Ashfield Council

MARRICKVILLE council

HAWTHORNE CANAL

FLOOD STUDY

FINAL REPORT (MARRICKVILLE COUNCIL)

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FEBRUARY 2015

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FEBRUARY 2015

Project Hawthorne Canal Flood Study	**Project Number** 111054
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Date 1 April 2015	**Verified by**

Revision	Description	Date
6	Final Report (Marrickville Council)	February 2015
5	Final Draft Report – For Public Exhibition (Marrickville)	26 November 2014
4	Final Draft Report – For Public Exhibition (Ashfield)	15 October 2013
3	Draft Report	6 August 2013
2	Preliminary Draft Report	10 June 2013
1	Progress Report	22 April 2013

Ashfield City Council and Marrickville City Council have prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. This document does not necessarily represent the opinions of the NSW Government or the Office of Environment and Heritage.

HAWTHORNE CANAL FLOOD STUDY

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LIST OF ABBREVIATIONS

1D	One (1) Dimensional
2D	Two (2) Dimensional
ALS	Airborne Laser Scanning
ARI	Annual Rainfall Intensity
DEM	Digital Elevation Model
FPA	Flood Planning Area
FPL	Flood Planning Level
IFD	Intensity-Frequency-Duration
LiDAR	Airborne Light Detection and Ranging Survey
PMF	Probable Maximum Flood
SWC	Sydney Water Corporation
TIN	Triangular Irregular Network

FOREWORD

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

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government provides funding for flood studies, floodplain risk management plans and works to alleviate existing flood problems, to undertake the necessary technical studies to identify and address the problem and provides specialist technical advice to assist councils in the discharge of their floodplain management responsibilities. The Federal Government may also provide funding in some circumstances.

In order to implement the Policy within its local government area (LGA), Ashfield Council and Marrickville Council have embarked on a program of studies and actions as set out in the NSW Floodplain Development Manual with the assistance of Sydney Water Corporation and the Office of Environment and Heritage.

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 for the full range of flood events up to the Probable Maximum Flood (PMF).
2. ***Floodplain Risk Management***
   - Evaluates management options for the floodplain in respect of both existing and proposed development taking into consideration social, ecological and environmental factors related to flood risk.
3. ***Floodplain Risk Management Plan***
   - Involves formal adoption by Council of a plan of management for the floodplain after consultation with the public.
4. ***Implementation of the Plan***
   - Involves construction of flood mitigation works to protect existing development, implementation of community awareness programs to heighten flood awareness, improved evacuation arrangements to minimise flood damages and the risk to life, and the introduction of development control policies at various levels within the planning framework to ensure new development is constructed in a manner compatible with the flood hazard.

The Hawthorne Canal Flood Study constitutes the first stage of the management process for the Hawthorne Canal catchment within the Ashfield and Marrickville LGAs.

EXECUTIVE SUMMARY

BACKGROUND

The Hawthorne Canal is located approximately eight kilometres west south-west of Sydney's CBD and its catchment area includes the suburbs of Ashfield, Dulwich Hill, Haberfield, Leichhardt, Lewisham, Petersham and Summer Hill (see Figure 1 and Figure 2). The catchment is drained by a series of pits (inlets), pipes and overland flow-paths into Iron Cove on the Parramatta River. Sydney Water Corporation (SWC) owns the larger "trunk" drainage assets and the smaller pit and pipe networks are owned by Ashfield, Marrickville and Leichhardt councils.

The Hawthorne Canal Stormwater Channel extends from Canterbury Road at Lewisham to Dobroyd Point at Iron Cove. Assets owned by SWC terminate at Marion Street, Leichhardt, about 1.4 km upstream of the Canal's outlet into Iron Cove on the Parramatta River. Approximately 46% of the catchment is within Marrickville Council, 39% is within Ashfield Council and the remaining 15% is within Leichhardt Council.

OBJECTIVES

The purpose of this Flood Study is to identify local overland flow as well as mainstream flow and define existing flood liability. This objective is achieved through the development of a suitable model that can also be used as the basis for a future Floodplain Risk Management Study and Plan for the study area, and to assist Ashfield Council and Marrickville Council when undertaking flood-related planning decisions for existing and future developments. Previous hydraulic modelling of the study area was limited in extent, did not systematically incorporate overland flow and did not provide flood level estimates for the catchment.

The primary objectives of the study are to:

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

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

- a summary of available flood related data and a summary of previous events;
- details on the build and verification of the hydrologic and hydraulic models;
- sensitivity analysis of the model results to variation of input parameters;
- potential implications of climate change predictions with regard to sea level rise and rainfall intensity increase; and

- the definition of design flood behaviour for existing catchment conditions.

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

FLOODING HISTORY

In examining the flooding history it must be noted that the drainage characteristics of this catchment have been significantly altered as a result of urbanisation in the area and as such older flood extents and depths for a given storm may not apply to present day conditions. There have been many instances of flooding in the past with 5th August 1986, 10th April 1998 and 7th March 2012 being the most significant recent storm events recorded as causing extensive flooding throughout the catchment. However flood issues, in Queen Street and Victoria Street for example, are reported by the local community to occur on an annual to bi-annual basis as detailed in Section 2.6.2.

HYDROLOGIC AND HYDRAULIC MODELLING PROCESS

The hydrologic modelling was undertaken using DRAINS and the hydraulic model was established using TUFLOW.

Due to the limited available data for calibration and significant changes to the catchment in recent history, the calibration and verification of the models to historic data was tentative. Sensitivity analyses were undertaken to assess the influences of modelling assumptions on key outputs, and the potential impacts of future climate change. In the context of the Hawthorne Canal catchment, sea level rise is not likely to affect structures within the Marrickville LGA and impacts are restricted to the downstream areas of Ashfield Council.

The design rainfall events that were modelled were the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI design events and the Probable Maximum Precipitation (PMP). The temporal patterns for the design events were sourced from Australian Rainfall and Runoff (AR&R) (Pilgrim, 1987) and the Intensity-Frequency-Duration (IFD) data was obtained from the Bureau of Meteorology's (BoM) internet-based tool. The PMP estimates were derived according to the BoM guidelines, the *Generalised Short Duration Method* (BoM, 2003).

OUTCOMES

The design flood modelling indicates that significant flood depths may occur in a number of locations included in the Haberfield, Petersham, Lewisham, Ashfield / Dulwich Hill and Summer Hill suburbs. A detailed examination of existing flood behaviour at these "hot spots" has been undertaken. The study shows that while the railway line exacerbate the flooding problem, rail transport itself is unlikely to be severely disrupted during flood events. Major routes such as Parramatta Road and Old Canterbury Road are both reported and shown to experience significant flooding during most ARI design events, likely leading to severe traffic disruption.

A preliminary investigation into properties subject to flood related development controls shows that approximately a 1,000 lots (out of total of ~7,000 so around 14%) are liable to be tagged under the criteria adopted for the study.

1. INTRODUCTION

1.1. Background

The study was initially commissioned by the Sydney Water Corporation (SWC) with the intent of modelling trunk elements owned by the SWC only. SWC subsequently invited Ashfield and Marrickville councils to join the study. Both councils took this opportunity and the scope of work has been expanded to include modelling of Council drainage assets, i.e. pits and pipes, as well as local overland flows. The provision of detailed pits and pipes network by Council yields a significantly more realistic representation of the real-life hydrology and hydraulics happening in the Hawthorne Canal catchment area. The resulting model was calibrated via depths recorded by the SES, validated against observations obtained through community consultation and verified by comparison to similar models completed for similar areas within the Sydney Metropolitan Area as well as previously predicted pipe and channel flows for the trunk drainage assets within the Hawthorne Canal.

1.2. General

Drainage elements in the catchment include kerbs and gutters, pits and pipes and a network of trunk drainage elements including culverts and channels. Ownership of the assets is split between SWC and Council, with SWC owning the trunk elements. Amongst the drainage assets is a length of brickwork drain that was one of the first nine purpose-built stormwater drains to be constructed in Sydney in the 1890's. Open sections of the channel commence from Iron Cove to upstream of Davis Street. There are several branches to this system which extend through Leichhardt, Petersham, Smith Street, Henson Street, Victoria Street and Grove Street. The largest branch is the Leichhardt branch which extends from the main channel immediately on the downstream side of Marion Street (see Figure 13 for a summary of the main drainage assets included in the hydraulic model).

1.3. Description of the Study Area

The study area catchment is fully urbanised with approximately 70% of the catchment zoned for residential developments, 15% for special purpose, 6% for industrial, 5% for open space areas (parks and recreation areas), and the remaining 4% for business/commercial areas. Of the residential zoned areas, a significant percentage is characterised by medium density residential.

Elevations in the upper part of the catchment reach approximately 55 m AHD with grades of between 2% and 4%. Overall catchment slope averages 0.5% along the main flow-path towards Iron Cove. The main channel is tidal to upstream of Parramatta Road with the width expanding from approximately 2 m in upper areas to approximately 22 m at its confluence with Iron Cove, shown in Photo 1.

Photo 1: Hawthorne Canal – Iron Cove Confluence

The Western Railway Line associated with the City Rail network bisects the catchment in an east-west direction. Unconnected to this railway line is the discontinued freight railway line along the north-south axis. The former freight railway line is part of the light rail extension track, for which construction work for the platforms commenced in November 2012.

The West Street Catchment (East of West Street) covers an area of approximately 55 ha and flow paths converge on Petersham Park before discharging to Parramatta Road likely leading to severe traffic disruption across the highway. The fully developed nature of the catchment with medium density residential housing exacerbates peak flow levels as a result of the shorter routing time due to the catchment high percentage imperviousness.

1.4. Objectives

The primary objective of this Flood Study is to develop computational hydrologic and hydraulic models that define design flood behaviour for the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI design events and the Probable Maximum Flood (PMF) in the Hawthorne Canal catchment and to:

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

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

1.5. Multiple Stakeholders

This Flood Study is a collaborative project with multiple stakeholders, namely Sydney Water Corporation (SWC), Ashfield Council and Marrickville Council. These three stakeholders were provided with this report and attached appendices, which are inclusive of the other stakeholders' areas of interest. However, the information provided to stakeholders specific to their area of interest, such as electronic spreadsheets of properties flood planning levels, were filtered to their relevant areas.

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2. AVAILABLE DATA

2.1. Overview

The first stage in the investigation of flooding matters is to establish the nature, size and frequency of the problem. On large river systems such as the Hawkesbury River there are generally stream height and historical records dating back to the early 1900's, or in some cases even further. However, in small urban catchments such as that of the Hawthorne Canal, there are no stream gauges or official historical records available. A picture of flooding must therefore be obtained from an examination of previous reports, rainfall records and local knowledge.

2.2. Data Sources

Data utilised in the study has been sourced from a variety of organisations. The table below lists the type of data sourced and from where it has been extracted.

Type of Data	Format Provided (Source)	Format Stored
Location, description and invert depths of pits, pipes and trunk drainage network	GIS (Sydney Water)	DRAINS and TUFLOW models
Ground levels from ALS data	GIS (Sydney Water)	GIS and TUFLOW model
Detailed Survey Data	GIS (Sydney Water)	GIS and TUFLOW model
GIS information (cadastre, drainage pipe layout)	GIS (Sydney Water)	GIS
Design rainfall	AR&R	DRAINS
Recorded flood data	Observation by Sydney Water and Local Community	Report (Hawthorne Canal Flood Study)
Hydrology (rainfall data)	ASCII Text (Bureau of Meteorology, Sydney Water)	DRAINS

2.3. Topographic Data

Airborne Light Detection and Ranging (LiDAR) survey of the catchment and its immediate surroundings was provided for the study by SWC. It was indicated that the data were collected in 2007 by AAMHatch. These data typically have accuracy in the order of:

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

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

From this data, a Triangular Irregular Network (TIN) was generated by WMAwater. This TIN was sampled at a regular spacing of 1 m by 1 m to create a Digital Elevation Model (DEM), which formed the basis of the two-dimensional hydraulic modelling for the study (shown in Figure 3).

2.4. Cross-section Data

Within the Hawthorne Canal catchment the main drainage network includes regular open channel sections. For these areas, the definition to the top of the concrete-lined channel was based on cross-sections provide by the SWC capacity assessment document (SWC, 1998), shown in Photo 2.

Photo 2: Typical cross-section within the Hawthorne Canal

Structures traversing the waterway such as bridges and culverts may have a significant influence on flood behaviour. Often such structures constrict and obstruct flow and their impact can vary with flood magnitude. Geometric details of these structures are required for the hydraulic model. These structures are typically not accurately captured by remote sensing technologies such as ALS and for this reason a traditional ground survey was commissioned and undertaken by Chase Burke & Harvey (CBH) Surveyors. From this, definition of the cross-sectional area was obtained, particularly where the underside of the bridge was not the same height as the top of the concrete-lined channel, as shown in Photo 3.

Photo 3: Marion Street bridge traversing open channel (provided by CBH Surveyors)

2.5. Pit and Pipe Data

The SWC capacity assessment document (SWC, 1998) provided dimensions for SWC owned underground pipes, in addition to the open channel cross-sections discussed above. Appended to this SWC drainage network are underground pipes owned by the various Council jurisdictions within the Hawthorne Canal catchment.

Ashfield City Council and Burwood City Council provided pit location and pipe dimensions for the infrastructure within the respective council area, where feasible. However, some pipe dimensions within the Ashfield LGA were not available due to the inaccessibility of the location, notably those pipes located along the busy thorough-fare of Parramatta Road. Lack of this data will only impact results to a very small degree and impacts will be less significant for larger events such as the 100 year ARI.

The pit and pipe details used have not been verified as part of the study, although details provided by the respective parties have been merged together and shown to demonstrate basic agreement.

2.6. Historical Flood Level Data

2.6.1. SWC Historic Flood Level Data

An historic flood database, supplied by SWC, provided information of flooding within the catchment from 1958 to 1993 (SWC, 2011). A summary of the SWC historical flood levels is provided in Figure 7and Table 1.

Table 1: Summary of SWC Historic Flood Levels

Flood Events	Approximate ARI	Total Records	Number of Observed Flood Levels
February 1958	-	1	1
October 1959	-	4	7
November 1989	-	1	0
February 1993	42	4	4

The SWC database advised that all but one of the records for Hawthorne Canal provided in this database were sourced from the file number 225419F1. This corresponds with the source file number referenced in the Leichhardt Flood Study (discussed in Section 0). The February 1958 event was sourced from file number 224152F9.

The information from the 1958 and 1959 events predates the establishment of nearby pluviometric gauges (discussed in Section 2.7.4). The 1989 event information only states that a property was flooded and SES assistance required; however, there was very little rainfall (5.5 mm) recorded during the days prior to the event making it unsuitable for calibration. The 1993 information matches a rainfall event in the pluviometer record, but the observed flood levels are all located near the Leichhardt branch, whereas ideally levels would be spread throughout the catchment.

2.6.2. Community Consultation

In collaboration with Ashfield and Marrickville councils, a questionnaire and newsletter were distributed to residents and owners of property within the two LGA's. The survey described the role of the Flood Study in the floodplain risk management process, and requested than any records of historical flooding (as well as any evidence of non-affectation) be submitted. Around 4,700 surveys were distributed with reply paid envelopes, and 315 responses were received (response rate of 7%).

Different distribution methods were implemented by the respective councils during the community consultation phase of the project. Ashfield Council distributed questionnaires to every household within the LGA whereas Marrickville Council used a more targeted approach and only distributed questionnaires to residents deemed likely to be affected by flooding. Both methodologies have their merit as the total number of responses from Ashfield was higher whereas information received from Marrickville respondents was more relevant and concise.

The information requested in the survey included details about length of residency in the catchment, descriptions of any experiences of flooding, and evidence of flood heights or extents such as photographs of flood marks (Appendix B). The number of respondents that recalled being affected by flooding are summarised in Table 2.

Table 2: Summary of Reported Incidences of Flooding

	Number of Responses	
	Ashfield	**Marrickville**
Property affected by flooding in the past	35	12
Houses flooded above floor level	4	1
Provided indicative flood depth or photo following flood from which depth could be estimated	15	12

The Community Consultation process was successful in obtaining responses from throughout the Hawthorne Canal catchment area as illustrated in Figure 5 and a summary of responses is presented on Figure 6. The consultation process enabled the identification of a number of flooding 'Hot spots' within the catchment.

Haberfield

Residents around the Batallion Circuit parkland have reported regular flooding. Flood water rushes in from Marion Street and accumulates at Hawthorne Parade roundabout causing severe traffic disruption as well as flooding to adjoining properties. During heavy rainfalls that coincide with a high tide period, the stormwater pipe between 43 - 45 O'Connor Street overflows resulting in private property flooding to adjacent houses, shown in Photo 4. Photo 5 shows overland flooding near Dalhousie Street in Haberfield.

Photo 4: Flood marks on O'Connor Street

Photo 5: Street flooding near Dalhousie Street

Petersham

A number of general flood complaints in the area. Specifically, two significant floods reported in the last five years on Station Street.

Lewisham

Reports of extensive flooding at Lewisham train station with estimated flood depths of 0.3 - 0.5

metres on Victoria Street as well as flooding on Hobbs Street due to overflowing gutters during periods of heavy rainfall.

