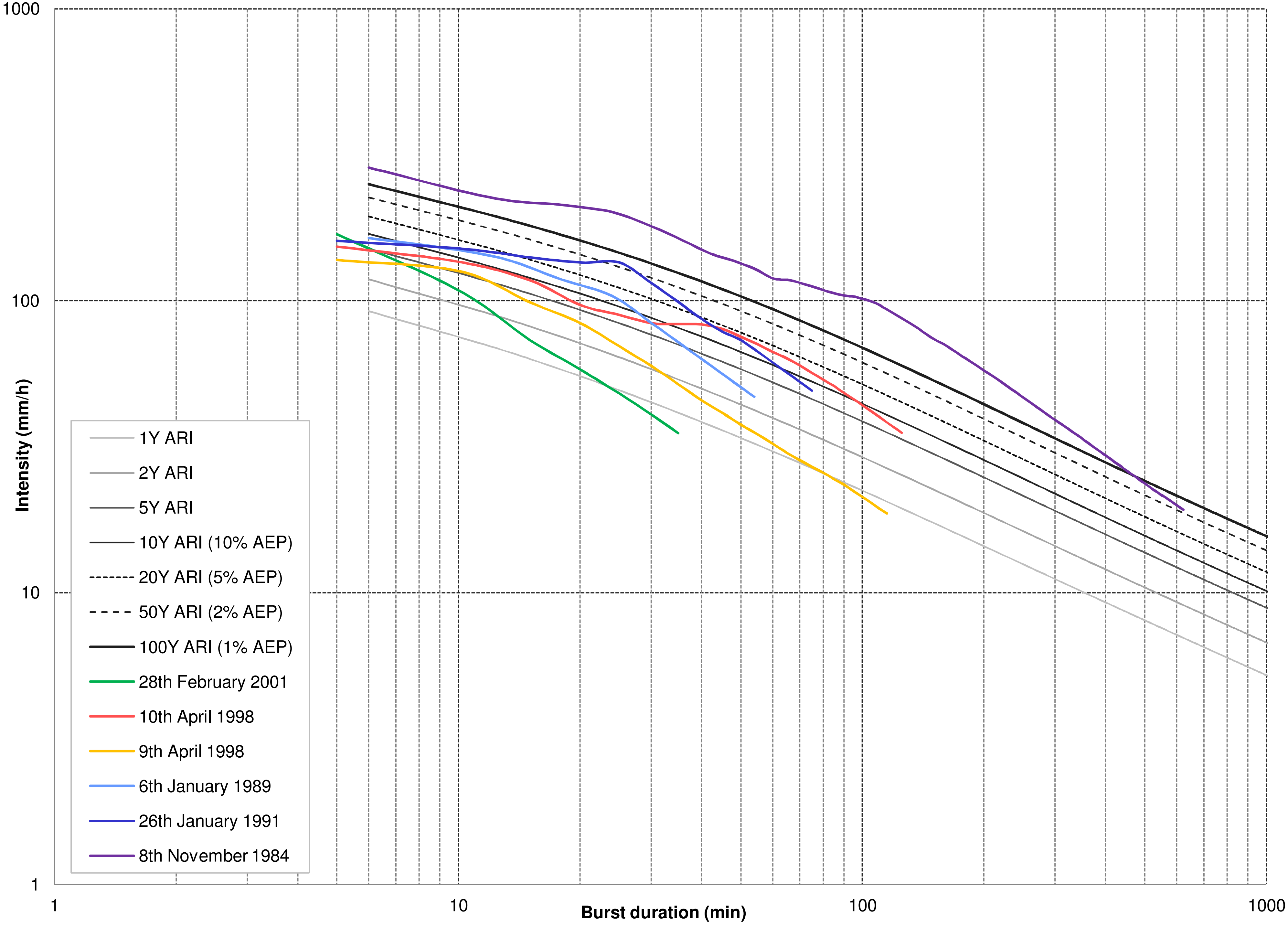

FIGURE 5
RAINFALL BURST AND FREQUENCY
SYDNEY OBSERVATORY HILL GAUGE

FIGURE 6
COMMUNITY CONSULTATION RESPONDENTS: BLACKWATTLE BAY

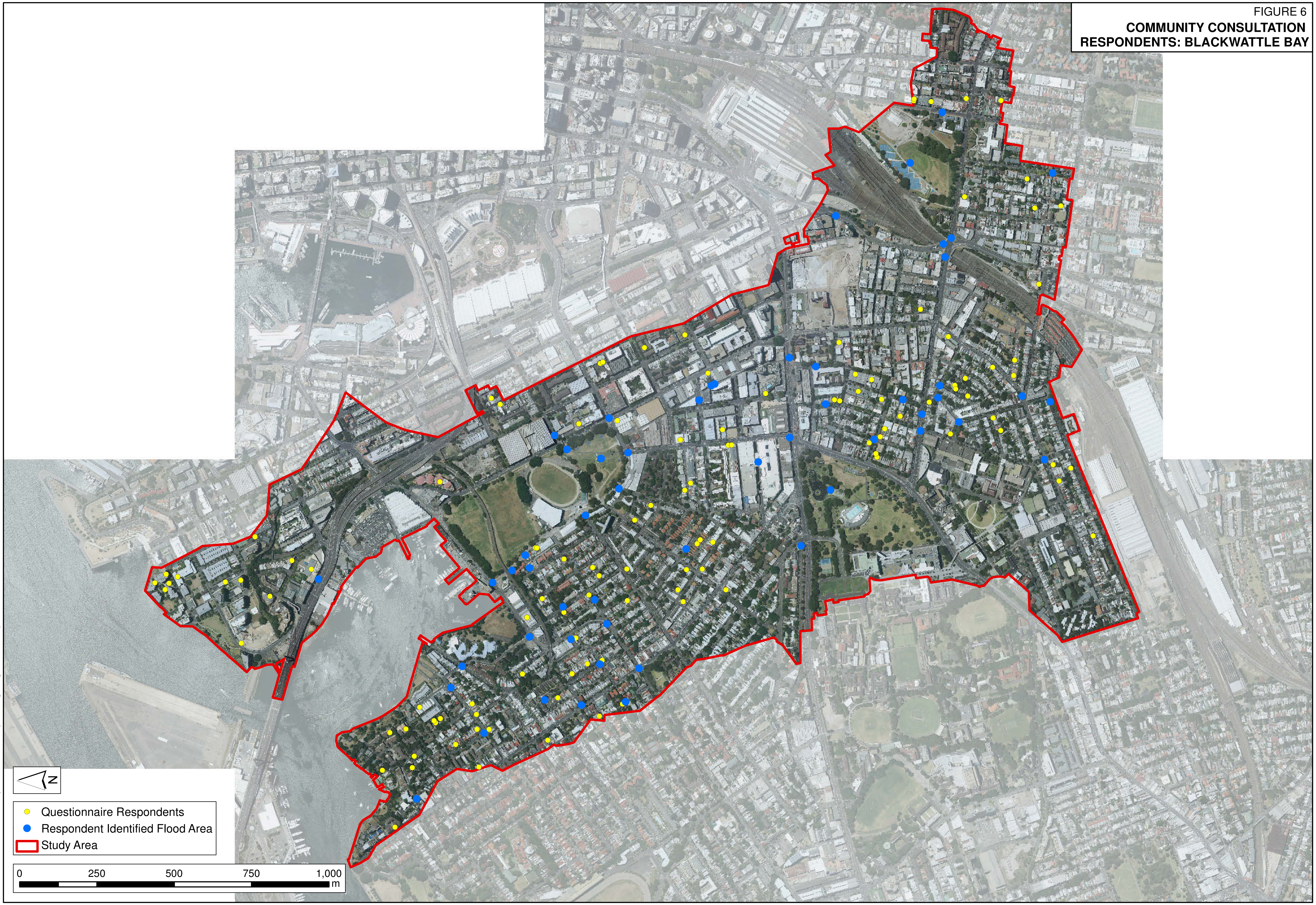

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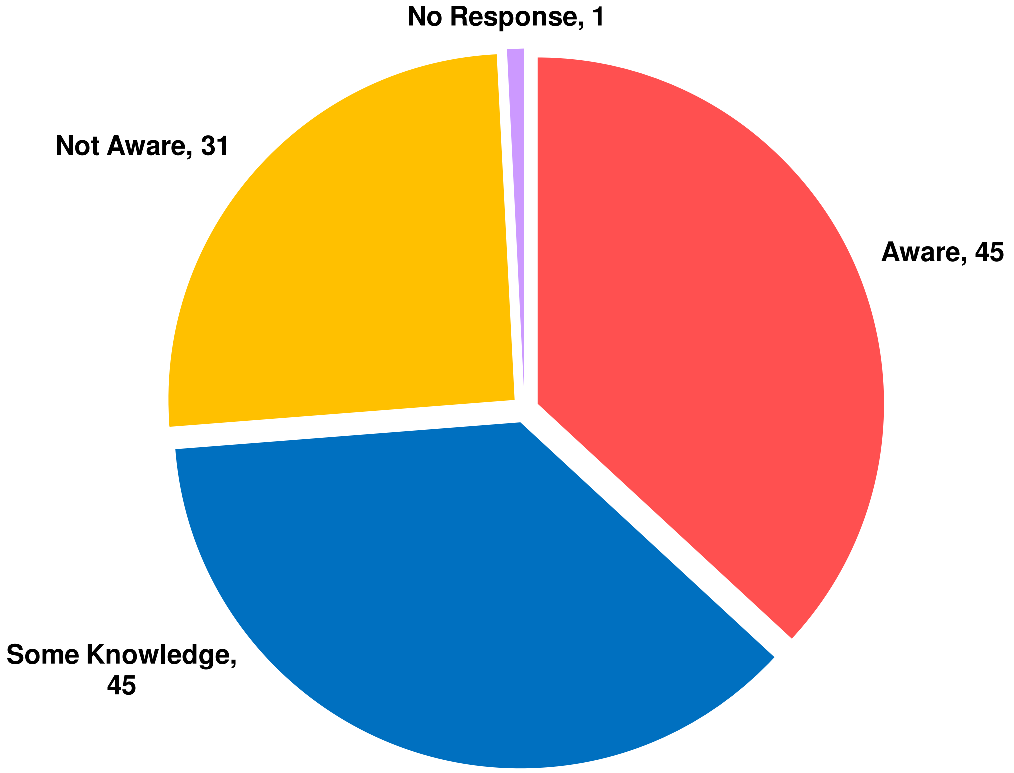
FIGURE 7A
COMMUNITY CONSULTATION RESULTS
BLACKWATTLE BAY

A: Awareness of Stormwater Flooding from Streets or Channels



B: Experience of Previous Floods

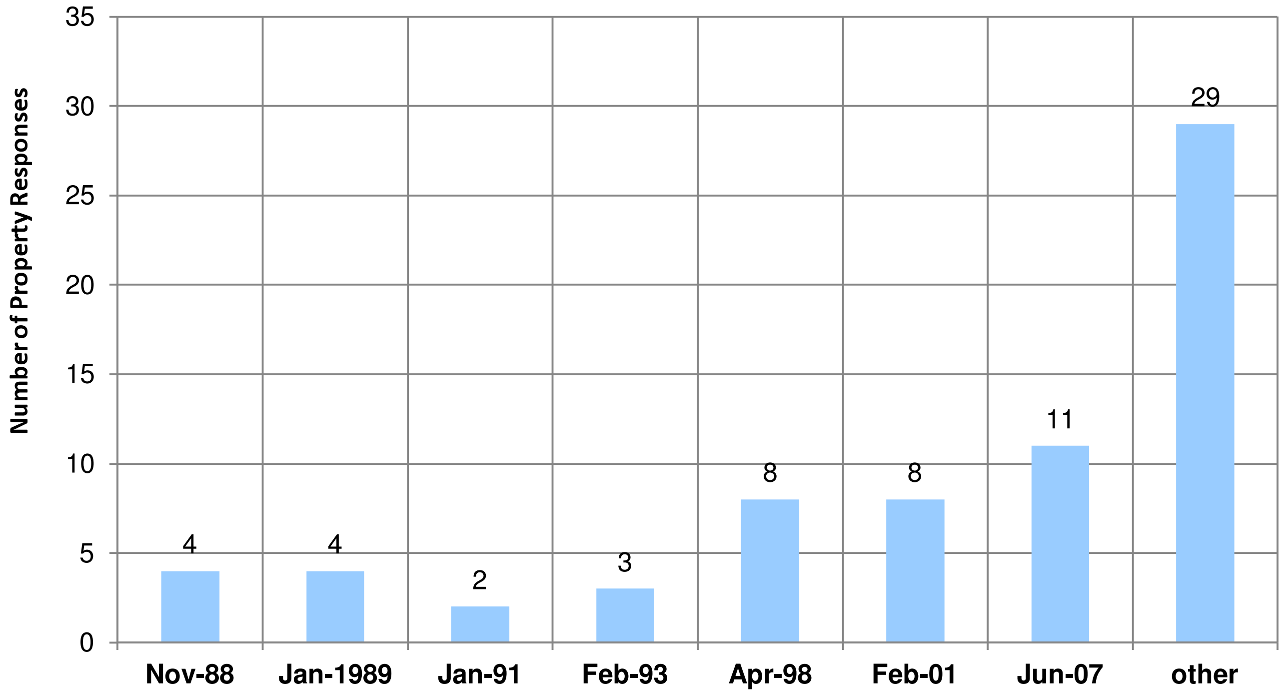
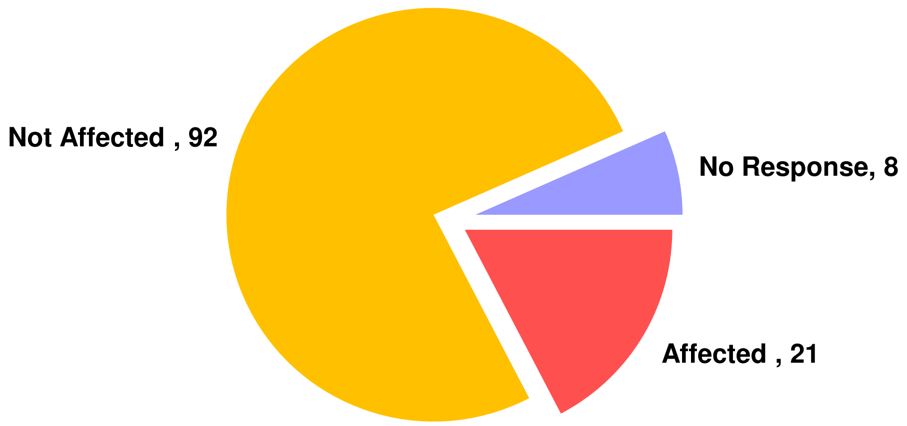
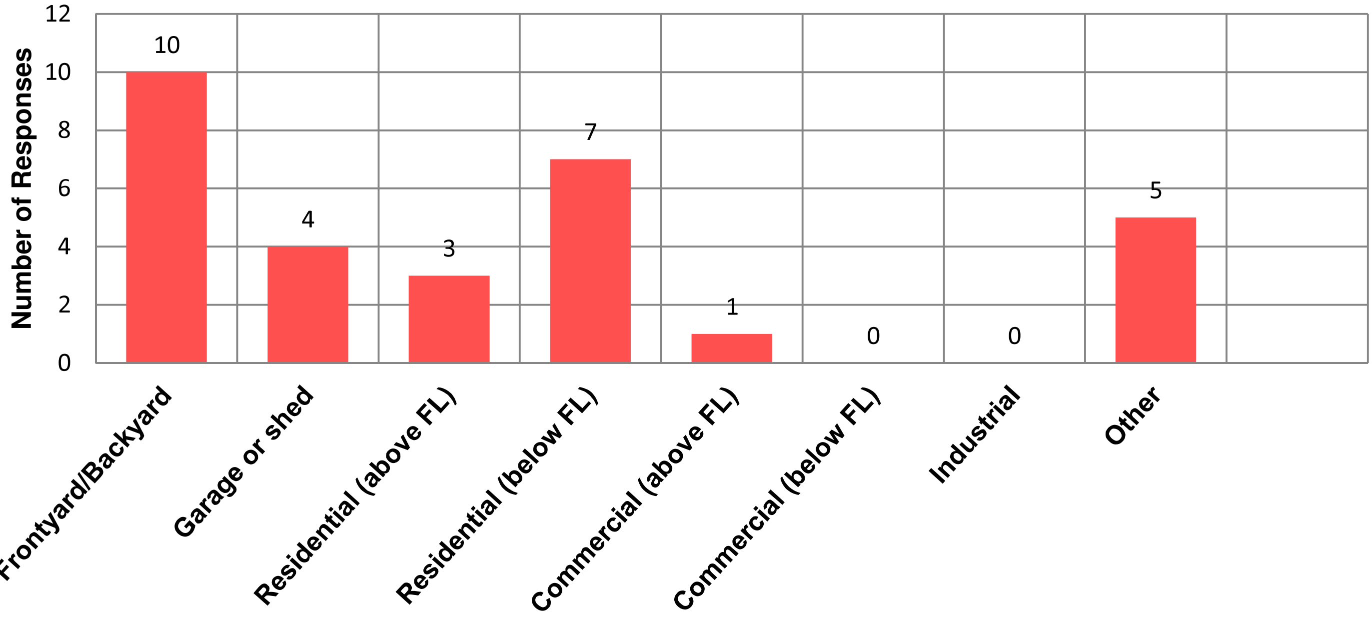