Ashfield / Dulwich Hill

Several cases of private property flooding along Queens Street as illustrated in Photo 6 - Photo 10 taken during the 2009 flood and the 2012 flood (subsequently used in the validation of the model). Overcharged culverts on Queen Street as well as on Armstrong Street claimed to flood Service Avenue. Resident on Service Avenue had floor level raised and other mitigation measure put in place in 1990 following floods in August 1986 when the highest daily rainfall on record was measured at the Ashfield gauging site. Further downstream, Elizabeth Avenue tends to flood during heavy rainfalls.

Photo 6: Queen Street

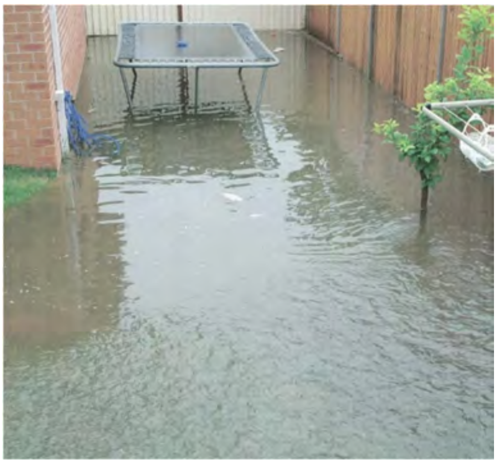


Photo 7: Queen Street

Photo 8: Queen Street

Photo 9: Queen Street

Photo 10: Queen Street

Summer Hill

Buildings with low floor levels such as private garages tend to flood on Prospect Road and Tintern Road. The stormwater drain on Carrington Road is seen to overflow during periods of heavy rainfall resulting in flooding along the street.

The flood experiences described in the survey responses generally related to smaller and more frequent flooding which mostly cause ponding of stormwater in roadways or gardens, although five instances of above floor flooding in residential properties were also reported.

A copy of the questionnaires sent out by respective councils, as well as a summary of responses received, is provided in Appendix B.

2.6.3. Overview of Calibration Data

The historical flood information gathered during the community consultation process undertaken by Ashfield Council and Marrickville Council yielded rough flood depths for events corresponding to a 1-5 year flood. Preliminary model validation included a comparison of the zones that the community indicated got flooded to flooding extents outputted by the model for a 1, 2 and 5 year storm. Figure 7 shows the three categories of points that were taken into consideration. The most useful data points were actual flood depths measured by local residents during period of floods or from floodmarks left once the flood had receded. Photographs and accounts of flood level using arbitrary units of measurement *"the water was knee-deep"* were also submitted and an estimate of the corresponding flood depth was carried out and used to assist in the verification of the model. The final set were mentions of flooded areas by local residents without any specific depth but these were still useful in verifying whether the flood extents obtained from the model agreed with observation in the field.

Due to the recentness of the March 2012 event, a number of local residents provided estimates of localised flooding. In all, 9 estimates situated throughout the Hawthorne Canal catchment were used for comparison against model results obtained from the validation runs for the above event. The advantage of a recent flood is that many structural changes occur within an urban catchment which renders historical flood marks less useful than recent flood marks.

Table 3: Calibration Flood Information

Location	Description	Flood Event	Level *(mAHD)*	Source
Corner of Darley Rd and Elswich St North		17/02/1993	2.56	SES
Upward St	Flood level at end entrance ramp	17/02/1993	7.93	SES
Corner of George St and McAleer St	Flood level above loading dock level	17/02/1993	10.52	SES
Parramatta Rd	Above floor level flooding	17/02/1993	11.74	SES
Station St	Flood level to the top of veranda	17/02/1993	12.18	SES
Hawthorne Pde, Haberfield	Street and front yard flooding	7/03/2012	~3.0	CC
Eltham St, Dulwich Hill	Street Flooding	7/03/2012	~13.4	CC
Hobbs St, Lewisham	Street and property flooding	7/03/2012	~18.2	CC
Corner of Railway Terrace	Street Flooding	7/03/2012	~28.1	CC
Abergeldie St, Dulwich Hill (South)	Property flooding	7/03/2012	~28.1	CC
Abergeldie St, Dulwich Hill (North)	Driveway flooded	7/03/2012	~32.9	CC
Prospect Rd, Summer Hill	Flood level 500mm in cellar	7/03/2012	~33.2	CC
Queen Street, Ashfield	Property flooding	7/03/2012	~38.7	CC
CC are depths obtained during the Community Consultation and as such are approximate				

2.7. Historic Rainfall Data

2.7.1. Overview

Rainfall data is recorded either daily (24 h rainfall totals measured at 9:00 am) or continuously (pluviometers measuring rainfall in small increments). Daily rainfall data has been recorded for over 100 years at many locations within the Sydney basin. In general, pluviometers have only been installed since the 1970's and are not available in nearly as many locations. Together these records provide a picture of when and how often large rainfall events have occurred in the past.

Care must be taken when interpreting historical rainfall measurements. Rainfall records may not provide an accurate representation of past events due to a combination of factors including local site conditions, human error or limitations inherent to the type of recording instrument used. Examples of limitations that may impact the quality of data used for the present study are highlighted in the following:

- Rainfall gauges frequently fail to accurately record the total amount of rainfall. This can occur for a range of reasons including operator error, instrument failure, overtopping and vandalism. In particular, many gauges fail during periods of heavy rainfall and records of large events are often lost or misrepresented.
- Daily read information is usually obtained at 9:00am in the morning. Thus if a single storm is experienced both before and after 9:00am, then the rainfall is "split" between two days of record and a large single day total cannot be identified.
- In the past, rainfall over weekends was often erroneously accumulated and recorded as

a combined Monday 9:00 am reading.

- The duration of intense rainfall required to produce overland flooding in the study area is typically less than 6 hours (though this rainfall may be contained within a longer period of rainfall). This is termed the "critical storm duration". For a larger catchment (such as the Parramatta River) the critical storm duration may be greater (say 9 hours). For the study area a short intense period of rainfall can produce flooding but if the rain stops quickly, the daily rainfall total may not necessarily reflect the magnitude of the intensity and subsequent flooding. Alternatively the rainfall may be relatively consistent throughout the day, producing a large total but only minor flooding.
- Rainfall records can frequently have "gaps" ranging from a few days to several weeks or even years.
- Pluviometer (continuous) records provide a much greater insight into the intensity (depth vs. time) of rainfall events and have the advantage that the data can generally be analysed electronically. This data has much fewer limitations than daily read data. Pluviometers can also fail during storm events due to the extreme weather conditions

Rainfall events which cause overland flooding (as opposed to mainstream flooding) in the Hawthorne Canal catchment are usually localised and as such are only accurately "registered" by a nearby gauge. Gauges sited even only a kilometre away can show very different intensities and total rainfall depths.

2.7.2. Rainfall Stations

Table 4 presents a summary of the official rainfall gauges (sourced from the Bureau of Meteorology) located close to or within the catchment. This includes daily read stations, continuous pluviometer stations, operational stations and synoptic stations. These gauges are operated either by Sydney Water Corporation (SWC) or the Bureau of Meteorology (BOM).

Table 4: Rainfall Stations within 6km of the centre of the Hawthorne Canal catchment

Station Number	Station Name	Operating Authority	Distance from centre of the catchment (km)	Elevation (m AHD)	Date Opened	Date Closed	Type
566112	Ashfield (Ashfield Park Bowling Club)	SWB	0.87	20	2/12/1993	1/02/2001	Continuous
66000	Ashfield Bowling Club	BOM	0.91	25	30/03/1896		Daily
66165	Ashfield Prospect Rd	BOM	1.60	43	01/01/1894	1/01/1904	Daily
566065	Lilyfield Bowling Club	SWB	2.19	20	21/12/1988		Continuous
66150	Canterbury Heights	BOM	2.93	61	30/08/1906	29/12/1916	Daily
566026	Marrickville Sps	SWB	3.07	5	1/05/1904		Continuous
566026	Marrickville Sps	SWB	3.07	5	1/05/1904		Daily
66017	Barnwell Park Golf Course	BOM	3.09	4	29/11/1929	28/11/2003	Daily
66036	Marrickville Golf Club	BOM	3.38	6	29/04/1904	29/12/1970	Daily
66036	Marrickville Golf Club	BOMNS	3.38	6	6/04/2001		Operational
66194	Canterbury Racecourse AWS	BOM	3.50	3	2/10/1995		Synop
66101	Fernbank	BOM	3.76		01/01/1889	1/01/1913	Daily
66034	Abbotsford (Blackwall Point Rd)	BOM	3.91	15	1/01/2004		Daily
66175	Schnapper Island	BOM	4.42	5	28/02/1932	29/12/1939	Daily
566078	South Cronulla	SWB	4.49	20	9/02/1990		Continuous
66113	Burwood 1	BOM	4.50		01/01/1884	1/01/1922	Daily
66026	Homebush	BOM	4.50		30/10/1924	29/12/1952	Daily
66149	Glebe Point Syd. Water Supply	BOM	4.51	15.2	30/05/1907	29/12/1914	Daily
66111	Croydon	BOM	4.58		30/01/1879	29/12/1921	Daily
66091	Burwood 2 Public School	BOM	4.79		29/09/1911	29/12/1923	Daily
66033	Alexandria (Henderson Rd)	BOM	5.15	15	29/04/1962	29/12/1963	Daily
66033	Alexandria (Henderson Rd)	BOM	5.15	15	30/03/1999	12/03/2002	Daily
66018	Earlwood Bowling Club	BOM	5.30	31.1	30/07/1914	29/12/1975	Daily
66178	Birchgrove School	BOM	5.48	10	29/04/1904	29/12/1910	Daily
66108	Hunters Hill St Josephs College	BOM	5.50		1/01/1916	1/01/1923	Daily
66071	Gladesville Champion Rd	BOM	5.62	10	27/02/1997	29/09/2000	Daily
66013	Concord Golf Club	BOM	5.68	15	1/01/1930		Daily
566020	Enfield (Composite Site)	SWB	5.68	10	14/04/1959		Continuous
566020	Enfield (Composite Site)	SWB	5.68	10	14/04/1959		Daily
66015	Crown St. Reservoir	BOM	5.70		30/01/1882	29/12/1960	Daily
66097	Ranwick Bunnerong Rd	BOM	5.82		1/01/1904	1/01/1924	Daily

2.7.3. Analysis of Daily Read Data

An analysis of the daily records for the nearest daily rainfall stations was undertaken to identify and place past storm events in some context. The Ashfield Bowling Club station is located close to the western catchment boundary and the Marrickville Golf Course station is located to the south. The Ashfield station was established in March 1896 and is still active. The Marrickville station was established in April 1904 and is still active, although there was a period between 1970 and 2001 in which this gauge was not operational.

Ashfield Bowling Club (66000)			
Mar 1896 – to date			
Rank	Date	Number of days accumulated	Rainfall (mm)
1	6/08/1986	1	245
2	9/03/1913	1	210
3	28/03/1942	1	206
4	3/02/1990	1	206
5	10/02/1956	1	194
6	17/06/1950	2	182
7	13/01/1911	1	175
8	27/11/1955	1	167
9	22/02/1954	2	160
10	26/03/1984	3	158
11	24/01/1955	2	157
12	11/03/1958	1	154
13	19/02/1959	1	152
14	10/01/1949	1	151
15	10/03/1958	2	150
16	11/06/1991	1	146
17	21/06/1975	1	144
18	16/06/1952	2	143
19	19/11/1961	1	143
20	30/04/1988	1	142

Marrickville Golf Club (66036)			
April 1904 – to date (ex. 1970-2001)			
Rank	Date	Number of days accumulated	Rainfall (mm)
1	9/03/1913	1	216
2	14/11/1969	1	144
3	13/01/1911	1	140
4	10/07/1904	1	127
5	5/02/2002	1	118
6	27/04/1966	1	116
7	5/05/1919	1	112
8	16/04/1969	1	108
9	22/07/2011	1	105
10	28/07/1908	1	104
11	2/04/1905	1	102
12	8/03/2012	1	101
13	29/01/2013	1	98
14	12/01/1918	1	98
15	9/11/1966	1	97
16	11/05/1925	1	97
17	2/07/1921	1	96
18	19/04/2012	1	94
19	26/02/1967	1	93
20	5/10/1916	1	92

The results indicate that the 1986, 1990, 2002, 2011 and 2012 were the largest daily rainfall events in recent times. The 1986 event is known to have caused flooding in the adjacent catchment of Dobroyd Canal (based upon SWC records), however it is not known whether Hawthorne Canal experienced similar flooding. The 8th March 2012 event corresponded with flooding reported by residents during the community consultation process, discussed in Section 2.6.2.

There is no evidence to suggest that the 1990, 2002 and 2011 storm events resulted in flooding within the catchment, based upon either SWC records or community consultation. However, this can be attributed to flooding within the catchment typically resulting from intense rainfall over sub-daily durations. High daily rainfall totals will not necessarily result in widespread flooding of the catchment, particularly if the rainfall is fairly evenly distributed throughout the day.

2.7.4. Analysis of Pluviometer Data

As noted previously, continuous pluviometer records provide a more detailed description of temporal variations in rainfall. As such, the Lilyfield Bowling Club (566065) and the Marrickville German Club (566026) pluviometer stations were analysed.

The highest daily totals for these pluviometers are provided in Table 6. The 1986 event at which time only the Marrickville gauge was operational was the highest daily event there. The largest events in recent times which feature in all three records occurred in February 1990 and April 1998.

The March 2012 event ranked at number 16 for the Marrickville gauge and number 14 for the Lilyfield gauge. Due to recentness of the event several indicative flood levels were reported for it by locals during the community consultation and therefore the event was used in the validation process (see Section 6.4).

While the 2012 event ranked highly on the daily rainfall charts, the rainfall intensities measured only corresponded to a 1 year ARI. Rainfall intensities for a 30 minutes duration recorded at Lilyfield on the 17th of September 1993 exceeded a 40 year ARI design intensity. For the same duration comparisons with design rainfall intensities indicate that the Marrickville gauge five kilometres away experienced intensities with a 14 year ARI. Pluviometric data for the Ashfield gauge was not available since records started in December 1993 however a full analysis of the Marrickville and Lilyfield pluviometer gauges was undertaken with a summary of the maximum rainfall and ARI for the 30, 60 and 120 minute durations results shown in Table 5.

Table 5: Rainfall Intensities for the February 1993 Event

Marrickville German Club (566026)

	17th of February 1993		
Duration	Max Rain	Intensity	ARI
minutes	*mm*	*mm/hr*	
30	47	94	14
60	63.5	64	13
120	81	41	11

Lilyfield Bowling Club (566065)

	17th of February 1993		
Duration	Max Rain	Intensity	ARI
minutes	*mm*	*mm/hr*	
30	58	116	42
60	71.5	72	22
120	87.5	44	16

Table 6: Highest Daily Total Rainfalls for each Representative Gauging Station

	Marrickville German Club (566026)			Lilyfield Bowling Club (566065)			Ashfield Bowling Club (566112)	
	Dec 1979 - To date			Jan 1989 - To date			Dec 1993 - To date	
Rank	Date	Rainfall	Rank	Date	Rainfall	Rank	Date	Rainfall
		mm			*mm*			*mm*
1	5/08/1986	240	1	2/02/1990	246	1	10/04/1998	153
2	10/06/1991	181	2	10/04/1998	185	2	18/05/1998	132
3	2/02/1990	176	3	8/02/1992	185	3	30/08/1996	119
4	29/04/1988	156	4	3/02/1990	145	4	7/08/1998	109
5	8/02/1992	134	5	9/02/1992	141	5	24/09/1995	101
6	9/02/1992	132	6	18/05/1998	127	6	30/01/2001	99
7	4/02/2002	128	7	7/08/1998	123	7	21/01/1999	95
8	13/02/1988	127	8	30/08/1996	119	8	24/02/1999	82
9	24/09/1995	126	9	6/09/2006	112	9	13/04/1994	79
10	18/05/1998	122	10	9/04/1998	109	10	6/03/1994	76
11	13/05/2003	118	11	8/06/2007	108	11	31/01/2001	76
12	30/08/1996	113	12	5/05/2001	105	12	8/02/1999	69
13	23/03/1984	113	13	4/02/2002	103	13	5/01/1996	62
14	3/02/1990	112	14	7/03/2012	97	14	2/01/1996	60
15	9/04/1998	111	15	19/08/2007	95	15	6/08/1998	58
16	7/03/2012	107	16	21/07/2011	95	16	25/09/1995	56
17	4/08/1986	107	17	19/03/2011	91	17	15/08/1998	56
18	7/08/1998	106	18	4/02/2008	89	18	9/04/1998	55
19	11/06/1991	106	19	14/06/2007	88	19	7/10/1997	54
20	30/01/2001	103	20	34225.375	87	20	23/10/1999	52

2.8. Design Rainfall Data

The design rainfall intensity-frequency-duration (IFD) data, for events up to and including the 100 year ARI event, were obtained from the Bureau of Meteorology's online design rainfall tool. The input parameters for these calculations were sourced from AR&R (1987). Uniform depths of rainfall were applied across the entire catchment as per ARR87 and a summary of the design rainfall depths is provided in Table 7. A comparison of the design rainfall Intensity-Frequency Duration (IFD) data with the February 1993 Event for the Lilyfield and Marrickville gauges is shown on Figure 11.