FIGURE 7B
COMMUNITY CONSULTATION RESULTS
BLACKWATTLE BAY

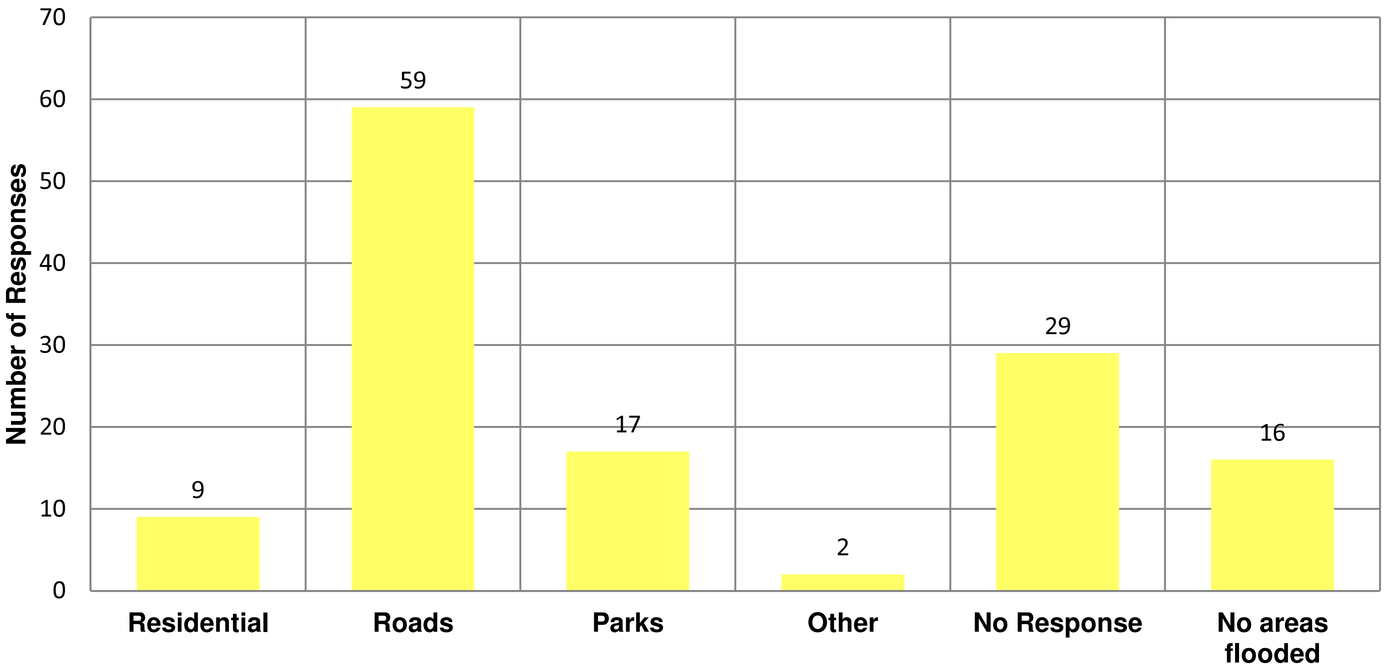
C: Homes Flooded

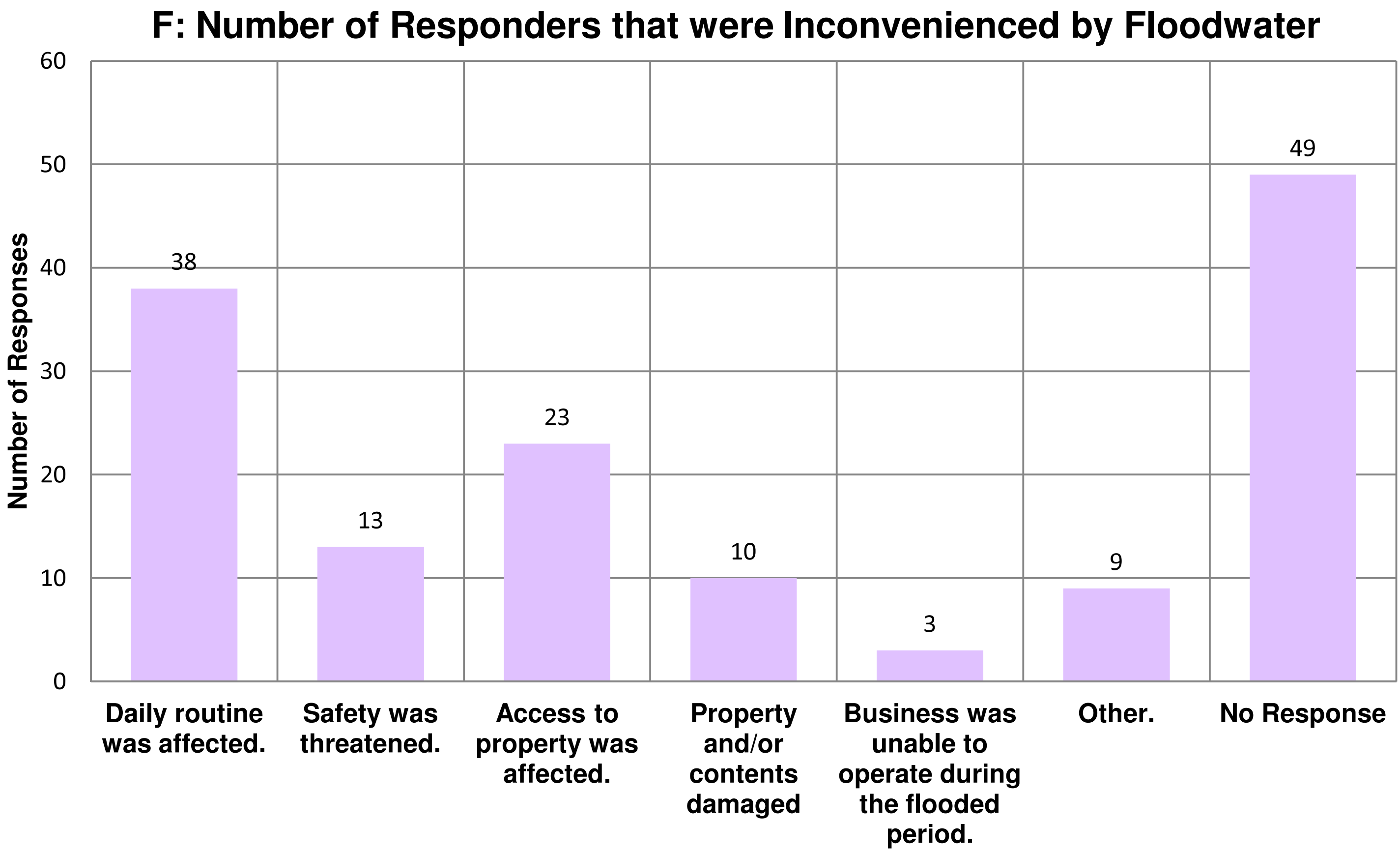


D: What part of the property was flooded?