Table 7: Rainfall IFD Data at the centre of the Hawthorne Canal Catchment

DURATION	Design Rainfall Intensity (mm/hr)						
	1 yr ARI	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI
5 minutes	96.7	124	158	177	203	236	261
6 minutes	90.5	116	148	166	190	221	245
10 minutes	74.2	95.3	122	137	158	184	205
20 minutes	54.3	70.1	90.9	103	119	140	156
30 minutes	44.2	57.2	74.7	84.9	98.4	116	130
1 hour	29.9	38.8	51.1	58.4	67.9	80.4	90
2 hours	19.5	25.4	33.5	38.3	44.5	52.8	59.1
3 hours	15	19.5	25.8	29.5	34.3	40.6	45.5
6 hours	9.6	12.5	16.4	18.7	21.7	25.7	28.8
12 hours	6.18	8.01	10.5	12	13.9	16.4	18.4
24 hours	4.02	5.22	6.85	7.8	9.05	10.7	12
48 hours	2.58	3.35	4.4	5.02	5.82	6.89	7.72
72 hours	1.93	2.5	3.28	3.74	4.34	5.13	5.74

The Probable Maximum Precipitation (PMP) estimates were derived according to Bureau of Meteorology guidelines, namely the *Generalised Short Duration Method* (BoM, 2003). The estimates obtained are summarised in Table 8.

Table 8: PMP Design Rainfall Intensity (mm/hr)

Duration	Design Rainfall Intensity (mm/hr)
30 minutes	470.4
1 hour	345.1
2 hours	219.8
3 hours	164.5
6 hours	102.55

2.9. Previous Studies

2.9.1. Hawthorne SWC 62 Capacity Assessment (SWC, 1998)

This report was prepared by Sydney Water and investigated the current performance of Sydney Water Corporation's Hawthorne SWC 62 and gives an estimate of the impact of simulated urban consolidation on that performance.

The study included a detailed land use investigation, and hydraulic performance of Sydney Water's trunk drainage system.

The drainage data used in the study was limited to the Sydney Water trunk drainage system only and the analysis was undertaken using a spreadsheet analysis based on:

- Rational Method for inflows,
- Approximate capacities of pipes based on grade and area,
- Approximation of channel capacities using Manning's "n" formula, and the
- Hydraulic Grade Line method.

The report notes that this results in an overestimation of flows and ponding depths in the smaller design events modelled. In spite of these limitations, the data provided a useful point of reference for the verification exercise undertaken in Section 6.5.1.

The hydraulic capacity in the main stormwater channel upstream of Marion Street (section B-C) was found to be 82 m^3/s (above the 100 year ARI). Approximately 60% of the existing trunk drainage system is able to contain flows from a 5 year ARI storm event.

2.9.2. West Street Catchment Drainage Study (Dalland and Lucas Pty. Ltd., 1996)

Dalland and Lucas Pty. Ltd. prepared this report on behalf of Marrickville Council in 1996. The catchment area of approximately 55 hectares was bounded by Parramatta Road to the north, Crystal Street to the east, Canterbury Road to the south and West Street to the west.

The objective of this study was to undertake flood estimations and propose mitigation options. This included hydrologic modelling and benefit cost analysis.

ILSAX was used for the hydrologic modelling. The study was unable to calibrate this model due to a lack of data. The model was verified against a basic RatHGL model, which was found to produce similar discharges and hydrographs as the ILSAX model for the 100 year ARI event.

The peak pipe flow and overflow from each of the sub-catchment areas were presented in the report for the range of events modelled. The property affectation was estimated based upon the modelled results and surveyed floor levels, shown in Table 9.

Table 9: Number of properties affected by 100 year ARI for existing conditions (Dalland and Lucas Pty. Ltd., 1996)

Recurrance Interval (Years)	AFL >300mm	AFL 0-300mm	AGL >300mm	AGL 0-300mm	Total Above Floor	Total Above Ground	Total
1	0	3	2	11	3	10	13
2	0	5	5	19	5	19	24
5	0	5	9	29	5	33	38
10	1	9	14	41	10	45	55
20	1	12	23	51	13	61	74
50	3	12	25	61	15	71	86
100	4	18	32	79	22	89	111

Note: AFL – Above Floor Level Flooding; AGL – Above Ground Level Flooding; Total – Total number of properties affected by flooding

Various mitigation options were proposed including:

- Increased pit capacity;
- Increased pipe capacity;
- Formalise overland flowpaths to direct flood waters away from property; and
- Construction of detention basins, such as at Petersham Park (identified as option D8, whereby the surrounding mound is extended across the driveway to Station Street and a pit and pipeline constructed to control discharge from the park into the existing drainage system).

Some of the proposed mitigation options have since been implemented and these are discussed in the catchment changes incorporated into the historic flood modelling within Section 6.3.2.

Remedial works for the existing drainage system were examined based upon CCTV of the stormwater system. The works recommended the replacement or relining of sections of pipeline shown to be damaged or deformed, sometimes occurring due to proximity to tree roots. The conditions prior to remedial works were not included in the calibration flood modelling as the extent, severity and length of time of the damage could not be quantified.

2.9.3. 35A-37 Hawthorne Parade Flood Study (GHD, 1999)

This study was undertaken on behalf of HHH Self Storage in 1999. The study area extended from the Marion Street Bridge to upstream of the site, with the Marion Street Bridge determined to be the controlling structure.

The data used in this study consisted of plan drawings, the Hawthorne Canal SWC 62 Capacity Assessment (discussed in Section 2.9.1), tidal water levels for Parramatta River and survey of Hawthorne Canal within the study area.

RAFTS was used for the hydrologic modelling and HEC-RAS was used for the hydraulic modelling. Flood levels were determined for the 100 year ARI event. A sensitivity analysis was undertaken for variations in impervious percentage applied to the total catchment area, using the values of 50%, 40% and 30%. The peak 100 year ARI flows and levels were found to be relatively insensitive to variations in impervious percentage, as assessed at the site and at the Marion Street Bridge. From this, an impervious percentage of 50% was selected.

The report found that the 100 year ARI peak flood level for the site was 4.83 m AHD. After applying a 300 mm freeboard for floors and a 150 mm freeboard for garages (as per Council requirements at the time), the report concluded that a floor level of 5.13 m AHD and a garage level of 4.98 m AHD was applicable for the site.

2.9.4. 40 Morris Street, Summer Hill Flood Risk Report (Northern Beaches Consulting Engineers Pty. Ltd., 2011)

This report was undertaken by Northern Beaches Consulting Engineers on behalf of the property owner. The flood study was prompted by a Development Application (DA) proposal for the site.

The catchment upstream of this site was reportedly 10.9 ha, with flows for this area calculated using the rational method equation. The upstream catchment runoff was found to be around 8.7 m^3/s for the 100 year ARI storm event. The stormwater culvert located through the middle of the property was estimated to have a capacity of 6.4 m^3/s based on the Manning's equation. The remaining 2.3 m^3/s was adopted as the overland flow for the site in a 100 year ARI storm event. The overland flow was modelled for the site using the HEC-RAS software.

The maximum ponding depth on the site was estimated to be 280 mm under pre-development conditions and 355 mm under post-development conditions. The increase was estimated to occur along the boundary between the site and 38 Morris Street. However, the report considered that the increase in flood levels would not have an adverse effect on the adjoining property as the front porch level was 380 mm above the estimated post-development flood levels and thereby above the Council required freeboard (at the time of the assessment) of 300 mm.

The report provided recommendations for the proposed development that included:

- Structural design be assessed for hydraulic forces (for the 100 year ARI flood level);
- Flood protection measures be implemented (specifically, a flood protection wall and raised entry area);
- Building over and adjacent to a Sydney Water drainage line (as per correspondence with Sydney Water);
- Types of construction materials; and
- Waterproofing methods.

2.9.5. Leichhardt Flood Study (Cardno Lawson Treloar, 2010)

Cardno Lawson Treloar carried out this flood study on behalf of Leichhardt Council in 2010.

XP-RAFTS was the hydrologic model used for the catchment area outside of the Leichhardt Council LGA. The hydrologic model routes the rainfall to estimate the runoff hydrograph, which was applied to the boundaries of the hydraulic model of the study area.

The "direct rainfall" (also known as "rainfall on grid") method was used within the study area, employing the SOBEK hydraulic model software package. The Direct Rainfall method routes the rainfall in the 2D hydraulic model directly instead of via the traditional hydrologic model.

The hydrologic and hydraulic models were jointly calibrated and verified. The February 1993 flood event was used for calibration and the January 1991 and April 1998 events were used for validation. The models were further verified against Sydney Water Studies (for the Whites Creek and Johnstons Creek catchments) and by comparing the Direct Rainfall method with XP-RAFTS. The Sydney Water Hawthorne Canal Study (discussed in Section 2.9.1) did not extend further downstream of Marion Street, within which the Leichhardt Council area is located.

The calibration locations within the Hawthorne Canal catchment are summarised in Table 10. These observed flood levels were sourced from LMC File No. 225419F1 provided to Cardno Lawson Treloar for the purpose of the flood study.

Table 10: Calibration Details – 17 February 1993 Event (Cardno Lawson Treloar, 2010)

Location ID	Location	Description	Observed Flood Level (m AHD)	Modelled Flood Level (m AHD)	Comments
M	Corner of George St and McAleer St	Flood Level 380mm above loading dock level	10.52	11.10	GL=10.3 m(AHD). The height of the loading dock is 0.38m above GL, and the flood level is 380mm above the loading dock (10.3+0.38+0.38 = 11.06mAHD)
N	Upward St	Flood level at end entrance ramp	7.93	9.20	GL=8.15 m(AHD). The ground level is higher than the recorded flood level.
P	Corner of Darley Rd and Elswich St	N/A	2.56	2.52	Exact location of observed level is uncertain.

The design flood events were the 5, 10, 20, 50 and 100 year ARI and PMF events. Sensitivity of the 100 year ARI event to various parameters was investigated, including:

- Blockage of inlet pits by 50% - Found to be insensitive with impacts of ± 0.1 m;
- Blockage of culverts and bridges using Wollongong City Council's blockage policy – Increased flood levels up to 0.5 m upstream of the railway embankment in the Hawthorne Canal catchment;
- Hydraulic roughness increased and decreased by 20% - Found to have a relatively minor impact.

3. STUDY METHODOLOGY

3.1. Approach

The approach adopted in flood studies to determine design flood levels largely depends upon the objectives of the study and the quantity and quality of the data (survey, flood, rainfall, flow etc). High quality survey datasets were available for this study, which enabled a detailed topographic model of the catchment to be established. However the historical data (such as rainfall, stream-flows and flood mark data) were relatively limited. A diagrammatic representation of the flood study process is shown in Diagram 1.

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

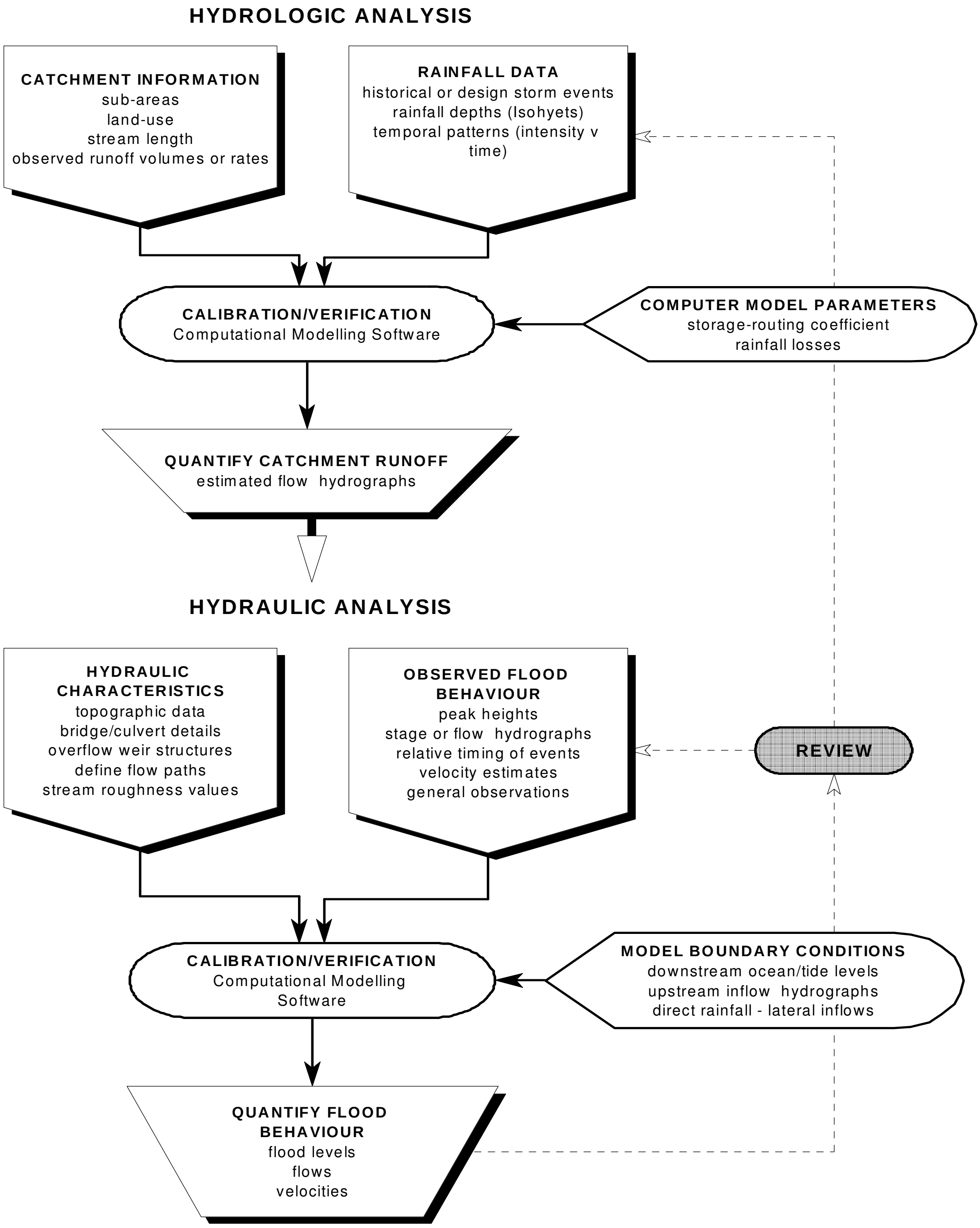
1. hydrologic modelling to convert rainfall estimates to overland flow and stream runoff; and
2. hydraulic modelling to estimate overland flow distributions, flood levels and velocities.

Good historical flood data facilitates calibration of the models and increases confidence in the estimates. The calibration process is undertaken by altering model input parameters to match the reproduction of observed catchment flooding. Recorded rainfall and stream-flow data area are required for calibration of the hydrologic model, while historic records of flood levels, velocities and inundation extents can be used for the calibration of hydraulic model parameters. In the absence of such data, model verification is the only option and a detailed sensitivity analysis of the different model input parameters constitutes current best practice.

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

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

Diagram 1: Flood Study Process



In view of the above, the broad approach adopted for this study was to use a widely utilised and well-regarded hydrologic model to conceptually model the rainfall concentration phase (including runoff from roof drainage systems, gutters, etc.). The hydrologic model used design rainfall patterns specified in Reference 15 and the runoff hydrographs were then used in a hydraulic model to estimate flood depths, velocities and hazard in the study area.

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

3.2. Hydrologic Model

DRAINS (Reference 14) is a hydrologic/hydraulic model that can simulate the full storm hydrograph and is capable of describing the flow behaviour of a catchment and pipe system for real storm events, as well as statistically based design storms. It is designed for analysing urban or partly urban catchments where artificial drainage elements have been installed.

The DRAINS model is broadly characterised by the following features:

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

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

Runoff hydrographs for each subcatchment area are calculated using the time area method and the conveyance of flow through the drainage system is then modelled using unsteady flow calculations. This provides improved prediction of hydraulic behaviour, consistency in design, and greater freedom in selecting pipe slopes. It requires more complicated design procedures, since pipe capacity is influenced by upstream and downstream conditions.

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

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

3.3. Hydraulic Model

The availability of high quality LIDAR/ALS data means that the study area is suitable for two-dimensional (2D) hydraulic modelling. Various 2D software packages are available and the TUFLOW package (Reference 19) was adopted as it is widely used in Australia and WMAwater have extensive experience with the model.

The Hawthorne Canal study area consists of a wide range of developments, with residential, commercial and open space areas. Overland flood behaviour in the catchment is generally two-dimensional, with flooding along road and railway areas prone to ponding. For this catchment, the study objectives require accurate representation of the overland flow system including kerbs and gutters and defined drainage controls.

The 2D model is capable of dynamically simulating complex overland flow regimes and interactions with subsurface drainage systems. It is especially applicable to the hydraulic analysis of flooding in urban areas which is typically characterised by short-duration events and a combination of underground piped and overland flow behaviour.