E: Other areas flooded





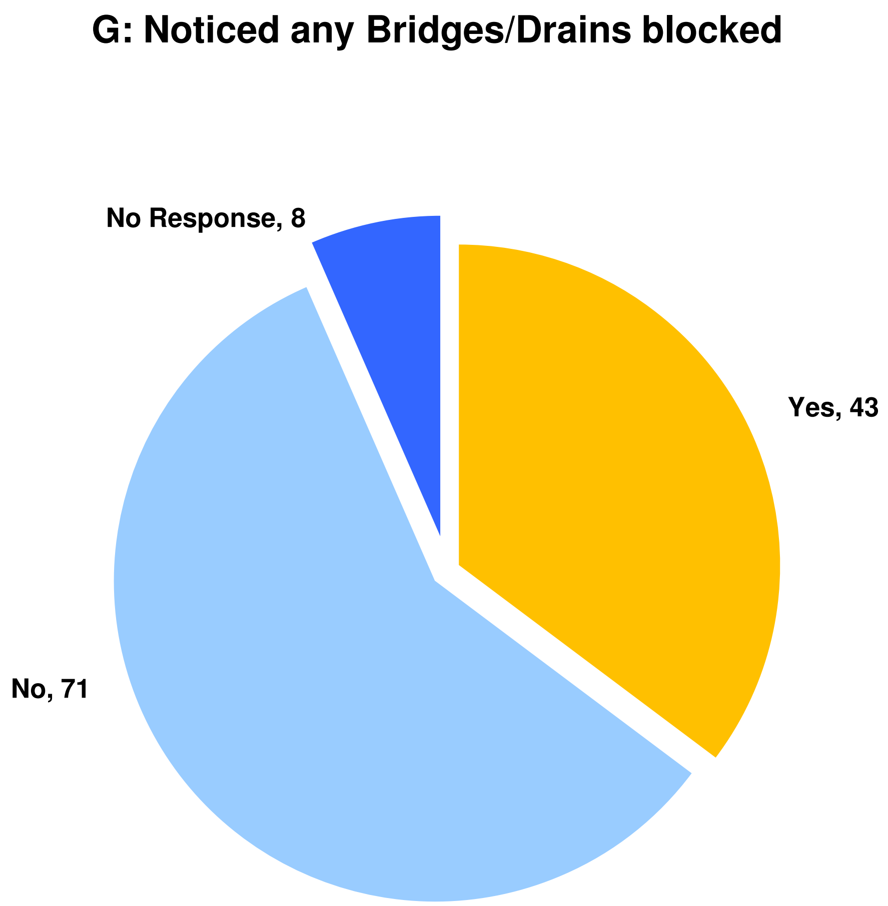


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FIGURE 8
HYDROLOGICAL SUBCATCHMENTS BLACKWATTLE BAY

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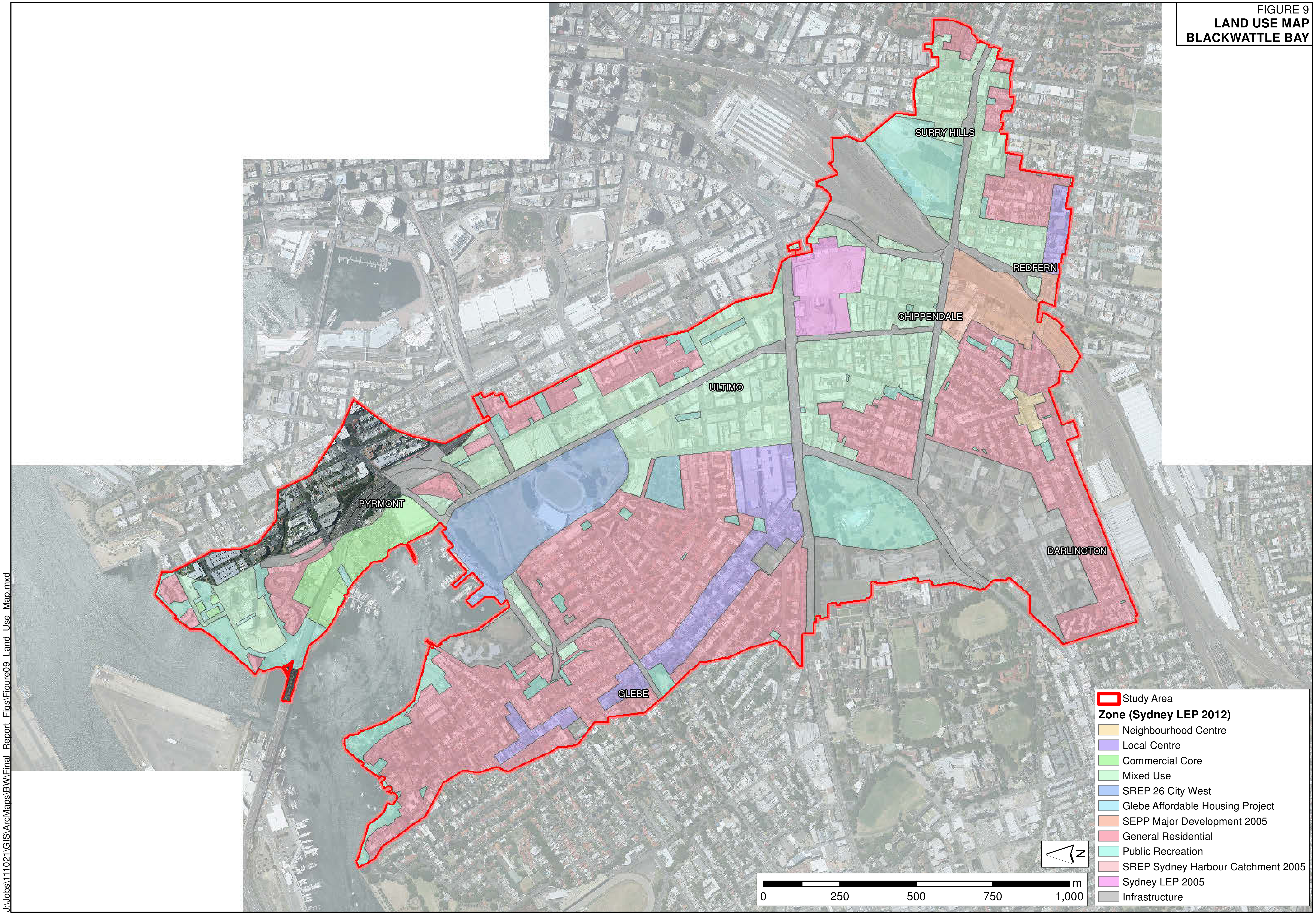
FIGURE 9
LAND USE MAP
BLACKWATTLE BAY



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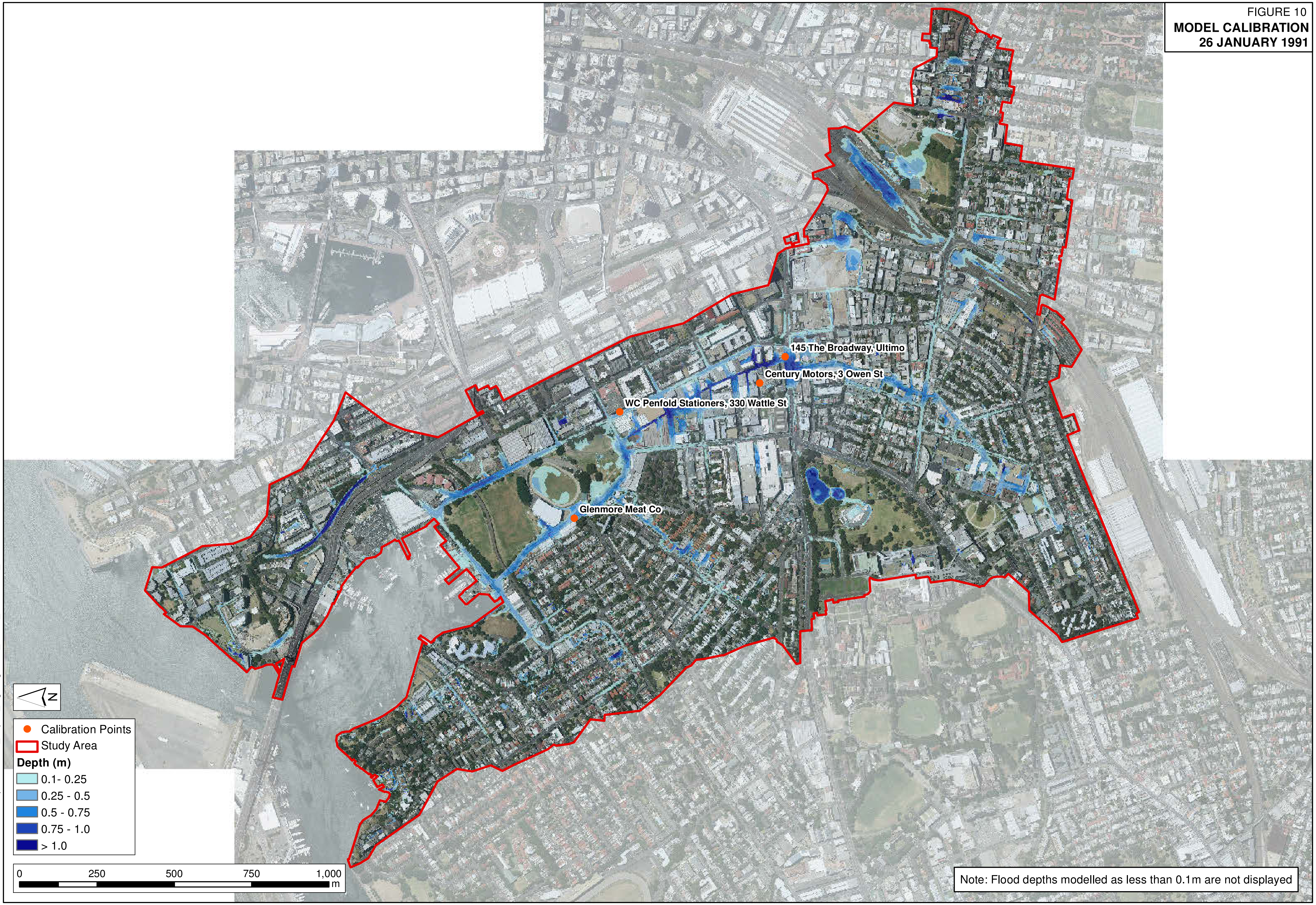

FIGURE 10
MODEL CALIBRATION
26 JANUARY 1991

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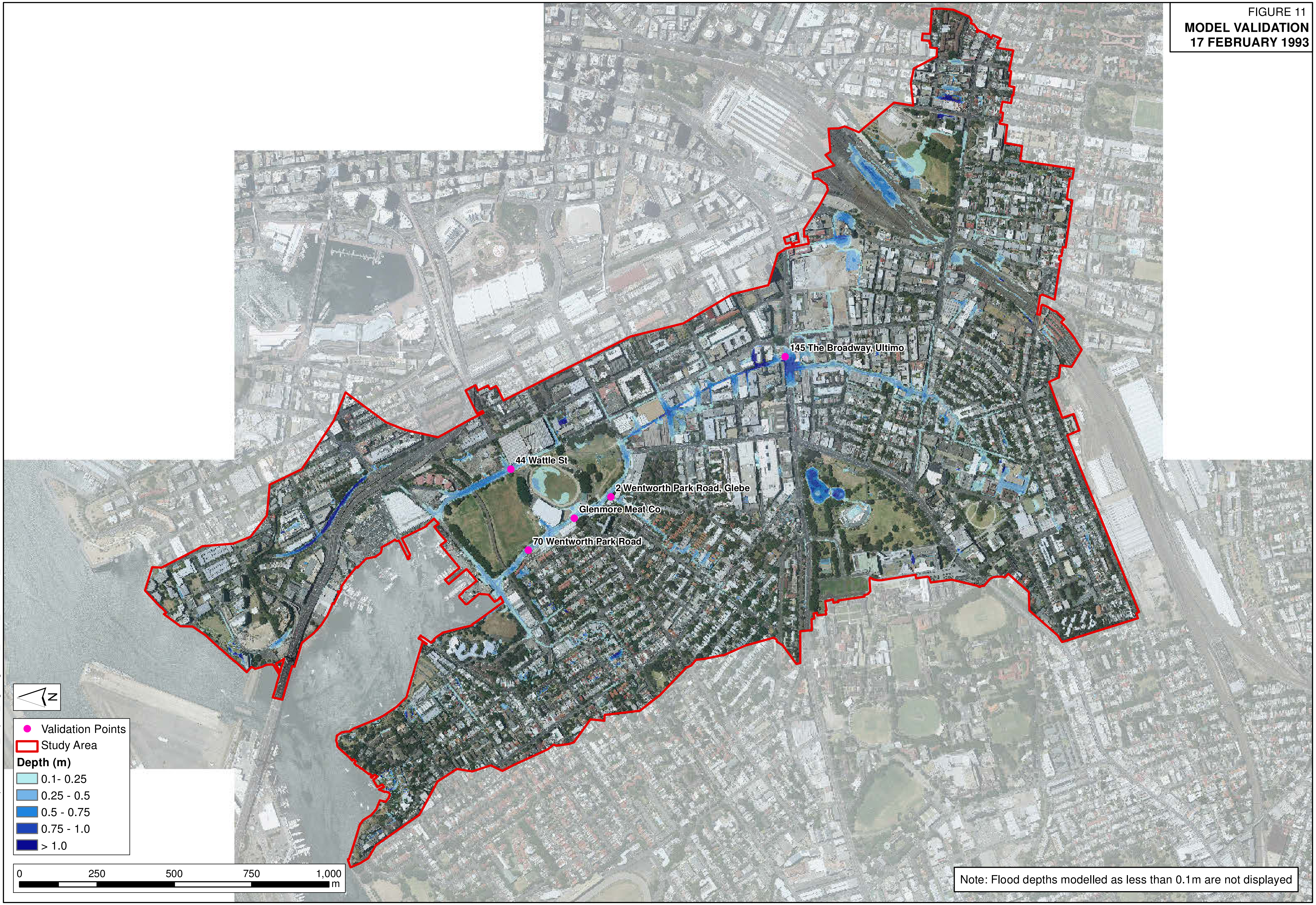
FIGURE 11
MODEL VALIDATION
17 FEBRUARY 1993



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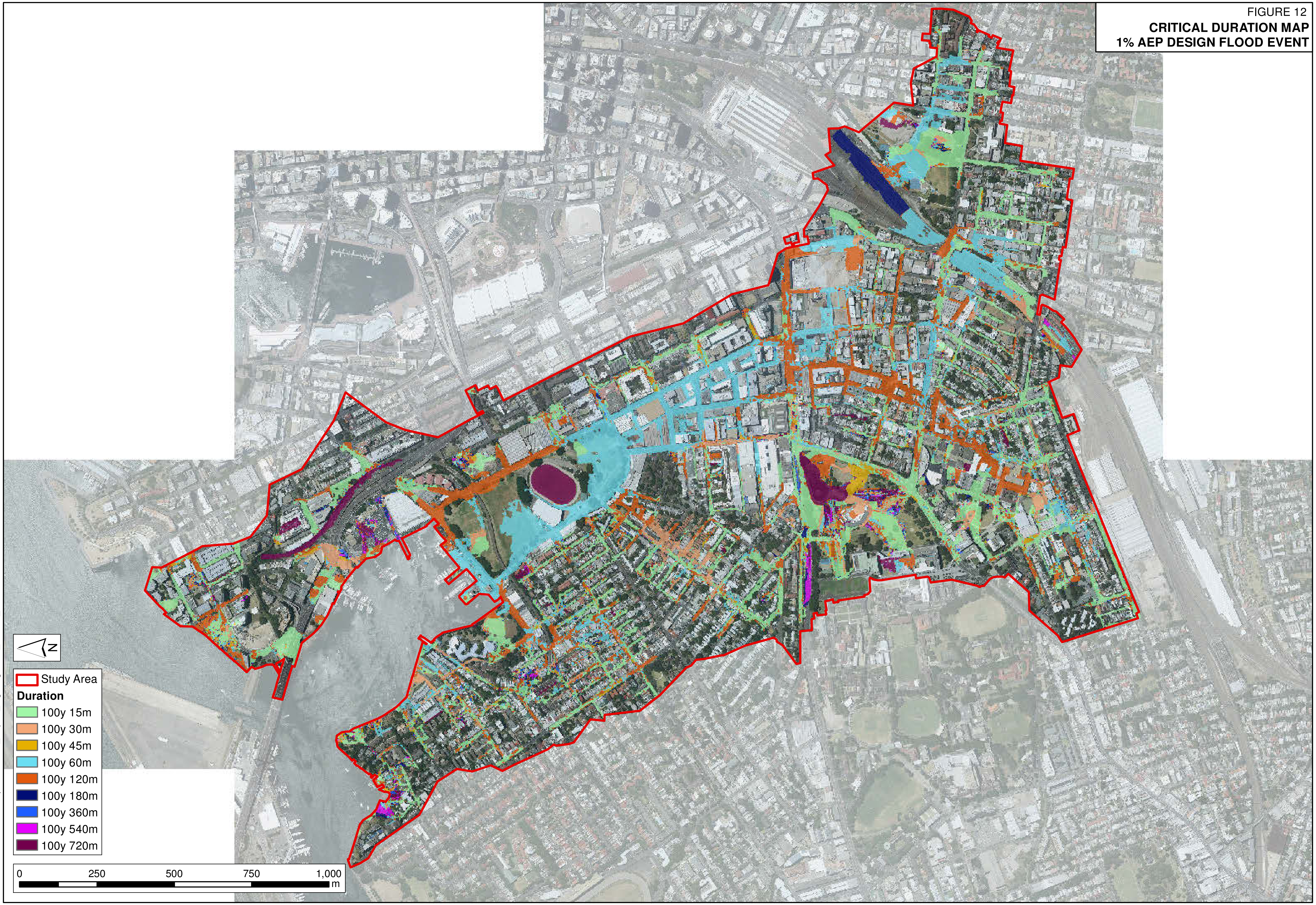