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

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

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

In TUFLOW the ground topography is represented as a uniformly-spaced grid with a ground elevation and a Manning's "n" roughness value assigned to each grid cell. The grid cell size is determined as a balance between the model result definition required and the computer run time (which is largely determined by the total number of grid cells). The model resolution aims to differentiate between drainage waters (typically depths below 150 mm) and local overland flow or trunk/mainstream flow (typically depths greater than 150 mm).

3.4. Design Flood Modelling

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

- some calibration was undertaken following information obtained during the community consultation;
- design outflows for localised subcatchments were obtained from the DRAINS hydrologic model and applied as inflows to the TUFLOW model;
- sensitivity analysis was undertaken to assess the relative effect of changing various TUFLOW modelling parameters.

4. HYDROLOGIC MODEL

4.1. Sub-catchment Definition

The total Hawthorne Canal catchment represented by the DRAINS model is 714 ha comprising of 298 ha in the Ashfield LGA, 212 ha in the Marrickville LGA and 205 ha in the Leichhardt LGA. This area has been represented by a total of 401 subcatchments (with 211 within Ashfield, 157 within Marrickville, 13 within Leichhardt and 20 subcatchments shared between the LGA's) giving an average sub-catchment size of approximately 1.8 ha. The sub-catchment delineation ensures that where hydraulic controls exist that these are accounted for and able to be appropriately incorporated into the hydraulic routing. The sub-catchment layout is shown in Figure 12.

4.2. Impervious Surface Area

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

DRAINS categorises these surface areas as either:

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

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

Table 11: Impervious Percentage per Land-use

Land-use Category	Impervious Percentage
Residential Property	85% Impervious
Commercial or Industrial Property	100% Impervious
Public Recreation Area	20% Impervious
Private Recreation Area	50% Impervious
Vegetation	0% Impervious
Roadway	100% Impervious

The proportion of each land-use category within a sub-catchment was determined based upon the hydraulic model roughness schematisation, shown in Figure 14. Although, further categorisation was undertaken on the property areas to specify residential, commercial or vacant land for each property lot based upon the cadastre provided by SWC.

The impervious percentages attributed to each land-use category were estimated based on aerial observation of a representative area, examples of which are shown in Photo 11 and Photo 12. Within these, the impervious area is shaded in red and the representative land-use area is outlined in blue.

Photo 11: Residential Area

Photo 12: Commercial Area

4.3. Rainfall Losses

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

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

The adopted parameters are summarised in Table 12. These are consistent with the parameters adopted in the adjacent catchment of Dobroyd Canal (WMAwater, 2013).

Table 12: Adopted DRAINS hydrologic model parameters

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

5. HYDRAULIC MODEL

5.1. Digital Elevation Model

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

The model uses a regularly spaced computational grid, with a cell size of 3 m by 3 m. This resolution was adopted as it provides an appropriate balance between providing sufficient detail for roads and overland flow paths, while still resulting in workable computational run-times. The model grid was established by sampling from a 1 m by 1 m DEM. This DEM was generated from a triangulation of filtered ground points from the LiDAR dataset, discussed in Section 2.3. This DEM is shown in Figure 3.

The TUFLOW hydraulic model includes the Hawthorne Canal catchment drainage into Iron Cove. The 2D model extends from New Canterbury Road to the south, down to Iron Cove resulting in a total area of 710 ha. The extents of the TUFLOW model are shown in Figure 2.

5.2. Boundary Locations

5.2.1. Inflows

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

5.2.2. Downstream Boundary

The downstream boundary was located at the confluence of the trunk drainage system with Iron Cove, as shown in Figure 13. At this location, the 2D domain is operating so the boundary condition was applied to this domain within the hydraulic model.

5.3. Roughness Co-efficient

The hydraulic efficiency of the flow paths within the TUFLOW model is represented in part by the hydraulic roughness or friction factor formulated as Manning's "n" values. This factor describes the net influence of bed roughness and incorporates the effects of vegetation and other features which may affect the hydraulic performance of the particular flow path.

The spatial variation in Manning's "n" values is shown on Figure 14. The Manning's "n" values adopted for these areas, including flowpaths (overland, pipe and in-channel), are shown in Table 13. These values have been adopted based on site inspection and past experience in similar floodplain environments. The values are consistent with typical values in the literature (Chow, 1959 and Henderson, 1966).

Table 13: Manning's "n" values adopted in TUFLOW

Surface	Manning's "n" Adopted
Pipes	0.02
Roads and Footpaths	0.02
Light Vegetation (such as parks with predominantly grass surfaces)	0.04
General Overland Areas	0.04
Properties	0.05
Medium-Heavy Vegetation	0.08

5.4. Hydraulic Structures

5.4.1. Buildings

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

5.4.2. Fencing and Obstructions

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

5.4.3. Bridges

Key hydraulic structures were included in the hydraulic model, as shown in Figure 13. Culverts and bridges were modelled as 1D features within the 1D channels, with the purpose of maintaining continuity within the model. Roadways underneath the railway embankment that contribute to the conveyance of flow were modelled in the 2D domain using a TUFLOW feature specifically designed for this purpose, whereby the energy losses and blockage caused by any piers and the deck can be applied directly to the grid cells.

The modelling parameter values for the culverts and bridges were based on the geometrical properties of the structures, which were obtained from detailed survey, photographs taken during site inspections, and previous experience modelling similar structures. Examples of key features included in the model are shown in Photo 13 to Photo 16.

Photo 13: Lilyfield Road Bridge

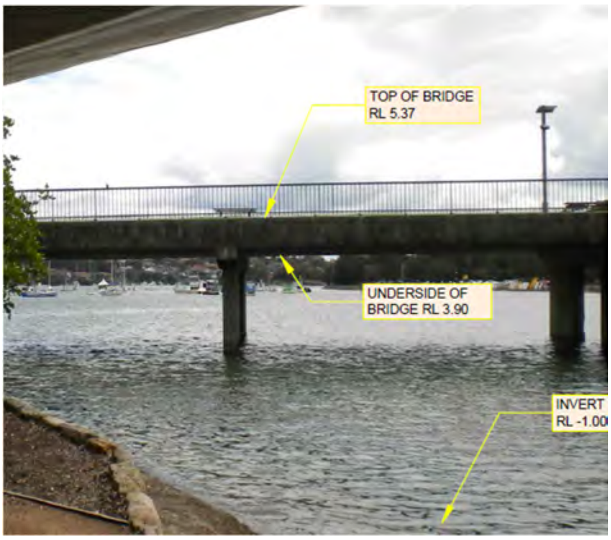


Photo 14: Marion Street Bridge

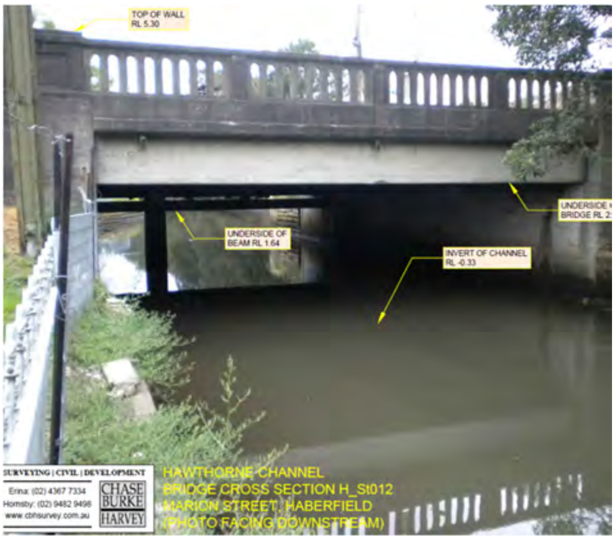


Photo 15: Parramatta Road Bridge

Photo 16: Longport Street Bridge

5.4.4. Sub-surface Drainage Network

Figure 13 shows the location and extent of drainage lines within the study catchment that have been included in the TUFLOW model. The drainage system defined in the model comprises:

- 163 open channel segments;
- 656 pipes; and
- 787 pits and nodes.

5.5. Blockage Assumptions

Blockage of hydraulic structures can occur with the transportation of a number of materials by flood waters. This includes vegetation, garbage bins, building materials and cars, the latter of which has been seen post-flood in Newcastle. However, the disparity in materials that may be mobilised within a catchment can vary greatly.

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

The potential effects of blockage include:

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

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

- the Queensland Urban Drainage Manual (Department of Natural Resources and Water, 2008) (Reference 16);
- AR&R Revision Project 11 Blockage of Hydraulic Structures (Engineers Australia, 2013) (Reference 2); and
- the policies of various local authorities and infrastructure agencies.

The guidelines proposed by the AR&R Revision Project 11 utilise generic blockage factors presented in Table 14.

Table 14: Suggested 'Design' and 'Severe' Blockage Conditions for Various Structures (Engineers Australia, 2013)

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

Current modelling has been undertaken assuming no blockage of pipes, culverts and bridges greater than 450 mm in diameter. Pipes less than 450 mm in diameter were conservatively assumed to be completely blocked.

Various scenarios have been investigated to assess the catchment's sensitivity to 20% and 50% blockage and the results of this are discussed in Section 8.4.2. Two scenarios were examined; either blockage of all pipes, or blockage of bridges and culverts over the open channel. The blockage of bridges and culverts over the open channel excluded the bridges at the confluence with Iron Cove, namely the City West Link, due to the bridges clear opening height exceeding 5 m.

Blockage was assumed to occur laterally across the cross-section. This is particularly relevant for structures that contain piers around which debris may become entangled. Alternative applications of blockage include reducing the cross-sectional area upwards from the invert.

6. MODEL CALIBRATION AND VERIFICATION

6.1. Introduction

Prior to use for defining design flood behaviour it is important that the performance of the overall modelling system be substantiated. Calibration involves modifying the initial model parameter values to produce modelled results that concur with observed data. Validation is undertaken to ensure that the calibration model parameter values are acceptable in other storm events with no additional alteration of values. Best practice is that the modelling system should be calibrated to one historical event and validated using multiple historical events. To facilitate this there needs to be adequate historical flood observations and sufficient pluviometer rainfall data.

However, there are several limitations which prevent a thorough calibration of the hydrologic and hydraulic models:

- There is only a limited amount of historical flood information available for the study area. For example, in Sydney (east of Parramatta) there are only two water level recorders in urban catchments similar to that of the study area;
- Rainfall records for past floods are limited and there is a lack of temporal information describing historical rainfall patterns within the catchment (pluviometer records are only available outside the catchment); and
- There appears to be some uncertainty regarding some of the observed flood levels within the Hawthorne Canal catchment, in many cases they were based on anecdotal evidence only.

These limitations are typical of the majority of urban catchments and while the accuracy or relevance of the data is somewhat imperfect, the calibration and validation exercise undertaken here constitutes current best practice.

6.2. Hydrologic Model Verification

A comparison against previous studies of nearby catchments can be undertaken to verify the model. For this study, the hydrologic model from the Rose Bay catchment was compared to Hawthorne Canal catchment. DRAINS was the hydrologic model used in Rose Bay and the catchment is located approximately 10 km from the Hawthorne Canal Catchment.

Sub-catchment	Hawthorne Canal			Rose Bay		
	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	8.2	3.8	0.5	1	0.6	0.7
2	5.0	2.4	0.5	0.4	0.2	0.6
3	1.5	0.8	0.6	0.6	0.4	0.6

The specific yields from the two different DRAINS models were found to be comparable.

6.3. Hydraulic Model Calibration

Calibration of the hydrologic and hydraulic models (known as a joint calibration) was undertaken using the 17th February 1993 event.

The February 1993 event lasted approximately 6 hours with a particularly intense 30 minute burst of rainfall with a recurrence interval exceeding 40 years (see Section 2.7.4). Four measurements of flood levels were recorded subsequent to the event. However, the recorded levels were very localised and situated close to/within the Leichhardt Council area.

Two pluviometer gauges in the surrounding area recorded data for this event, being the Lilyfield and Marrickville gauges. The model was run using rainfall intensity values from both. Modelled water levels were marginally higher for the Lilyfield gauge inputs than those obtained from the Marrickville gauge. However the difference was minimal and the Lilyfield gauge levels were selected for comparison with observed levels due to the closer proximity of the Lilyfield recording site to the centre of the Hawthorne Canal catchment.

6.3.1. Boundary Conditions

The calibration locations were situated in the overland flow areas that were found to be insensitive to downstream boundary conditions, shown in Section 8.4.3. As such, a constant water level of 1.38 m AHD was adopted for the confluence with Iron Cove.

6.3.2. Catchment Conditions

There have been a number of changes to the catchment conditions over recent years. Some have been documented in the *West Street Catchment Drainage Study* (Dalland and Lucas Pty. Ltd., 1996), although there are presumably changes that have occurred for which no records have been maintained. Such changes that may not have been recorded include increased impervious surfaces within the catchment.

Where details were provided for the changes, these have been incorporated into the hydraulic model schematisation as befitting the relevant historical event. Where details were not provided, the current conditions were used. These are summarised in Table 15.

Table 15: Catchment condition changes included in the calibration hydraulic model

Location	1993 Conditions	Existing Conditions
Pipe Upgrade from Brighton Road to Petersham Park (Marrickville Council) (Option D4B)	600mm diameter pipe	750mm diameter pipe
Pipe Upgrade from Railway Street to Palace Street Via Fisher Reserve and Carrington Lane (Marrickville Council) (Option D3)	450mm diameter pipe	• 525mm diameter pipe (across Railway Street) • 600mm diameter pipe (along Carrington Lane and Fisher Reserve) • 975mm diameter pipe (along Palace Street)
Pipe Upgrade O'Connor Street (Ashfield Council)	Pre-Upgrade (assorted pipe sizes)	Additional pipes ranging from 600mm diameter pipes to 1.8m (Wide) by 1.6m (High) pipes
Petersham Pool	Pre-Refurbishment	Post-Refurbishment
Meriton Development	Pre-Development	Post-Development

The *West Street Catchment Drainage Study* (Dalland and Lucas Pty. Ltd., 1996) makes reference to the detention basin within Fort Street High School as having "… been constructed over the past few years". However, as the completion date for these works was unknown and the topographical details of pre-construction were unknown, it was assumed that the current conditions were in effect during the 1993 event.

6.3.3. Results

Comparison between the hydraulic model levels produced in the current study to observed levels are provided in Table 16.

Table 16: Observed vs Modelled Levels – February 1993

Location	Observed Level (m AHD)	Hydraulic Model Level (m AHD)	Difference (m)
Corner of Darley Rd and Elswich St North	2.56	2.59	0.03
Upward St	7.93	7.80	-0.13
Corner of George St and McAleer St	10.52	9.74	-0.78
Parramatta Rd	11.74	11.07	-0.67
Station St	12.18	12.13	-0.05

The modelled results were found to be typically lower than the observed levels. It is possible that blockage due to debris and/or the changes to the pit and pipe system that could not be quantified and incorporated into the hydraulic model (discussed in Section 2.9.2) have resulted in an overestimation of the pit and pipe capacity during this event.

6.4. Hydraulic Model Validation

Accounts of flood levels by the local community for the March 2012 event enabled a tentative validation to be carried out. Cumulative rainfall levels of approximately 100 mm were recorded at the Lilyfield and Marrickville gauge sites however, the rainfall intensities only registered as a 1 year ARI. While recorded levels were comparatively less reliable than the 1993 event, their provenance from throughout the catchment provided valuable information on the overall flood extent as illustrated in Figure 14.

6.4.1. Boundary Conditions

The majority of validation locations were situated in the overland flow areas that were found to be insensitive to downstream boundary conditions, shown in Section 8.4.3. As such, a constant water level of 1.0 m AHD was adopted for the confluence with Iron Cove.

6.4.2. Catchment Conditions

Table 17 summarises the differences between the catchment conditions that were known to be prevailing during the 2012 event and those that were incorporated into the design modelling events as existing conditions.

Table 17: Catchment condition changes included in the validation hydraulic model

Location	2012 Conditions	Existing Conditions
Pipe Upgrade O'Connor Street (Ashfield Council)	Pre-Upgrade (assorted pipe sizes)	Additional pipes ranging from 600mm diameter pipes to 1.8m (Wide) by 1.6m (High) pipes
Petersham Pool	Pre-Refurbishment	Post-Refurbishment
Meriton Development	Pre-Development	Post-Development

6.4.3. Results

Table 18 provides comparison of the hydraulic model levels produced in the current study to observed levels. The observed levels were approximated based upon estimated flood depth provided anecdotal by the community and ALS ground level for the relevant areas.

Table 18: Observed vs Modelled Levels – March 2012

Location	Observed Depth (m) and Approximated Level (m AHD)	Hydraulic Model Level (m AHD)	Difference (m)
Hawthorne Pde, Haberfield	2.98	1.96	-1.02
Eltham St, Dulwich Hill	13.42	13.34	-0.08
Hobbs St, Lewisham	18.21	18.06	-0.15
Corner of Railway Terrace	28.07	27.97	-0.10
Abergeldie St, Dulwich Hill (South)	28.28	27.87	-0.41
Abergeldie St, Dulwich Hill (North)	32.93	32.66	-0.27
Prospect Rd, Summer Hill	33.23	32.75	-0.48
Queen St, Ashfield	38.70	38.68	-0.02

Given the uncertainty of the recorded flood levels in Table 18, the limited accuracy of the LiDAR survey, the resolution of the hydraulic model and the low flood depths involved, the precision required in this area of the model is outside the bounds of certainty and no further adjustments to the hydraulic model were made.