FIGURE 12
CRITICAL DURATION MAP
1% AEP DESIGN FLOOD EVENT

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FIGURE 13
TABULATED FLOODING LOCATIONS
BLACKWATTLE BAY CATCHMENT

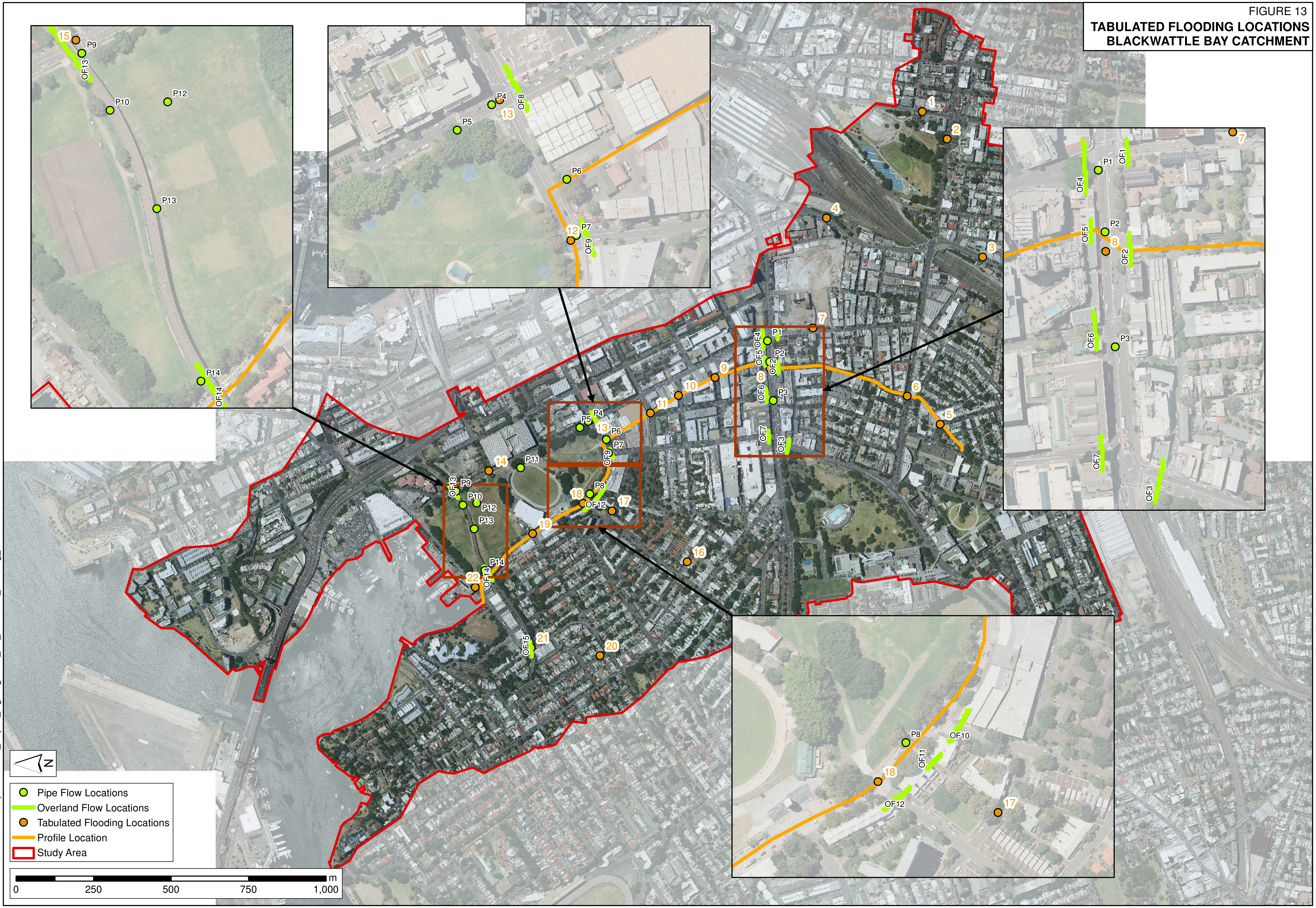

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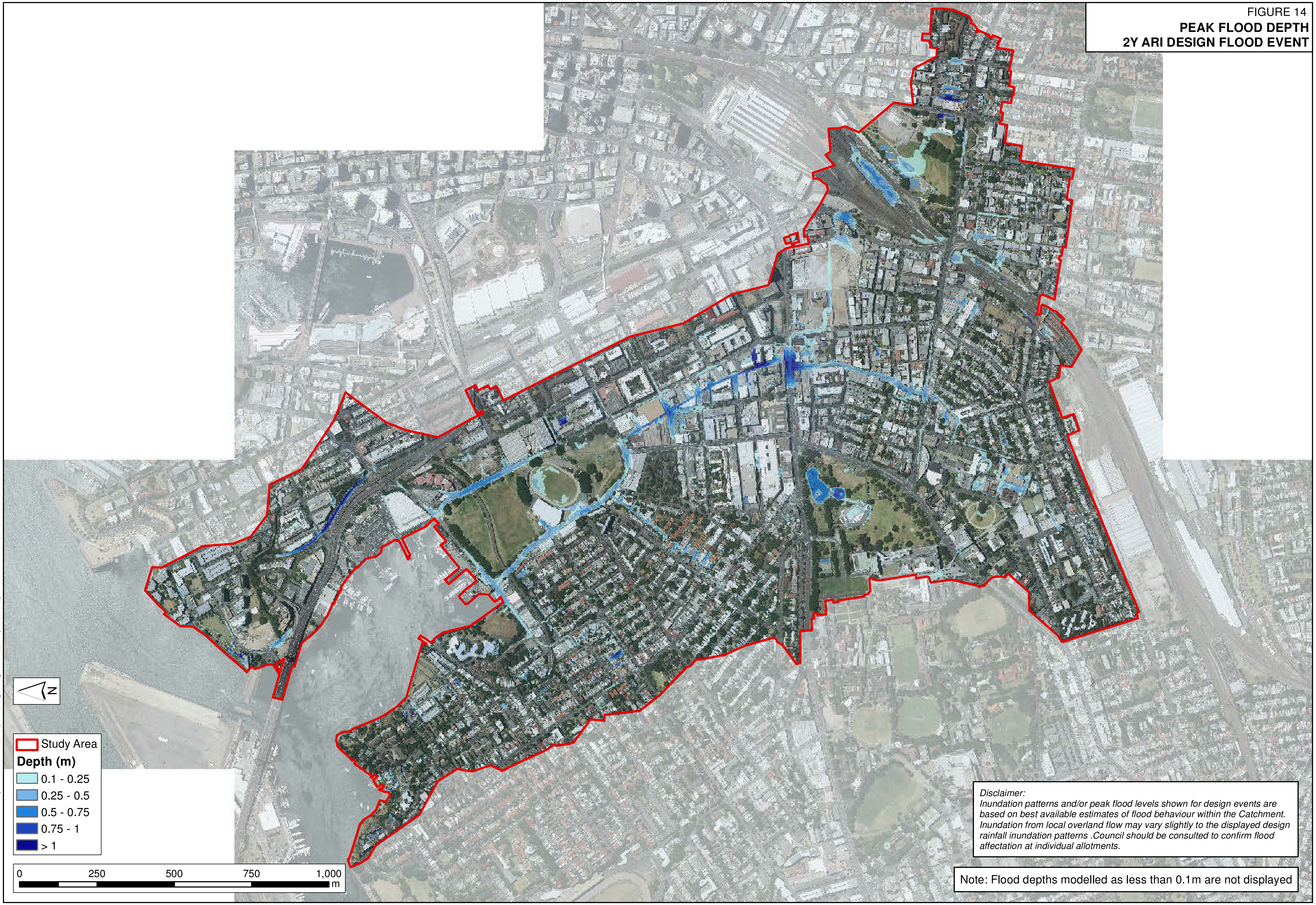
FIGURE 14
PEAK FLOOD DEPTH
2Y ARI DESIGN FLOOD EVENT



Disclaimer:
Inundation patterns and/or peak flood levels shown for design events are based on best available estimates of flood behaviour within the Catchment. Inundation from local overland flow may vary slightly to the displayed design rainfall inundation patterns .Council should be consulted to confirm flood affectation at individual allotments.

Note: Flood depths modelled as less than 0.1m are not displayed

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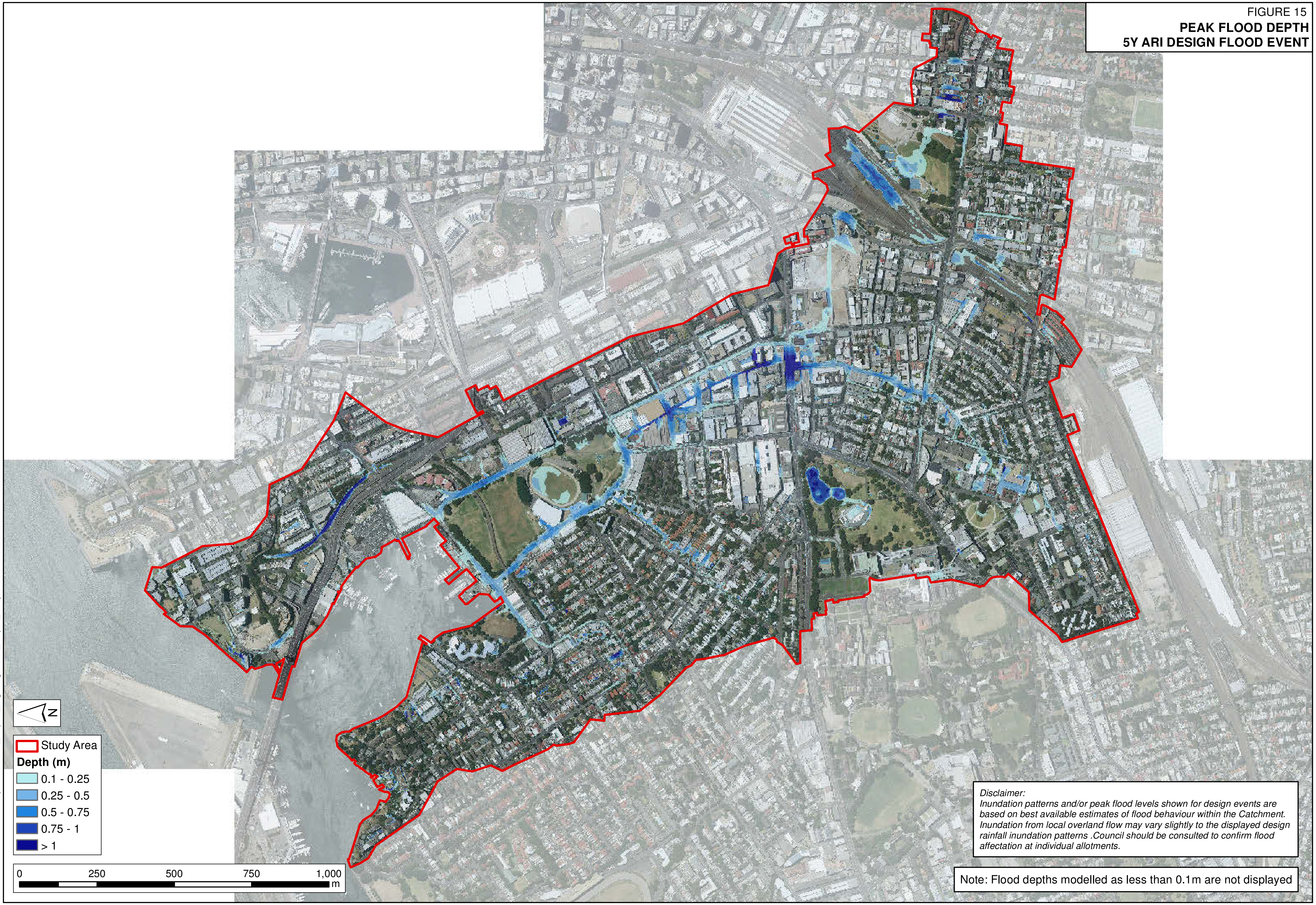
FIGURE 15
PEAK FLOOD DEPTH
5Y ARI DESIGN FLOOD EVENT



Disclaimer:
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Note: Flood depths modelled as less than 0.1m are not displayed

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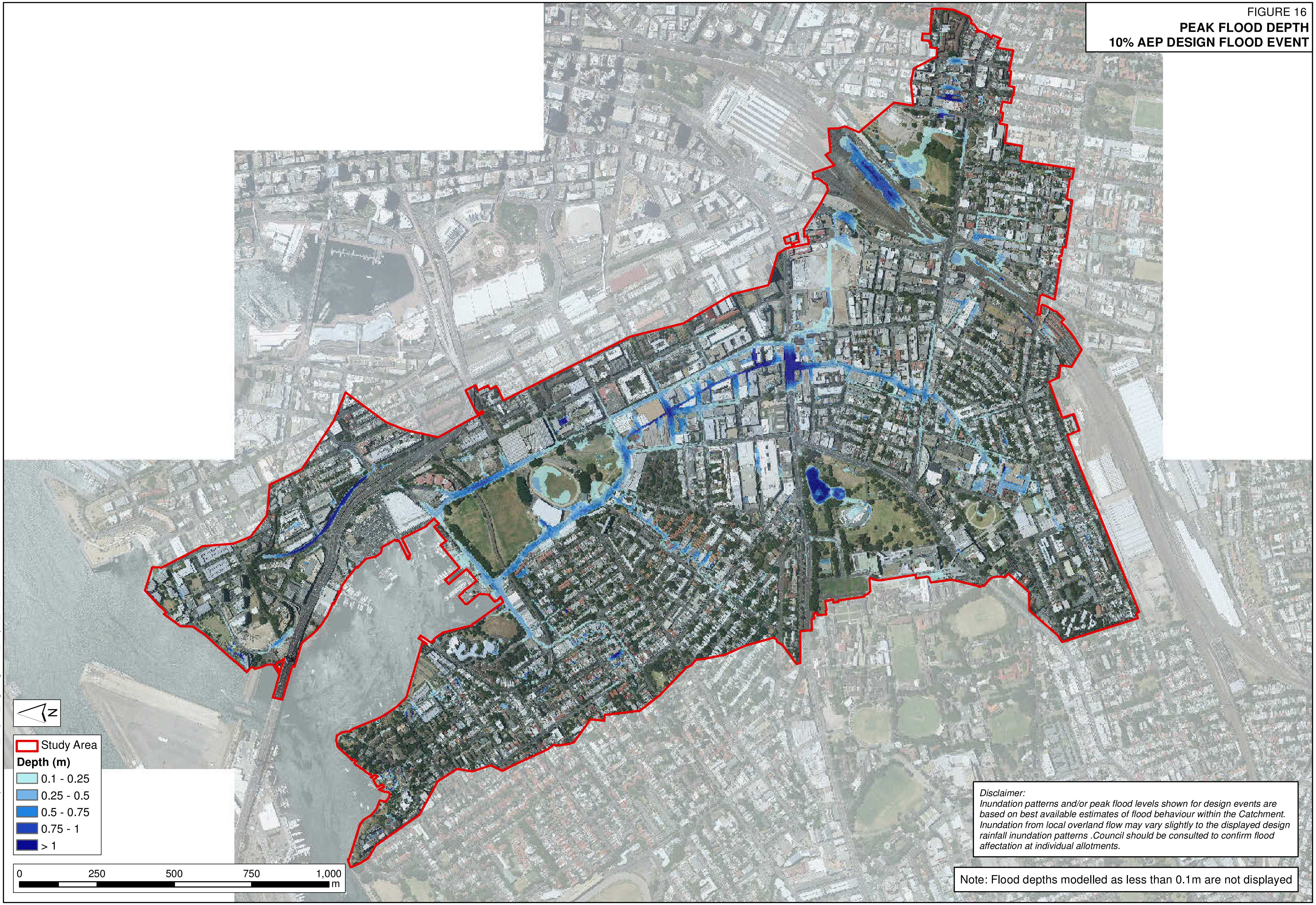
FIGURE 16
PEAK FLOOD DEPTH
10% AEP DESIGN FLOOD EVENT



Disclaimer:
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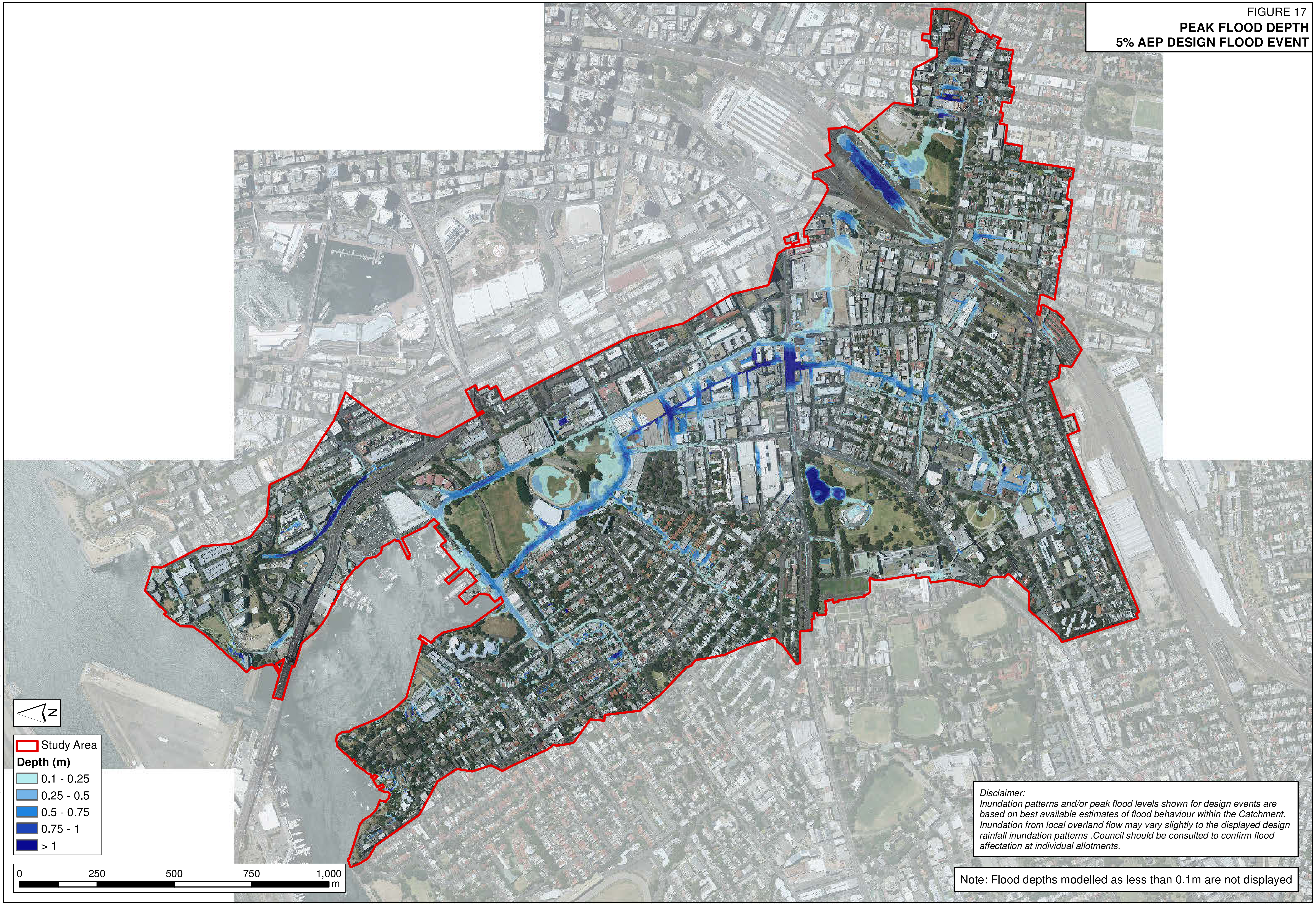
Note: Flood depths modelled as less than 0.1m are not displayed

FIGURE 17
PEAK FLOOD DEPTH
5% AEP DESIGN FLOOD EVENT



Disclaimer:
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Note: Flood depths modelled as less than 0.1m are not displayed

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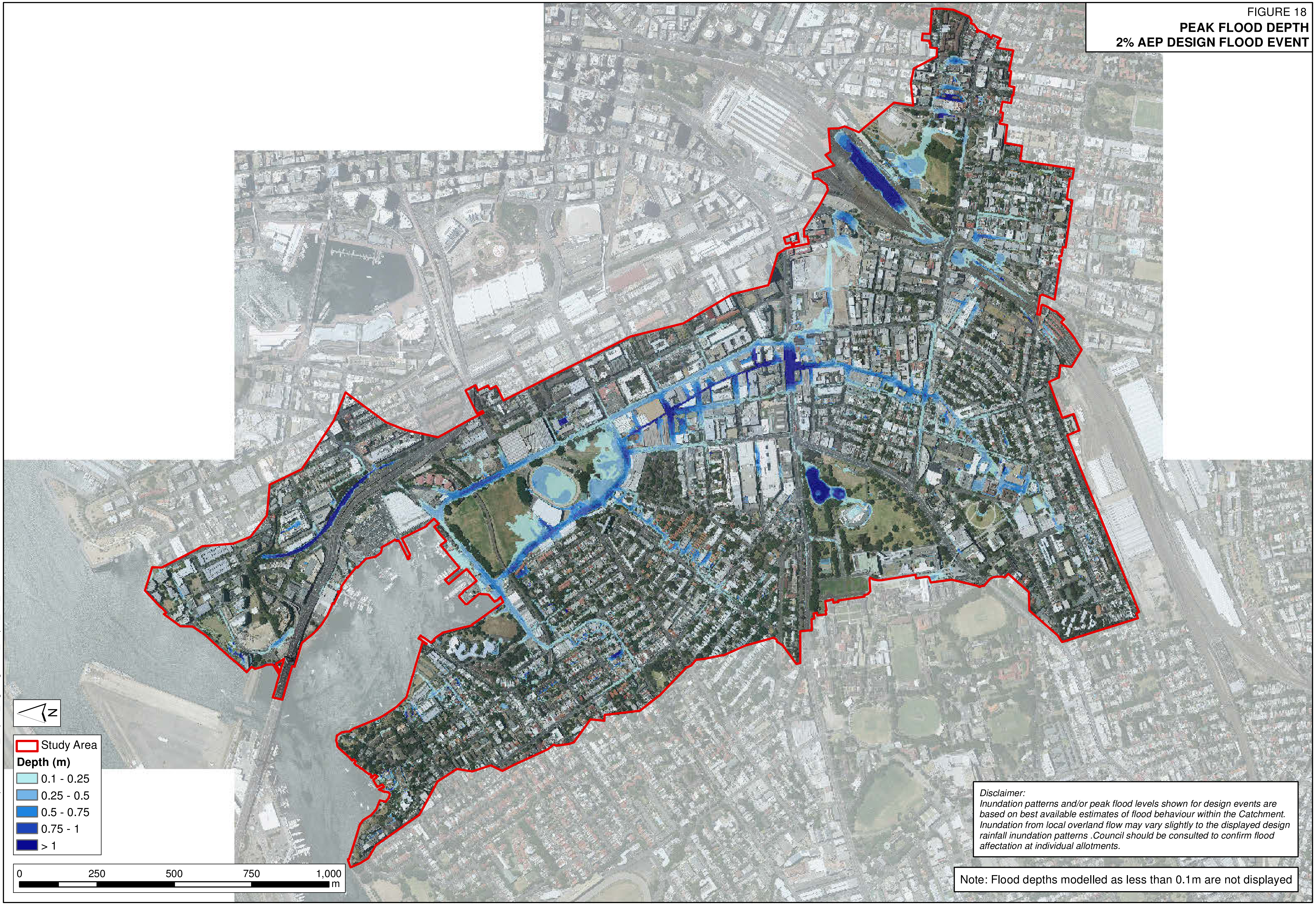
FIGURE 18
PEAK FLOOD DEPTH
2% AEP DESIGN FLOOD EVENT