6.5. Hydraulic Model Verification

Verification of the hydraulic model was undertaken by comparing the modelled design results against the 1998 report by SWC.

6.5.1. Comparison with the SWC (1998) report

Reference 17 is a capacity assessment of the Hawthorne Canal undertaken by Sydney water in 1998. The impact of urban consolidation on the quantitative performance of the canal itself and the trunk drainage assets was investigated. The ability of the major pipe network to deal with a 5 year ARI event formed part of the analysis and expected conveyance values in the respective pipes were computed for the likely scenario of such an event taking place. Table 19 and Figure 18 present a comparison between results obtained in the study against those from the current TUFLOW model.

Table 19: SWC (1998) results compared to the current study results – for the 5 year ARI event

Open Channel / Pipe	Location	1998 SWC Study (m^3/s)	Current Study (m^3/s)
A-B	Main Channel	61	51.3
B-C	Main Channel	47	38.7
D-E	Main Channel	46	37.1
J-K	Main Channel	28	22.3
K-L	Main Channel	28	22.3
M-N	Main Channel	27	21.6
P-Q	Main Channel	26	23.1
Q-R	Main Channel	24	22.6
R-S	Main Channel	22	21.9
Y-Z	Main Channel	5	2.9
ZE-ZF	Old Main Channel	2	1.9
ZL-ZM	Old Main Channel	4	2.1
ZM-ZN	Old Main Channel	3	1.9
B-B13	Petersham Park Branch	13	5.5
F-F20	Petersham Branch	4	2.7
G22-G31	Henson Street Branch	3	2.3

Results show reasonable agreement, with the current study values being smaller in some cases than those of Reference 17. This is in agreement with the fact that the empirical methods used in Reference 17 tend to overestimate flows (Section 2.9.1).

7. DESIGN EVENT MODELLING

7.1. Overview

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

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

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

7.2. Critical Duration

To determine the critical storm duration for various parts of the catchment, modelling of the 100 year ARI event was undertaken for a range of design storm durations from 15 minutes to 9 hours, using temporal patterns from AR&R (1987). An envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

It was found that the 30, 60 and 120 minute storms were critical for the majority of the catchment, with the upper reaches of the catchment having a critical duration of 30 minutes, the bulk of the catchment having a critical duration of 60 minutes and the downstream trunk area a critical duration of 120 minutes. The peak flood depths produced for various durations were generally found to be within ±0.05 m throughout the catchment. Given the relatively small change in peak flood levels, the 60 minute duration was taken to be the critical storm duration.

Additionally, the critical storm duration was determined for the PMF event for a range of storm durations, ranging from 30 minutes to 6 hours. Similarly, an envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

It was found that a combination of the 30 minute and 1 hour storm duration was critical in the PMF event. The 1 hour storm duration was critical in the open channel sections, from Etham Street (located upstream of Old Canterbury Road) to the confluence with Iron Cove. The 30 minute storm duration was critical in the remaining catchment area, which accounted for approximately 94% of the flood affected catchment area when compared to the 1 hour duration. Flood levels varied by up to 0.25 m (over 90% of the flood affected area) in areas where the 30 minute duration was critical, whereas where the 1 hour duration was critical flood levels varied by up to 0.25 m (over 4% of the flood affected area) between the two events. Therefore, a peak envelope of the 30 minute and 1 hour storm durations was adopted.

7.3. Downstream Boundary Conditions

In addition to runoff from the catchment, downstream areas can also be influenced by high water levels at the confluence of Iron Cove and the trunk drainage system. Consideration must therefore also be given to accounting for the joint probability to coincident flooding from both catchment runoff and backwater effects.

A full joint probability analysis to consider the interaction of these two mechanisms is beyond the scope of the present study. It is accepted practice to estimate design flood levels in these situations using a ‘peak envelope’ approach that adopts the highest of the predicted levels from the two mechanisms. The constant water level applied to the downstream boundary for events greater than and equal in magnitude to a 20 year ARI rainfall event was 1.38 m AHD, which corresponds to a 20 year ARI tidal event. For rainfall events of a smaller magnitude than the 20 year ARI, a constant water level of 1 m AHD was applied.

For the 2050 and 2100 sea level rise scenarios, a constant water level of 1.78 m AHD and 2.28 m AHD were specified respectively, in accordance with guidelines from the NSW State Government (2010).

7.4. Design Results

The results from this study are presented as:

- Peak flood level profiles in Figure 26;
- Peak flood depths and level contours in Figure 19 to Figure 25;
- Peak flood velocities in Figure 27;
- Provisional hydraulic hazard in Figure 28 to Figure 31;
- Provisional hydraulic categorisation in Figure 32 to Figure 36;
- Preliminary flood emergency response classification of communities in Figure 37; and

The definition and methodology used to derive these categorisations from the results are discussed below.

The results have been provided to Ashfield Council and Marrickville Council in digital format compatible with council’s Geographic Information System (GIS).

7.4.1. Summary of Results

Peak flood levels, depths and flows at key locations within the catchment are summarised below. These key locations coincide with the key locations used for the sensitivity analysis discussed in Section 8. The placement of the key locations is shown in Figure 15.

A tabulated summary of peak flood depth and level results at key locations are detailed in Table

20.

Table 20: Peak Flood Levels (m AHD) and Depths (m) at Key Locations

ID	Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
H01	Open Channel – Upstream of City West Link	Level	1.04	1.05	1.05	1.42	1.43	1.45	2.61
		Depth	2.53	2.54	2.54	2.91	2.92	2.93	4.10
H02	Hawthorne Parade – Near Waratah Street	Level	1.76	1.80	1.82	1.85	1.87	1.89	3.18
		Depth	0.26	0.29	0.31	0.34	0.36	0.38	1.67
H03	Hawthorne Parade – Corner of Battalion Circuit	Level	2.05	2.13	2.16	2.27	2.39	2.49	3.89
		Depth	0.38	0.45	0.49	0.60	0.71	0.81	2.21
H04	Marion Street	Level	4.16	4.23	4.28	4.32	4.35	4.38	5.84
		Depth	0.30	0.38	0.42	0.46	0.49	0.53	1.99
H05	Parramatta Road – Corner of West Street	Level	11.58	11.71	11.78	11.86	11.92	11.98	12.94
		Depth	0.50	0.64	0.70	0.78	0.85	0.90	1.86
H06	Open Channel – Upstream of Longport Street	Level	5.48	6.05	6.43	6.87	7.76	8.27	13.93
		Depth	2.92	3.49	3.87	4.31	5.19	5.71	11.37
H07	Smith Street	Level	10.84	10.96	11.06	11.16	11.24	11.31	13.95
		Depth	0.05	0.17	0.27	0.37	0.45	0.52	3.16
H08	Grosvenor Crescent	Level	25.96	26.25	26.39	26.56	26.72	26.84	27.31
		Depth	0.71	0.99	1.13	1.30	1.46	1.58	2.06
H09	Open Channel – Upstream of Old Canterbury Road	Level	10.96	12.30	13.04	13.59	13.94	14.22	16.16
		Depth	2.58	3.92	4.66	5.21	5.56	5.84	7.78
H10	Hoskins Park – Adjacent of Open Channel	Level	18.35	18.78	18.90	19.10	19.30	19.44	20.99
		Depth	0.02	0.44	0.57	0.77	0.96	1.11	2.65
H11	Old Canterbury Road – Adjacent to Gough Reserve	Level	34.55	34.61	34.65	34.69	34.73	34.76	35.20
		Depth	0.17	0.23	0.27	0.31	0.35	0.38	0.82
H12	Queen Street	Level	40.22	40.24	40.25	40.26	40.28	40.29	40.63
		Depth	0.04	0.06	0.07	0.08	0.10	0.11	0.45

A tabulated summary of peak flows at key locations is presented in Table 21.

Table 21: Peak Flows (m^3/s) at Key Locations

ID	Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Q01	Open Channel – Between the City West Link and Waratah Street	Overland	0.4	0.4	0.3	0.8	1.4	2.9	145.2
		Open Channel	52.3	72.4	81.0	97.3	111.5	128.2	361.8
Q02	Open Channel – Downstream of Battalion Circuit	Overland	0.0	0.0	0.0	0.8	2.9	6.1	235.1
		Open Channel	49.1	69.0	77.5	91.3	100.1	109.5	187.3
Q03	Under Railway Embankment – Marion St	Overland	4.5	7.1	8.6	10.3	12.1	14.1	52.7
Q04	Marion St	Overland	2.3	3.7	4.5	5.4	6.4	7.4	207.7
		Open Channel Culvert	38.8	53.9	60.9	72.5	84.8	95.4	140.7
Q05	Under Railway Embankment – Parramatta Rd	Overland	0.4	0.9	1.2	1.7	2.1	2.6	9.6
Q06	Parramatta Rd	Open Channel Culvert	27.4	38.4	44.5	121.1	103.9	130.3	439.7
Q07	Smith St – Between Edward St and Spencer St	Pipe	3.9	7.7	9.9	13.2	16.3	19.4	75.5
		Overland	4.1	4.1	4.1	4.1	4.1	4.1	4.0
Q08	Davis St	Rail Underpass	0.4	0.6	0.7	0.8	0.9	1.0	25.5
		Open Channel Culvert	3.5	10.7	14.4	19.6	24.3	30.9	116.5
		Overland	11.2	11.5	11.6	11.7	11.7	11.8	12.3
Q09	Union St	Pipe	5.1	8.8	11.0	14.2	17.1	20.0	66.4
		Overland	3.1	3.2	3.2	3.2	3.3	3.3	3.7
Q10	Old Canterbury Rd – Adjacent to Gough Reserve	Pipe	4.9	7.4	9.0	11.2	12.9	14.9	41.8
		Overland	1.7	1.7	1.7	1.7	1.7	1.7	1.9

The tabulated summary of peak velocities within the open channel and overtopping structures traversing the open channel is presented in Table 22.

Table 22: Peak Velocities (m/s) in Open Channel

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
City West Link	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Upstream Open Channel	1.1	1.5	1.6	1.6	1.9	2.2	5.0
Marion St	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	2.2
	Upstream Open Channel	1.9	2.2	2.1	2.2	2.3	2.3	3.3
Parramatta Rd	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Upstream Open Channel	1.9	2.1	2.2	2.7	2.7	2.7	4.8
Longport St	Culvert	2.7	4.0	4.4	5.6	5.5	8.3	13.4
Light Rail Between Old Canterbury Rd and Longport St	Culvert	4.2	6.9	7.1	7.2	7.2	7.2	7.2
Old Canterbury Rd	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Upstream Open Channel	2.5	2.4	2.4	2.4	2.4	2.4	3.2
Light Rail Adjacent to Eltham St	Overtopping Structure	0.0	1.2	1.9	2.3	2.5	2.7	3.4
	Upstream Open Channel	2.9	2.9	2.9	2.9	2.9	2.9	6.1
Davis St	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Upstream Open Channel	3.6	3.7	3.7	3.8	3.8	3.8	4.8

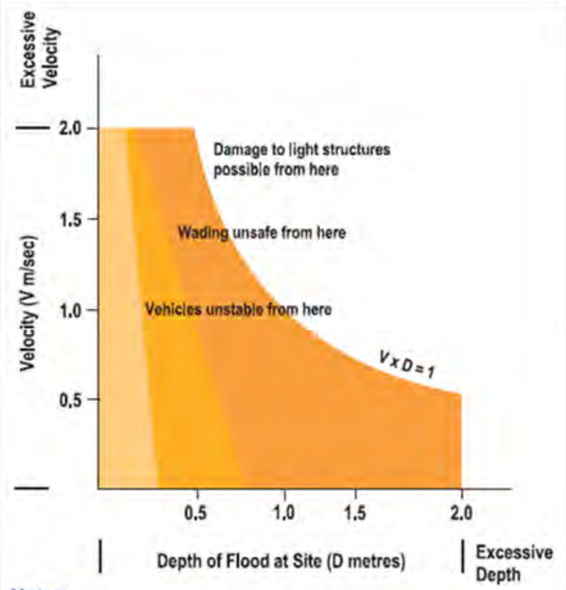
Longitudinal flood level profiles along the open channel are provided in Figure 26. The profiles illustrate the varying degrees of afflux caused by structures within the Hawthorne Canal catchment. Minimal afflux was shown to occur across small pedestrian bridges and bridges with culvert obverts higher in elevation than the peak flood levels. Alternatively, culverts and bridges that cause a significant restriction to flow produced significant afflux across the structure, with greater flood levels upstream of the restriction than downstream.

7.4.2. Provisional Flood Hazard Categorisation

Hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual (Reference 10), the relevant section of which is shown in Diagram 2. For the purposes of this report, the transition zone presented in Diagram 2 (L2) was considered to be high hazard.

Maps of provisional hydraulic hazard in the Hawthorne Canal catchment are presented in Figure 28 to Figure 31.

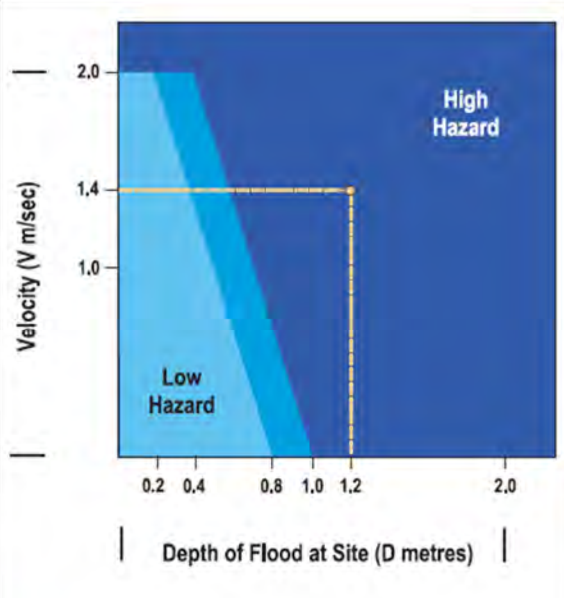
Diagram 2: (L1) Velocity and Depth Relationship; (L2) Provisional Hydraulic Hazard Categories (NSW State Government, 2005)



Notes

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

Derived from laboratory testing and flood conditions which caused damage.



Notes

The degree of hazard may be either -

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

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

Example:

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

7.4.3. Provisional Hydraulic Categorisation

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

For this study, hydraulic categories were defined by the following criteria, which correspond in part with the criteria proposed by Howells et. al. (2003) (Reference 6):

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

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

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

However, councils are increasingly moving away from the practice of defining Floodway, Flood Storage and Flood Fringe, as the mapping of Flood Fringe may allow landowners to bypass a Council Development Application and instead apply to a private certifier, under the 2008 Exempt and Complying SEPP. To avoid this, a "Low Risk" and "High Risk" classification was adopted where:

- High Risk corresponds with areas classified as Floodway and Flood Storage; and
- Low Risk corresponds with areas classified as Flood Fringe.

Figure 32 to Figure 36 show the provisional hydraulic categorisations for the Hawthorne Canal catchment for the 5 year ARI, 20 year ARI, 100 year ARI and PMF events.

7.4.4. Preliminary Flood Emergency Response Classification of Communities

The Floodplain Development Manual, 2005 (Reference 10) requires flood studies to address the management of continuing flood risk to both existing and future development areas. As continuing flood risk varies across the floodplain so does the type and scale of emergency response problem and therefore the information necessary for effective Emergency Response Planning (ERP). Classification provides an indication of the vulnerability of the community in flood emergency response and identifies the type and scale of information needed by the SES to assist in emergency response planning (ERP).

Criteria for determining flood ERP classifications and an indication of the emergency response required for these classifications are provided in the Floodplain Risk Management Guideline, 2007 (Flood Emergency Response Planning: Classification of Communities). Table 23 summarises the response required for areas of different classification. However, these may vary depending on local flood characteristics and resultant flood behaviour, i.e. in flash flooding or overland flood areas.

Table 23: Response Required for Different Flood ERP Classifications

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

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

ERP Classification for the Hawthorne catchment is shown in Figure 37. Areas to which road access is likely to be entirely cut off during a 100 year ARI event are classified as Low Flood Island. These high priority areas include the retirement home St Joan of Arc Villa on Hawthorne Parade, the area surrounding Battalion Circuit, and the corner between Hawthorne Parade and Parramatta Road. Further upstream, the area around Smith Street, Weston Street and Elizabeth Avenue are also classified as Low Flood Islands. Other areas such as the stretch between Queen Street and Victoria Street have been classified as High Flood Island as they are only isolated during a PMF event. The areas with Rising Road Access have roads rising steadily uphill and away from the rising floodwaters and therefore people should not be trapped unless they delay their evacuation from their homes.