Disclaimer:
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Note: Flood depths modelled as less than 0.1m are not displayed

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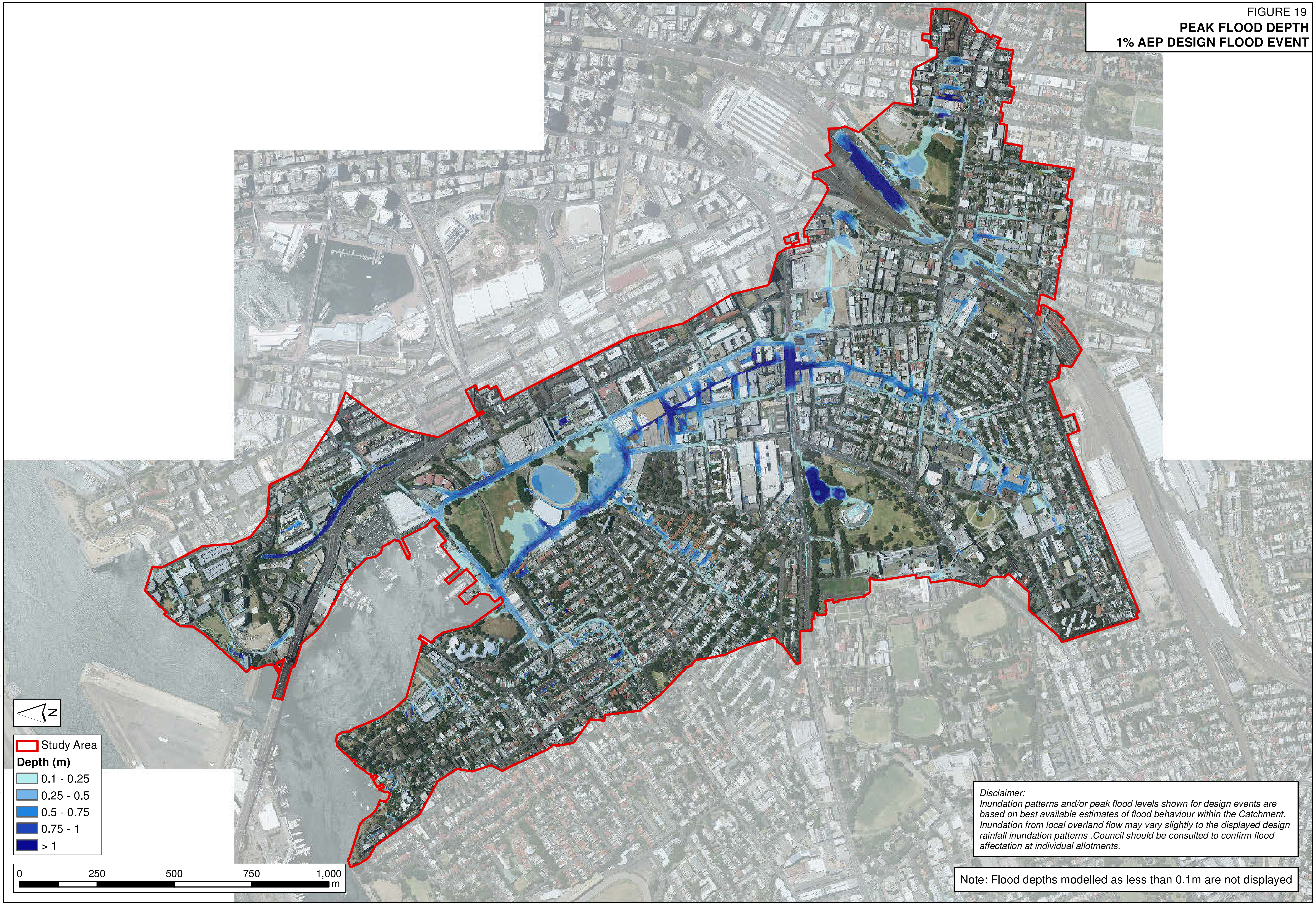
FIGURE 19
PEAK FLOOD DEPTH
1% AEP DESIGN FLOOD EVENT
N
Study Area
Depth (m)
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
> 1
0
250
500
750
1,000
m
Disclaimer:
Inundation patterns and/or peak flood levels shown for design events are based on best available estimates of flood behaviour within the Catchment. Inundation from local overland flow may vary slightly to the displayed design rainfall inundation patterns .Council should be consulted to confirm flood affectation at individual allotments.
Note: Flood depths modelled as less than 0.1m are not displayed
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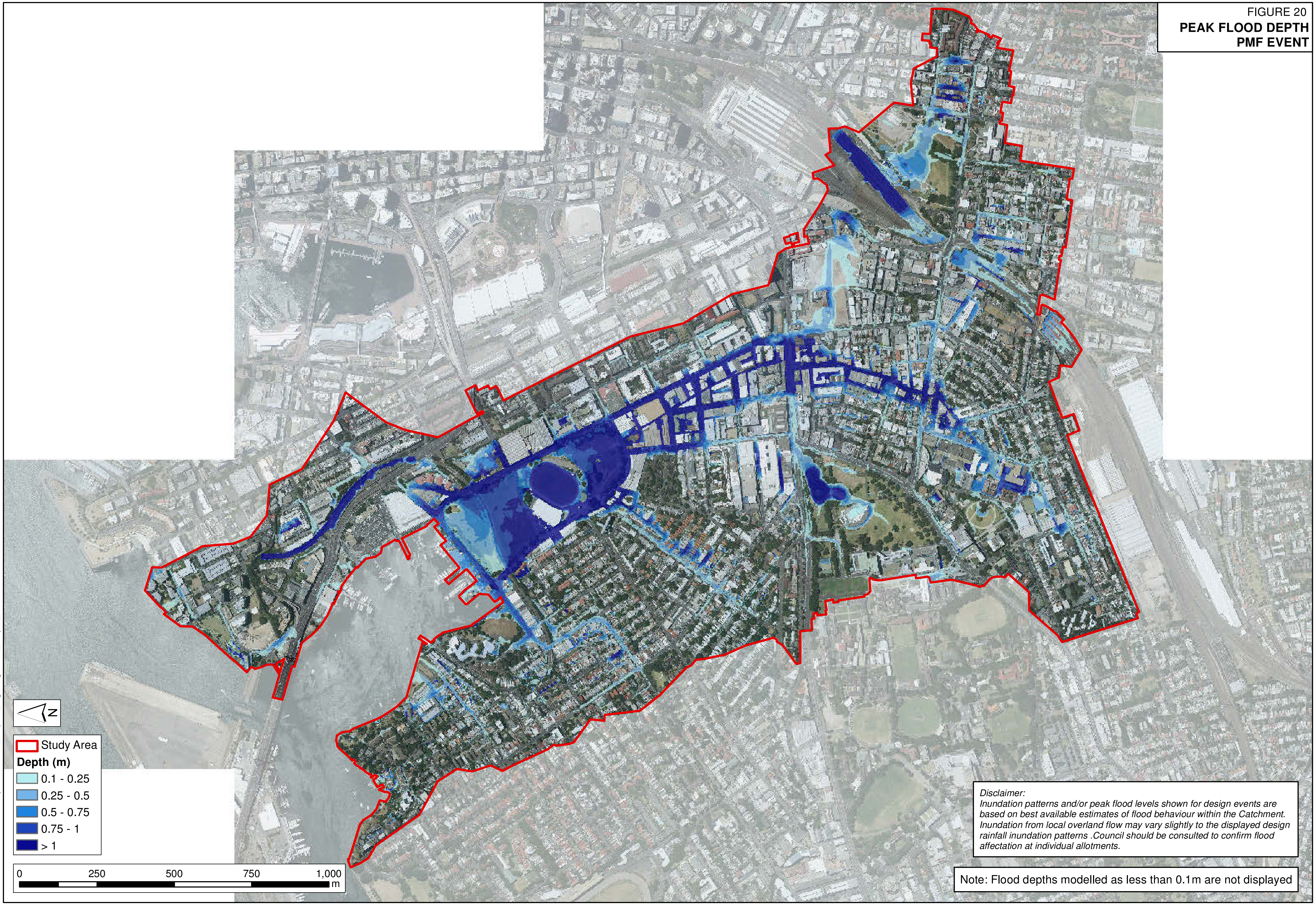
FIGURE 20
PEAK FLOOD DEPTH PMF EVENT



Disclaimer:
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Note: Flood depths modelled as less than 0.1m are not displayed

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