8. SENSITIVITY ANALYSIS

8.1. Overview

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

- Routing Lag: The hydrologic routing length values were increased and decreased by 20% for all subcatchments;
- Manning's "n": The hydraulic roughness values were increased and decreased by 20%;
- Blockage (pipes): Sensitivity to blockage of all pipes was assessed for 20% and 50% blockage;
- Blockage (bridges): Sensitivity to blockage of culverts and bridges over the open channel (excluding the City West Link Bridge and adjacent pedestrian bridge, due to clear opening heights of greater than 5 m) was assessed for 20% and 50% blockage;
- Climate Change (Rainfall Increase): Sensitivity to rainfall/runoff estimates were assessed by increasing the rainfall intensities by 10%, 20% and 30% as recommended under current guidelines;
- Climate Change (Sea Level Rise): Sea level rise scenarios of 0.4 m and 0.9 m were assessed.

8.2. Climate Change Background

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

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

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

8.2.1. Rainfall Increase

The Bureau of Meteorology has indicated that there is no intention at present to revise design rainfalls to take account of the potential climate change, as the implications of temperature changes on extreme rainfall intensities are presently unclear, and there is no certainty that the changes would in fact increase design rainfalls for major flood producing storms. There is some recent literature by CSIRO that suggests extreme rainfalls may increase by up to 30% in parts of NSW (in other places the projected increases are much less or even decrease); however this information is not of sufficient accuracy for use as yet (Reference 13).

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

Projected increases to evaporation are also an important consideration because increased evaporation would lead to generally dryer catchment conditions, resulting in lower runoff from rainfall. Mean annual rainfall is projected to decrease, which will also result in generally dryer catchment conditions. The influence of dry catchment conditions on river runoff is observable in climate variability using the Indian Pacific Oscillation (IPO) index (Reference 20). Although mean daily rainfall intensity is not observed to differ significantly between IPO phases, runoff is significantly reduced during periods with fewer rain days.

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

In light of this uncertainty, the NSW State Government advice (Reference 13) recommends sensitivity analysis on flood modelling should be undertaken to develop an understanding of the effect of various levels of change in the hydrologic regime on the project at hand. Specifically, it is suggested that increases of 10%, 20% and 30% to rainfall intensity be considered.

8.3. Sea Level Rise

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

The Policy Statement says:

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

In light of this uncertainty, the NSW State Government's advice is subject to periodical review. As of 2012 and after the commencement of this Flood Study, the NSW State Government withdrew endorsement of sea level rise predictions but still require sea level rise to be considered. At the commencement of this Flood Study the benchmarks required Council to plan for projected sea level rise of 0.4 m by 2050 and 0.9 m by 2100 (NSW State Government, 2010), relative to 1990 levels.

8.4. Results

The sensitivity scenario results were compared to the 100 year ARI rainfall event with the 20 year ARI ocean level. A summary of peak flood level and peak flow differences at various locations are provided in:

- Table 24 and Table 25 for variations in routing and roughness;
- Table 26 and Table 27 for variations in blockage;
- Table 28 and Table 29 for variations in climate conditions.

Comparison of peak flood levels have been highlighted such that yellow highlighting indicates that the magnitude of the change is greater than 0.1 m, while red highlighting indicates changes greater than 0.3 m in magnitude.

8.4.1. Routing and Roughness Variations

Overall peak flood level results were shown to be relatively insensitivity to variations in the routing parameter and increases to the roughness parameter. Generally, these results were found to be within ± 0.1 m, which can usually be accommodated within the freeboard (typically 0.5 m), applied to the 100 year ARI results to determine the Flood Planning Levels.

However, decreasing the roughness parameter resulted in increased peak flood levels at two key locations. These locations (the open channel section upstream of Longport Street and the open channel section upstream of Old Canterbury Road) are both influenced by downstream hydraulic structures. As such, the cumulative effects of decreased attenuation upstream of these locations resulted in a faster concentration of flows at these flow constrictions.

Table 24: Results of Sensitivity Analysis – 100 year ARI Depths (m)

ID	Location	Peak Flood Depth 100 yr ARI	Difference with 100 yr ARI (m)			
			Routing Decreased by 20%	Routing Increased by 20%	Roughness Decreased by 20%	Roughness Increased by 20%
H01	Open Channel – Upstream of City West Link	2.93	0.00	0.00	0.00	0.00
H02	Hawthorne Parade – Near Waratah Street	0.38	0.00	0.00	-0.01	0.01
H03	Hawthorne Parade – Corner of Battalion Circuit	0.81	0.00	0.00	0.00	0.00
H04	Marion Street	0.53	0.00	0.00	-0.02	0.03
H05	Parramatta Road – Corner of West Street	0.90	0.00	0.00	-0.01	0.00
H06	Open Channel – Upstream of Longport Street	5.71	0.02	-0.02	0.21	-0.25
H07	Smith Street	0.52	0.00	0.00	0.00	-0.01
H08	Grosvenor Crescent	1.58	0.00	0.00	0.00	0.00
H09	Open Channel – Upstream of Old Canterbury Road	5.84	0.01	0.00	0.05	-0.06
H10	Hoskins Park – Adjacent of Open Channel	1.11	0.00	0.00	0.02	-0.02
H11	Old Canterbury Road – Adjacent to Gough Reserve	0.38	0.00	0.00	-0.03	0.02
H12	Queen Street	0.11	0.00	0.00	-0.03	0.03

Table 25: Results of Sensitivity Analysis – 100 year ARI Flows (m^3/s)

ID	Location	Type	100 yr ARI	Routing Decreased by 20%	Routing Increased by 20%	Roughness Decreased by 20%	Roughness Increased by 20%
Q01	Open Channel – Between the City West Link and Waratah Street	Overland	2.9	2.9	2.8	3.9	2.1
		Open Channel	128.2	128.3	127.7	130.1	125.2
Q02	Open Channel – Downstream of Battalion Circuit	Overland	6.1	6.1	6.1	8.0	4.8
		Open Channel	109.5	109.7	109.4	111.0	108.0
Q03	Under Railway Embankment – Marion St	Overland	14.1	14.3	14.0	14.6	13.8
Q04	Marion St	Overland	7.4	7.5	7.3	7.6	7.2
		Open Channel Culvert	95.4	95.7	95.7	98.5	93.1
Q05	Under Railway Embankment – Parramatta Rd	Overland	2.6	2.6	2.6	2.8	2.6
Q06	Parramatta Rd	Open Channel Culvert	130.3	112.6	120.6	125.4	130.6
Q07	Smith St – Between Edward St and Spencer St	Pipe	19.4	19.5	19.3	20.2	18.4
		Overland	4.1	4.1	4.1	4.1	4.1
Q08	Davis St	Rail Underpass	1.0	1.1	1.0	1.1	1.0
		Open Channel Culvert	30.9	31.0	30.8	32.6	29.0
		Overland	11.8	11.8	11.8	11.8	11.8
Q09	Union St	Pipe	20.0	20.1	19.9	20.7	19.3
		Overland	3.3	3.3	3.3	3.3	3.3
Q10	Old Canterbury Rd – Adjacent to Gough Reserve	Pipe	14.9	15.1	14.7	15.0	14.7
		Overland	1.7	1.7	1.7	1.7	1.7

8.4.2. Blockage Variations

Peak flood level results were found to be relatively insensitivity to blockage of the underground pipes in the drainage system. In all but one location, blockage of the pipes resulted in less than a 0.1 m variation in peak flood levels. Grosvenor Crescent was the exception due to the limited provision for alternative conveyance of flow that cannot be drained via the pit and pipe system. This is discussed further in Section 10.1.2, where Grosvenor Crescent is identified as a hotspot.

Generally, blockage of bridge and culvert structures over the open channel resulted in increased flood levels in the vicinity of the channel. However, locations subject to overland flow were relatively insensitive to this blockage scenario.

Table 26: Results of Blockage Analysis – 100 year ARI Depths (m)

ID	Location	Peak Flood Depth 100 yr ARI	Difference with 100 yr ARI (m)			
			Blockage (Pipes) by 20%	Blockage (Pipes) by 50%	Blockage (Bridges) by 20%	Blockage (Bridges) by 50%
H01	Open Channel – Upstream of City West Link	2.93	0.00	0.00	0.00	-0.02
H02	Hawthorne Parade – Near Waratah Street	0.38	0.00	0.01	0.00	0.00
H03	Hawthorne Parade – Corner of Battalion Circuit	0.81	-0.01	-0.02	0.02	0.01
H04	Marion Street	0.53	0.00	0.00	0.00	0.00
H05	Parramatta Road – Corner of West Street	0.90	0.01	0.04	0.00	0.00
H06	Open Channel – Upstream of Longport Street	5.71	-0.03	-0.06	1.32	2.50
H07	Smith Street	0.52	0.02	0.05	0.00	0.01
H08	Grosvenor Crescent	1.58	0.06	0.12	0.00	0.00
H09	Open Channel – Upstream of Old Canterbury Road	5.84	0.00	-0.01	0.19	0.42
H10	Hoskins Park – Adjacent of Open Channel	1.11	-0.01	0.01	0.07	0.18
H11	Old Canterbury Road – Adjacent to Gough Reserve	0.38	0.01	0.01	0.00	0.00
H12	Queen Street	0.11	0.00	0.01	0.00	0.00

Table 27: Results of Blockage Analysis – 100 year ARI Flows (m^3/s)

ID	Location	Type	100 yr ARI	Blockage (Pipes) by 20%	Blockage (Pipes) by 50%	Blockage (Bridges) by 20%	Blockage (Bridges) by 50%
Q01	Open Channel – Between the City West Link and Waratah Street	Overland	2.9	2.8	2.7	2.2	1.0
		Open Channel	128.2	127.4	126.6	121.0	103.6
Q02	Open Channel – Downstream of Battalion Circuit	Overland	6.1	5.8	5.4	5.1	3.2
		Open Channel	109.5	108.9	107.9	104.1	91.8
Q03	Under Railway Embankment – Marion St	Overland	14.1	14.1	14.1	14.1	14.1
Q04	Marion St	Overland	7.4	7.4	7.4	7.4	9.5
		Open Channel Culvert	95.4	94.8	93.1	89.8	71.9
Q05	Under Railway Embankment – Parramatta Rd	Overland	2.6	2.6	2.6	2.6	2.6
Q06	Parramatta Rd	Open Channel Culvert	130.3	119.7	118.2	63.3	58.3
Q07	Smith St – Between Edward St and Spencer St	Pipe	19.4	20.3	21.5	19.5	19.5
		Overland	4.1	3.2	1.9	4.1	4.1
Q08	Davis St	Rail Underpass	1.0	1.0	1.0	1.0	1.0
		Open Channel Culvert	30.9	30.6	31.2	33.3	37.3
		Overland	11.8	11.8	11.9	9.3	4.8
Q09	Union St	Pipe	20.0	20.7	21.9	20.0	20.0
		Overland	3.3	2.7	1.7	3.3	3.3
Q10	Old Canterbury Rd – Adjacent to Gough Reserve	Pipe	14.9	15.2	15.5	14.9	14.9
		Overland	1.7	1.4	0.9	1.7	1.7

8.4.3. Climate Variations

The effect of increasing the design rainfalls by 10%, 20% and 30% has been evaluated for the 100 year ARI event with impacts on peak flood levels observed throughout the study area. The 100 year ARI event with a rainfall increase of 30% is approximately equivalent to a 500 year ARI event in present day conditions and an impact on flood levels particularly in flow paths/storage areas is not unexpected.

The sea level rise scenarios had very little impact on flood levels within the catchment except along Hawthorne Parade and within the open channel adjacent to Hawthorne Parade. The sea level rise impacts along this section were found to decrease with increasing distance from the Iron Cove confluence.

Table 28: Results of Climate Change Analysis – 100 year ARI Depths (m)

ID	Location	Peak Flood Depth 100 yr ARI	Difference with 100 yr ARI (m)				
			Rainfall Increase 10%	Rainfall Increase 20%	Rainfall Increase 30%	2050 Sea Level Rise + 0.4 m	2100 Sea Level Rise + 0.9 m
H01	Open Channel – Upstream of City West Link	2.93	0.01	0.02	0.04	0.38	0.86
H02	Hawthorne Parade – Near Waratah Street	0.38	0.01	0.05	0.10	0.15	0.53
H03	Hawthorne Parade – Corner of Battalion Circuit	0.81	0.07	0.13	0.19	0.05	0.23
H04	Marion Street	0.53	0.03	0.05	0.08	0.00	0.00
H05	Parramatta Road – Corner of West Street	0.90	0.05	0.09	0.13	0.00	0.00
H06	Open Channel – Upstream of Longport Street	5.71	0.79	1.49	1.97	0.00	-0.01
H07	Smith Street	0.52	0.07	0.12	0.18	0.00	0.00
H08	Grosvenor Crescent	1.58	0.08	0.13	0.17	0.00	0.00
H09	Open Channel – Upstream of Old Canterbury Road	5.84	0.19	0.34	0.52	0.00	0.00
H10	Hoskins Park – Adjacent of Open Channel	1.11	0.14	0.27	0.40	0.00	0.00
H11	Old Canterbury Road – Adjacent to Gough Reserve	0.38	0.03	0.06	0.09	0.00	0.00
H12	Queen Street	0.11	0.02	0.03	0.05	0.00	0.00

Table 29: Results of Climate Change Analysis – 100 year ARI Flows (m^3/s)

ID	Location	Type	100 yr ARI	Rainfall Increase 10%	Rainfall Increase 20%	Rainfall Increase 30%	2050 Sea Level Rise + 0.4 m	2100 Sea Level Rise + 0.9 m
Q01	Open Channel – Between the City West Link and Waratah Street	Overland	2.9	4.4	6.0	7.6	11.7	27.4
		Open Channel	128.2	141.3	153.4	164.3	122.2	112.3
Q02	Open Channel – Downstream of Battalion Circuit	Overland	6.1	9.2	12.3	15.7	10.3	21.4
		Open Channel	109.5	116.7	122.4	128.2	108.2	98.2
Q03	Under Railway Embankment – Marion St	Overland	14.1	15.8	17.7	19.4	14.1	14.1
Q04	Marion St	Overland	7.4	8.3	9.4	10.2	7.4	7.4
		Open Channel Culvert	95.4	103.8	111.2	117.8	96.2	96.0
Q05	Under Railway Embankment – Parramatta Rd	Overland	2.6	3.1	3.5	4.0	2.6	2.6
Q06	Parramatta Rd	Open Channel Culvert	130.3	125.3	136.4	136.7	67.9	66.9
Q07	Smith St – Between Edward St and Spencer St	Pipe	19.4	22.3	25.2	28.1	19.4	19.5
		Overland	4.1	4.1	4.1	4.1	4.1	4.1
Q08	Davis St	Rail Underpass	1.0	1.1	1.3	1.4	1.0	1.0
		Open Channel Culvert	30.9	35.8	40.1	45.5	30.9	30.9
		Overland	11.8	11.8	11.9	11.9	11.8	11.8
Q09	Union St	Pipe	20.0	22.7	25.5	28.2	20.0	20.0
		Overland	3.3	3.4	3.4	3.4	3.3	3.3
Q10	Old Canterbury Rd – Adjacent to Gough Reserve	Pipe	14.9	16.8	18.6	20.3	14.9	14.9
		Overland	1.7	1.7	1.8	1.8	1.7	1.7

9. PRELIMINARY FLOOD PLANNING AREAS

9.1. Background

Land use planning is considered to be one of the most effective means of minimising flood risk and damages from flooding. The Flood Planning Area (FPA) identifies land that is subject to flood related development controls and the Flood Planning Level (FPL) is the minimum floor level applied to new developments within the FPA.

The process of defining FPA's and FPL's is somewhat complicated by the variability of flow conditions between mainstream and local overland flow, particularly in urban areas. The more traditional approaches typically having been developed for riverine environments and mainstream flow.

Defining the area of flood affectation due to overland flow (which by its nature includes shallow flow) often involves determining at which point it becomes significant enough to classify as "flooding". The difference in peak flood level between events of varying magnitude may be minor in areas of overland flow, such that applying the typical freeboard can result in a FPL greater than the Probable Maximum Flood (PMF) level.

The FPA should include properties where future development would result in impacts on flood behaviour in the surrounding area and areas of high hazard that pose a risk to safety or life. Further to this, the FPL is determined with the purpose to decrease the likelihood of over-floor flooding of buildings and the associated damages.

The Floodplain Development Manual suggests that the FPL generally be based on the 100 year ARI event plus an appropriate freeboard. The typical freeboard cited in the manual is that of 0.5 m; however it also recognises that different freeboards may be deemed more appropriate due to local conditions. In these circumstances, some justification is called for where a lower value is adopted.

Further consideration of flood planning areas and levels are typically undertaken as part of the Floodplain Management Study where council decides which approach to adopt for inclusion in their Floodplain Management Plan.

9.2. Methodology

The methodology used in this report is consistent with that adopted in a number of previous studies. It divides flooding between Mainstream flooding and Overland flooding using the following criteria:

- Mainstream flooding: Any percentage of the cadastral area is affected by mainstream flooding in the 100 year ARI event. This has been defined as the peak flood level within the open channel section of Hawthorne Canal plus a 0.5 m freeboard, with the level extended perpendicular to the flow direction.

- Overland flooding: Greater than or equal to 10% of the “active” cadastral area is affected by the 100 year ARI peak flood depth of greater than 0.15 m. The “active” cadastral area was considered to be the cadastral area excluding the building area that was modelled as impermeable

In situations where a cadastral lot is subject to both mainstream flooding and overland flooding, the mechanism that produces the highest Flood Planning Level is given precedence, although both levels have been provided.

9.3. Results

A summary of properties tagged is provided in Table 30. Figure 38 identifies the extent of mainstream or overland flow property affectation.

Table 30: Number of Properties Tagged

	Mainstream	Overland	Both Mainstream and Overland	Total
Ashfield	60	386	161	607
Marrickville	33	304	92	429
Total	93	690	253	1036

A total of 607 properties were tagged for flood related development controls in Ashfield and 429 properties in Marrickville. This gives very similar averages of 2.0 properties per hectare for Ashfield and 2.1 properties per hectare for Marrickville.

Properties that are not tagged as part of this process may not be excluded from development controls. It is advisable that new developments (regardless of whether they are tagged as flood liable or not) have habitable floor levels a minimum of 300 mm above the surrounding ground level to minimise affectation due to local overland flow.

10. DISCUSSION

Various locations were identified as "hotspots" or "areas of interest" within the Hawthorne Canal Catchment, shown in Figure D 1 to Figure D 9. These locations were identified based upon flood behaviour occurring at ground level. The above floor flood liability of these locations has not yet been determined due to a lack of surveyed floor levels at this stage. However, some over floor flood liability is likely at each of these locations.

10.1. Hotspots

The following discussion examines areas identified herein as "hotspots" within the Hawthorne Canal Catchment. The locations were identified based upon areas defined in the hydraulic model as being subject to significant levels of flooding.

10.1.1. Lewisham

The Lewisham area has a number of significant structures through road and railway embankments that intersect substantial flow-paths. Due to the relatively close proximity of these structures, the flooding behaviour in the vicinity of these structures could be considered inter-related with changes to one or more likely to result in significant impacts in the vicinity of the remaining structures.

The Longport Street road embankment is located at the convergence of three significant flow-paths originating from the south by way of the open channel, the west by way of overland and SWC trunk drainage flow along Smith Street, and from the east by way of the Lewisham drainage branch. The flow from the west is within the Ashfield Council LGA and the flow from the east is within the Marrickville Council LGA. The boundary between the two councils is located along and bisects the open channel transporting flows from the south.

The Old Canterbury Road embankment intersects the main open channel south of (and upstream of) the Longport Street embankment. Adjacent to the open channel upstream of this embankment are residential properties located along Fred Street.

Interchanging along the open channel is the light rail line that bisects the Hawthorne catchment in a north-south direction. It traverses the open channel upstream of the Old Canterbury Road embankment and again between the Longport Street embankment and the Old Canterbury Road embankment. The railway track has a lower elevation than either of the two road embankments. So although it acts as an obstruction when intersecting the open channel, the railway track also acts as an alternative flow-path through the road embankments when aligned parallel to the open channel.

Flooding Behaviour

The contributing catchment area upstream of the Longport Street embankment is approximately 277 ha, of which 180 ha is located on the southern branch upstream of the Old Canterbury Road embankment, 67 ha is located within the western branch conveyed via Smith Street, 19 ha is located in the eastern branch and the remaining area is located between the Longport Street embankment and the Old Canterbury Road embankment. The peak flows and peak flood levels/depths in the vicinity of this hotspot are shown in Table 31 and Table 32.

Flow originating from the south along the open channel intercepts the light rail embankment in the vicinity of Eltham Street, located upstream of the Old Canterbury Road embankment. The elevation of the light rail embankment at this location is approximately 14.3 m AHD and the invert of the open channel upstream of the embankment is 9.33 m AHD. As such, when the capacity of the 2.28 m diameter pipe is exceeded, flow accumulates upstream of the embankment until the accumulated flood water level exceeds the embankment elevation and overtopping occurs. In events greater than and equal in magnitude to a 5 year ARI event, the pipe was found to be functioning at capacity and the embankment was overtopped. Flow that overtops the light rail embankment in the vicinity of Eltham Street, rejoins the open channel flow downstream of the embankment.

The Old Canterbury Road embankment has two modes of conveyance through it. The first is through a culvert attached to the open channel network, with a cross sectional area of approximately 4.5 m^2. The second is through the railway underpass parallel to and located to the west of the open channel.

Upstream of the Old Canterbury Road embankment, the culvert has an invert elevation of 7.8 m AHD and the railway underpass has an elevation of approximately 12.8 m AHD. Therefore, flow that cannot be immediately conveyed through the culvert does not redirect through the light rail underpass until the flood waters accumulating upstream of the embankment exceed 5 m in depth within the open channel.

In relatively small events, such as the 2 year and 5 year ARI events, the flood level in the open channel does not exceed the light rail embankment elevation. In these events, the overland flow originating from the west along Old Canterbury Road conveys a marginal amount of flow through the light rail underpass. The remainder of this overland flow continues east from the light rail to merge with the open channel flow upstream of the Old Canterbury Road culvert. This results in different flood levels between the open channel and light rail tracks upstream of the Old Canterbury Road embankment in smaller events.

In events larger than and equal in magnitude to the 10 year ARI event, relatively uniform levels are observed within the open channel and on the light rail tracks upstream of the Old Canterbury Road embankment. In these events, the culvert connected to the open channel is functioning at capacity, which is not the case in the 2 year and 5 year ARI events. The flow that cannot be conveyed via the culvert and light rail underpass accumulate upstream, extending into properties to the east (along Fred Street) and to the west of the light rail tracks.

In the PMF event, an additional flow-path across Old Canterbury Road is observed at the intersection with Summer Hill Street, located to the east of the open channel. The flood level upstream of Old Canterbury Road must exceed approximately 14.9 m AHD (the elevation of the roadway at the low point) for this alternative flow-path to function.

The light rail embankment between Old Canterbury Road and Longport Street has an approximate elevation of 11.3 m AHD and the upstream invert of the culvert underneath the embankment was 5.49 m AHD. In events less than and equal in magnitude to a 10 year ARI event, the embankment is not overtopped at this location. In the 20 year ARI event, and those of a greater magnitude, the light rail embankment is overtopped with some flow joining the open channel downstream of the embankment and some flow occurring along the light rail tracks from Old Canterbury Road to Longport Street.

The Smith Street branch converges with the open channel upstream of the Longport Street embankment and downstream of the light rail embankment. The pipe discharging into the open channel is an oviform, with a width of 1.675 m and height of 1.37 m. Upstream of Edward Street, this pipe is functioning at capacity in a 2 year ARI event. The largest peak flood depth along Smith Street was found to occur at the intersection of Smith Street and Edward Street.

The pipe draining the Lewisham branch to the east discharges into the open channel downstream of the Longport Street embankment. Where the pipe traverses Old Canterbury Road near Henry Street it has a rectangular cross-section with a width of 1 m and a height of 0.9 m. The pipe was found to be functioning at capacity in a 2 year ARI event. Flow that is not drained by the pipe system is conveyed via overland flow toward the area upstream of the Longport Street embankment.

The Longport Street embankment has two modes of conveyance through it. The primary mode of conveyance is through a culvert attached to the open channel network, with a cross-sectional area of approximately 11.6 m^2. The secondary mode of conveyance is through the light rail underpass parallel to and located to the east of the open channel.

Upstream of the Longport Street embankment, the culvert has an invert elevation of 2.37 m AHD and the light rail track has an elevation of approximately 9.5 m AHD. Therefore, flow that cannot be immediately conveyed through the culvert does not redirect through the light rail underpass until the flood waters accumulating upstream of the embankment exceed 7.13 m in depth within the open channel.

In events smaller than and equal in magnitude to a 100 year ARI event, the accumulated flood water in the open channel upstream of the Longport Street embankment does not exceed the elevation of the light rail track. This results in significantly different flood levels between the two flow-paths through the Longport Street embankment, up to 4 m difference in the 2 year ARI event. The flow through the light rail underpass originates from flows occurring along the light rail track and the overland flow-path from the Lewisham branch to the east. The flow through the culvert originates from the open channel, the flow from the Smith Street branch to the west and run-off from the light rail tracks. The run-off from the light rail tracks occurs at two separate locations; adjoining to the Longport Street embankment and directly adjacent to the open channel where it intercepts the light rail embankment.

In the PMF event, uniform peak flood levels of 12.7 m AHD were observed within the open channel and on the light rail tracks upstream of the Longport Street embankment. The backwater effects of this constriction extended up to the intersection of Smith Street with Edward Street and up to Old Canterbury Road, where it intersects the open channel from the south and the Lewisham overland flow-path from the east.

Downstream of the Longport Street embankment, the open channel traverses under the Western Railway Line embankment. The railway embankment itself does not obstruct flows with an underside elevation of 16.76 m AHD, placing it 15.09 m above the invert of the open channel at this location.

Table 31: Lewisham – Peak Flows (m^3/s)

Location	**Type**	**2 yr ARI**	**5 yr ARI**	**10 yr ARI**	**20 yr ARI**	**50 yr ARI**	**100 yr ARI**	**PMF**
Light Rail Embankment (Adjacent to Eltham St)	Pipe	15.7	19.5	19.5	19.1	18.8	18.5	27.2
Old Canterbury Rd Embankment (Along Open Channel)	Open Channel Culvert	16.3	22.7	25.2	26.4	26.6	26.6	27.6
	Light Rail Underpass	0.1	0.1	1.6	8.0	14.9	22.5	120.4
	Overland (via Old Canterbury Rd & Summer Hill St)	0.0	0.0	0.0	0.0	0.0	0.0	44.3
Light Rail Embankment (Between Old Canterbury Rd and Longport St)	Pipe	16.3	22.3	25.6	27.4	27.3	27.3	25.6
Longport St Embankment	Open Channel Culvert	23.9	33.0	37.5	44.8	51.5	60.8	98.2
	Light Rail Underpass	0.0	0.4	0.8	1.3	1.8	2.2	110.7
Western Railway Line Embankment	Open Channel	25.4	34.2	39.1	44.4	51.9	60.9	205.0
	Overbank adjacent to open channel	1.2	2.0	2.1	2.9	3.7	4.4	54.8
Eastern Branch traversing Old Canterbury Rd	Overland	1.4	2.8	3.8	5.1	6.1	7.4	32.0
	Pipe	1.9	1.9	1.9	1.9	1.9	1.9	2.2
Western Branch via Smith St	Overland	3.9	7.7	9.9	13.2	16.3	19.4	87.8
	Pipe	4.1	4.1	4.1	4.1	4.1	4.1	4.0

Table 32: Lewisham – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Light Rail Line (Adjacent to Eltham St)								
Open Channel Upstream	Level	13.88	14.83	14.95	15.05	15.12	15.18	16.25
	Depth	4.25	5.20	5.33	5.42	5.49	5.56	6.63
On Embankment	Level	0.00	14.60	14.73	14.82	14.91	14.99	16.20
	Depth	0.00	0.29	0.42	0.52	0.60	0.68	1.90
Upstream of Old Canterbury Rd								
Open Channel	Level	10.96	12.30	13.04	13.59	13.94	14.22	16.16
	Depth	2.58	3.92	4.66	5.21	5.56	5.84	7.78
Light Rail Line	Level	12.90	12.91	13.03	13.57	13.91	14.20	16.16
	Depth	0.03	0.04	0.16	0.70	1.04	1.33	3.29
Light Rail Line (Between Longport St and Old Canterbury Rd)								
Open Channel Upstream	Level	8.26	9.46	10.47	11.36	11.61	11.76	13.96
	Depth	2.30	3.50	4.51	5.41	5.66	5.80	8.01
On Embankment	Level	0.00	0.00	0.00	0.00	11.52	11.59	13.95
	Depth	0.00	0.00	0.00	0.00	0.09	0.13	2.46
Upstream of Longport St								
Open Channel	Level	5.48	6.05	6.43	6.87	7.76	8.27	13.93
	Depth	2.92	3.49	3.87	4.31	5.19	5.71	11.37
Light Rail Line	Level	9.60	9.80	9.87	9.93	9.99	10.04	13.93
	Depth	0.23	0.44	0.50	0.57	0.62	0.67	4.56
Eastern Branch								
Old Canterbury Rd (Adjacent to Henry St)	Level	12.12	12.17	12.20	12.23	12.26	12.29	13.93
	Depth	0.05	0.10	0.12	0.16	0.19	0.21	1.86
Western Branch								
Intersection of Smith St and Edward St	Level	10.84	10.96	11.06	11.16	11.24	11.31	13.95
	Depth	0.05	0.17	0.27	0.37	0.45	0.52	3.16

A significant degree of afflux was found to occur where major structures traversed the open channel. The largest afflux occurred at the Old Canterbury Road embankment and Longport Street embankment. No afflux was observed where the Western Railway Line traverses the open channel (downstream of Longport Street) or at the Parramatta Road Bridge, due to the undersides of both bridges being higher than the PMF peak flood level.

Table 33: Lewisham – Afflux – Peak Flood Levels (m AHD)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Light Rail Embankment – Adjacent to Eltham St	Upstream	13.88	14.83	14.95	15.05	15.12	15.18	16.25
	Downstream	11.05	12.32	13.05	13.59	13.94	14.23	16.16
	Afflux (m)	2.82	2.50	1.90	1.45	1.18	0.96	0.10
Old Canterbury Rd Embankment	Upstream	10.96	12.30	13.04	13.59	13.94	14.22	16.16
	Downstream	9.97	10.28	10.60	11.40	11.64	11.78	13.95
	Afflux (m)	0.98	2.02	2.44	2.19	2.29	2.44	2.21
Longport St Embankment	Upstream	5.48	6.05	6.43	6.87	7.76	8.27	13.93
	Downstream	5.23	5.43	5.51	5.61	5.75	5.91	8.31
	Afflux (m)	0.25	0.63	0.92	1.26	2.01	2.36	5.62

10.1.2. Grosvenor Crescent, Summer Hill

Grosvenor Crescent is located to the north of the Western Railway Line that bisects the Hawthorne Canal catchment in an east-west direction. Between Liverpool Street and Summer Hill train station, the railway is situated on an embankment with the Grosvenor Crescent roadway and surrounding area forming a topographical low point. At this location the embankment has an elevation of approximately 28 m AHD and the roadway has an elevation of approximately 25.2 m AHD.

To the west of this low point, both the roadway and the railway line rise in elevation. The roadway has a steeper grade (at approximately 3.4%) compared to the railway line grade (at less than 1%), leading up to the Liverpool Street bridge over the railway line.

To the east of the Grosvenor Crescent low point, the roadway increases in elevation while the railway line decreases in elevation, such that in the vicinity of Summer Hill Train Station the railway is comparative level with the Grosvenor Crescent roadway. The roadway to the east has a smaller grade (at approximately 1%) compared to the roadway grade to the west.

Flooding Behaviour

The contributing catchment area is approximately 6.4 ha. A 0.55 m diameter pipe conveys flow underneath the railway embankment to Carlton Crescent. This pipe was found to be functioning at capacity in events greater than and equal in magnitude to a 2 year ARI event.

Flows that cannot be conveyed via the pit and pipe system accumulate along Grosvenor Crescent. Given the grade to the east is shallower than to the west, accumulation outside this low point tends to the east. In events smaller than and equal in magnitude to a 100 year ARI event, this flow is contained in the low point until the pit and pipe system can convey the excess flow.

In the PMF event, the east bound accumulation of water reaches the point at which the roadway is comparatively level with the railway line. As such, this flow traverses the railway line in the vicinity of Summer Hill Train Station. This flow is in conjunction with those originating from the junction of Sloane Street and Grosvenor Crescent, such that the flow reported (in the table below) across the railway line is not independent.

Table 34: Grosvenor Crescent – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Grosvenor Crescent	Overland	0.0	0.0	0.0	0.0	0.0	0.1	9.1
	Pipe	0.6	0.6	0.6	0.7	0.7	0.7	0.8

The peak flood levels and depths on Grosvenor Crescent are provided in Table 35. Blockage of the pipes underneath the railway resulted in increases to the peak flood level of approximately 0.12 m in the case of 50% blockage.

Table 35: Grosvenor Crescent – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Grosvenor Crescent	Level	25.96	26.25	26.39	26.56	26.72	26.84	27.31
	Depth	0.71	0.99	1.13	1.30	1.46	1.58	2.06

10.1.3. West Street, Petersham

The intersection of West Street, Flood Street and Parramatta Road form a topographical low point, with each of the four roadways rising in elevation leading away from this intersection. The natural overland flow-path draining this location occurs through the properties located to the north-west of the intersection. However, the buildings constructed on these properties create an obstruction to overland flow.

Of these roadways, Parramatta Road is a major road bisecting the Hawthorne Canal catchment in an east-west direction, West Street is a secondary road leading away from the intersection to the south and Flood Street is a minor road leading away from the intersection to the north.

Flooding Behaviour

The contributing catchment area is approximately 60.5 ha, including the 9.2 ha that contribute to the Trafalgar Street hotspot (discussed in Section 10.1.4). The pipe draining this area has a diameter of 1.2 m and was found to be operating at capacity in the 2 year ARI event.

Table 36: West Street – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Parramatta Rd	Overland	3.6	6.8	8.8	11.7	14.3	17.1	71.6
	Pipe	4.1	4.2	4.2	4.2	4.2	4.2	4.3

The peak flood levels and depths on the intersection of West Street, Flood Street and Parramatta Road are provided in Table 37. This location was found to be relatively insensitive to blockage of the trunk drainage pipes, with peak flood levels increasing by 0.04 m in the case of 50% blockage.

Table 37: West Street – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Parramatta Rd	Level	11.58	11.71	11.78	11.86	11.92	11.98	12.94
	Depth	0.50	0.64	0.70	0.78	0.85	0.90	1.86

10.1.4. Trafalgar Street, Petersham

The intersection of Trafalgar Street and Nelson Place is a trapped low point where the Western Railway Line embankment and RailCorp Training Centre intersects an overland flow-path originating from the south. The RailCorp grounds have the higher elevation of approximately 29 m AHD, compared to Trafalgar Street to the south (with an approximate elevation of 26.5 m AHD) and the railway tracks to the north (with an approximate elevation of 27.7 m AHD).

The corner of Railway Terrace and Gordon Street form a low point to the west of the Trafalgar Street low point. Flows to this area are impeded, although not completely restricted, from entering the railway tracks via a brick wall. The roadway at this location is higher in elevation than the railway tracks.

Of these roadways, Railway Terrace is a main road and Trafalgar Street is a secondary road.

Flooding Behaviour

The contributing catchment area is approximately 9.2 ha. Flow is conveyed across the railway via a 0.75 m diameter pipe from Trafalgar Street and a 0.45 m diameter pipe from Railway Terrace. The capacity of these pipes was found to be less than a 2 year ARI event.

Flows that cannot be conveyed via the pit and pipe system accumulate at both of these locations. In events less than and including the 100 year ARI event, water is contained at the low point of Trafalgar Street and does not overtop the embankment of the RailCorp Training Centre. In the PMF event, flood levels from Trafalgar Street exceed the railway embankment and flow across the RailCorp Training Centre grounds as well as extend west to join the Railway Terrace flows. These flows are summarised in Table 38.

Table 38: Trafalgar Street – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Trafalgar St	Overland via Railway	0.0	0.0	0.0	0.0	0.0	0.0	4.9
	Pipe	1.0	1.1	1.2	1.3	1.3	1.3	1.3
Railway Terrace	Overland via Railway	0.1	0.2	0.3	0.5	0.6	0.7	4.8
	Pipe	0.3	0.4	0.4	0.4	0.4	0.4	0.4
Trafalgar St to Railway Terrace	Overland	0.0	0.0	0.0	0.0	0.0	0.0	2.7

The peak flood levels and depths on Trafalgar Street and Railway Terrace are provided in Table 39. Flood levels on Trafalgar Street were found to be more sensitive to blockage of pipes than Railway Terrace, with increases to peak flood levels (in the case of 50% blockage) of 0.34 m and 0.01 m respectively.

Table 39: Trafalgar Street – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Trafalgar St	Level	27.03	27.31	27.47	27.72	27.93	28.10	28.96
	Depth	0.55	0.83	0.99	1.24	1.45	1.62	2.48
Railway Terrace	Level	28.03	28.04	28.05	28.07	28.08	28.09	28.37
	Depth	0.38	0.40	0.41	0.42	0.43	0.45	0.72

10.2. Areas of Interest

Additional areas of interest were identified by council, in some cases based upon flooding concerns raised by residents prior to commencement of this flood study.

10.2.1. Light Rail Track

The proposed light rail extension line bisects the Hawthorne Canal catchment in a north-south orientation. To the north of the City Rail Western Railway Line it forms an embankment. The embankment is parallel to the open channel that is located to the west of it. To the south of Hill Street it is lower in elevation than the surrounding ground and forms a primary overland flow-path. Connecting these locations, the light rail alternates several times between functioning as a flow-path and forming an embankment.

The light rail has been discussed in the previous hotspots where it interacts with other infrastructure. In this section, the light rail as a hydraulic feature in itself is discussed.

Flooding Behaviour

Two SWC trunk drainage lines cross the embankment. The trunk drainage conveys water from Marion Street and Beeson Road to the open channel. Marrickville Council pipes drain the Brown Street area to the open channel and Leichhardt Council pipes drain the area to the north of Marion Street.

Roadway and pedestrian underpasses convey additional flow across the embankment. Roadway underpasses were located on Charles Street, Marion Street and Parramatta Road, and pedestrian underpasses were located on Darley Road and Lords Road.

Flood waters that cannot be immediately conveyed via the drainage system and underpasses accumulate to the east of the light rail embankment. In events greater than and equal in magnitude to a 20 year ARI event, the accumulated flood water exceeds the height of the embankment and overtopping occurs along Darley Road between Allen Street and William Street. Table 40 provides the flows through the embankment at these locations as well as flows to the south of Hill Street where the light rail acts as a floodway.

Table 40: Light Rail Track – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Charles St (Roadway Underpass	Overland	0.2	0.2	0.2	0.2	0.2	0.2	13.9
Darley Rd (Overtopping)	Overland	0.0	0.0	0.0	0.5	4.9	10.3	117.0
Darley Rd (Pedestrian Underpass)	Overland	3.1	4.8	5.9	7.2	8.3	8.9	21.7
Marion St (Roadway Underpass)	Overland	4.5	7.1	8.6	10.3	12.1	14.1	52.7
Lords Rd (Pedestrian Underpass)	Overland	2.5	5.7	7.2	9.6	12.2	14.6	43.8
Parramatta Rd (Roadway Underpass)	Overland	0.4	0.9	1.2	1.7	2.1	2.6	11.9
Constitution Rd (Flow-Path)	Overland	3.6	7.3	9.4	12.5	15.6	18.9	80.1

10.2.2. Ashfield / Dulwich Hill

Several instances of flooding were reported along Queen Street via community consultation.

The overland flow-path from Queen Street to Yeo Park is orientated perpendicular to the roadway alignment. From Old Canterbury Road to Dixson Avenue, the overland flow-path is parallel to the roadway alignment of Cobra Street and Elizabeth Avenue. This flow-path occurs along the boundary of properties located on the two roadways. Between Dixson Avenue and Arlington Recreation Reserve, the overland flow-path is again orientated perpendicular to the roadway.

Old Canterbury Road forms the LGA boundary between Ashfield Council and Marrickville Council. The SWC trunk drainage system extends to the east from Old Canterbury Road with Marrickville Council pipes connected to it. To the west of Old Canterbury Road, Ashfield Council pipes service the primary flow-path before discharging into the SWC trunk drainage system.

Flooding Behaviour

The contributing catchment area upstream of Queen Street is approximately 10 ha, which is drained via a 0.6 m diameter pipe. Two 0.6 m diameter pipes drain Service Avenue and Victoria Street is drained via a 0.6 m and a 0.75 m diameter pipe.

The contributing catchment area upstream of Old Canterbury Road is approximately 34 ha and is drained via a 1.05 m diameter pipe. From Dixson Avenue, a 1.35 m diameter pipe is in operation.

The pipe system from Queen Street along to Dixson Avenue was found to be functioning at capacity in the 2 year ARI event.

Table 41: Ashfield / Dulwich Hill – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Queen St	Overland	1.4	2.2	2.7	3.3	3.8	4.5	16.3
	Pipe	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Service Av	Overland	2.7	4.2	5.2	6.4	7.4	8.6	30.9
	Pipe 1	0.4	0.4	0.4	0.5	0.5	0.5	0.5
	Pipe 2	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Victoria St	Overland	3.1	4.8	5.8	7.2	8.4	9.7	35.0
	Pipe 1	0.3	0.3	0.3	0.3	0.3	0.3	0.4
	Pipe 2	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Old Canterbury Rd	Overland	4.9	7.4	9.0	11.2	12.9	14.9	53.2
	Pipe	1.7	1.7	1.7	1.7	1.7	1.7	2.0
Dixson Ave	Overland	6.1	9.5	11.5	14.4	16.9	19.6	74.3
	Pipe	2.2	2.3	2.3	2.3	2.3	2.3	2.4

The peak flood levels provided in Table 42 were found to be relatively insensitive to blockage, increasing by 0.03 m in the case of 50% blockage.

Table 42: Ashfield / Dulwich Hill – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Queen St	Level	40.22	40.24	40.25	40.26	40.28	40.29	40.63
	Depth	0.04	0.06	0.07	0.08	0.10	0.11	0.45
Old Canterbury Rd	Level	34.55	34.61	34.65	34.69	34.73	34.76	35.20
	Depth	0.17	0.23	0.27	0.31	0.35	0.38	0.82
Dixson Ave	Level	29.69	29.81	29.87	29.95	30.01	30.07	30.79
	Depth	0.67	0.80	0.86	0.94	1.00	1.06	1.77

10.2.3. Petersham

The Petersham area of interest encompasses the area between the Trafalgar Street hotspot (upstream and to the south of this location) and the West Street hotspot (downstream and to the north of this location).

Petersham Park acts as an informal detention basin, with the Station Street driveway being the only provision for flow exiting this area. The mound constructed around the perimeter of the oval for spectator seating contains flows within the oval that cannot be immediately conveyed via the driveway.

The park is located at the convergence of three overland flow paths, originating from the south from Trafalgar Street to Brighton Street, the south-east from Railway Street to the intersection of Brighton Street and Station Street, and the east from Fort Street to Station Street.

Flooding Behaviour

The Trafalgar Street to Brighton Street branch is characterised by flow occurring perpendicular to the roadway alignment. It includes a minor low point on Searl Street. The Brighton Street low point extends from this branch (near the southern entrance to Petersham Park) up to Station Street.

Peak flood depths were relatively low at Brighton Street and Searl Street in events up to and including the 100 year ARI event due to the small contributing catchment area (approximately 12 ha upstream of Brighton Street) and the retention of flows upstream of the railway embankment (at the Trafalgar Street hotspot). These are summarised in Table 43. The peak flood levels at these locations were found to be relatively insensitive to blockage, increasing by 0.02 m in the case of 50% blockage.

Table 43: Petersham (Trafalgar St to Brighton St) – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Brighton St (West)	Level	17.42	17.45	17.47	17.50	17.52	17.55	17.96
	Depth	0.18	0.21	0.23	0.26	0.29	0.31	0.72
Searl St	Level	20.84	20.89	20.91	20.94	20.96	20.99	21.64
	Depth	0.22	0.27	0.29	0.32	0.34	0.37	1.02

The branch from Railway Street to the intersection of Brighton Street and Station Street consists of shallow overland flow from Railway Street via Brighton Street, Carrington Lane and Fishers Reserve. This shallow flow culminates at the Palace Street low point. Flow that cannot be conveyed by the 1.05 m (W) by 1.3 m (H) pipe draining this location is conveyed overland through the properties to the west of Palace Street. Downstream of the Palace Street low point is the Brighton Street (east) low point. The peak flood depths at these two low points are provided in Table 44. The peak flood levels at these locations were found to be relatively insensitive to blockage, increasing by 0.03 m in the case of 50% blockage.

Table 44: Petersham (Railway St to Brighton St) – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Brighton St (East)	Level	17.84	17.91	17.95	17.99	18.02	18.05	18.39
	Depth	0.22	0.29	0.33	0.37	0.40	0.43	0.77
Palace St	Level	21.46	21.54	21.58	21.62	21.64	21.67	22.03
	Depth	0.05	0.14	0.17	0.21	0.23	0.27	0.63

The Fort Street to Station Street branch consists of shallow overland flow. Within Fort Street High School is a detention basin that retards flow conveyed overland from Queen Street to the north. Flow along Andreas Street from the school grounds originate from areas not within the detention basin's catchment area as well as overflow from the detention basin when it reaches capacity. Additional to run-off from the school grounds, Andreas Street also receives flow from Palace Street to the east. From Andreas Street to Station Street, overland flow is conveyed through properties. The peak flood levels provided in Table 45 were found to be relatively insensitive to blockage at these locations, increasing by 0.01 m in the case of 50% blockage.

Table 45: Petersham (Fort St to Station St) – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Station St	Level	18.47	18.48	18.49	18.49	18.50	18.51	18.56
	Depth	0.01	0.02	0.02	0.02	0.03	0.04	0.09
Andreas St	Level	22.23	22.24	22.25	22.26	22.27	22.28	22.38
	Depth	0.00	0.00	0.00	0.00	0.00	0.01	0.10
Fort St High School Detention Basin	Level	25.80	25.82	25.83	25.85	25.86	25.88	26.02
	Depth	0.13	0.15	0.16	0.18	0.19	0.21	0.35

Petersham Park oval acts as a detention basin with inflow from the south and outflow through the Station Street driveway to the north. The peak flood levels and depths within the oval at the downstream border, adjacent to the Station Street driveway, are presented in Table 46.

Table 46: Petersham (Petersham Park) – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Petersham Park	Level	13.03	13.19	13.25	13.33	13.38	13.43	13.92
	Depth	0.34	0.49	0.56	0.63	0.69	0.74	1.23

10.2.4. Sloane Street, Ashfield

The intersection of Sloane Street and Parramatta Road is a localised topographical low point. Sloane Street rises in elevation in both directions leading away from the intersection, with a grade of between 2% and 3%. Parramatta Road has a steeper rise in elevation leading away from the intersection to the north-west, with a grade of approximately 4.5%. To the south-east of the intersection, Parramatta Road gently slopes away, with a grade of less than 1%. This downward slope occurs over a distance of approximately 25 m, after which the road rises in elevation again.

Flooding Behaviour

The contributing catchment area is approximately 19.3 ha. At this location, flows converge from two flow-paths originating from Sloane Street to the south-west and Parramatta Road to the north-west of the intersection.

The pipe draining this area has a diameter of 0.9 m and was found to be functioning at capacity in the 2 year ARI event. The water that cannot be conveyed via the pipe consequently flows along Parramatta Road to the south-east. Where Parramatta Road begins to rise to the south-east of the intersection, the only provision for the flow is through properties to the north of Parramatta Road. The flow entering and exiting this location are presented in Table 47.

Table 47: Sloane Street – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Western Inflow	Overland (Parramatta Rd)	0.5	0.8	1.0	1.2	1.5	1.9	9.2
	Overland (Sloane St)	1.6	2.3	2.6	3.2	3.7	4.3	15.2
	Pipe (Parramatta Rd)	0.4	0.6	0.6	0.7	0.7	0.7	0.8
	Pipe (Sloane St)	0.3	0.3	0.3	0.3	0.3	0.3	0.4
Eastern Outflow	Overland (Parramatta Rd)	1.3	2.5	3.2	4.2	5.1	6.0	20.4
	Overland (Sloane St)	0.4	0.4	0.6	0.9	1.2	1.5	7.3
	Pipe	1.8	1.8	1.8	1.8	1.9	1.9	1.9

The peak flood levels and depths at the intersection are provided in Table 48. The peak flood level at this location was found to be relatively insensitive to blockage, increasing by 0.03 m in the case of 50% blockage.

Table 48: Sloane Street – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Intersection of Sloane St and Parramatta Rd	Level	10.92	11.02	11.06	11.12	11.15	11.19	11.59
	Depth	0.25	0.35	0.40	0.45	0.49	0.52	0.92

10.2.5. Tressider Avenue, Haberfield

The primary flow-path through this location occurs along the south-west / north-east axis. Across Stanton Road, Ramsay Street and Tressider Avenue, this flow-path is orientated perpendicular to the roadway alignment.

Shallow overland flow from the north-west crosses O'Connor Street to merge with the primary flow-path. This flow is distributed along O'Connor Street from the junction with Ramsay Street to the junction with Deakin Avenue.

Flooding Behaviour

The contributing catchment area upstream of Tressider Avenue and along the south-west flow-path is approximately 297 ha. Along this drainage line, the pipe draining Stanton Road to Ramsay Street has a diameter of 0.6 m and the pipe draining Tressider Avenue, through Battalion Circuit, has a diameter of 0.9 m. Both pipes were found to be operating at capacity in the 2 year ARI event.

The contributing catchment area upstream of O'Connor Street is approximately 150 ha. The pipe conveying flow through Battalion Circuit from O'Connor Street has a 1.35 m diameter. This pipe was found to be operating at capacity in events greater than and equal in magnitude to the 20 year ARI event.

Table 49: Tressider Avenue – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Ramsay St	Overland	2.9	4.9	6.0	7.7	9.2	10.8	41.8
	Pipe	0.4	0.4	0.4	0.4	0.4	0.4	0.5
O'Connor St	Overland	1.8	2.9	3.6	4.6	5.4	6.4	23.6
Battalion Circuit	Overland	4.8	7.5	9.2	11.9	14.3	17.1	68.1
	Pipe (from Tressider Ave)	1.2	1.2	1.2	1.2	1.2	1.2	1.3
	Pipe (from O'Connor St)	1.4	1.6	1.7	1.8	1.9	1.9	2.1

The peak flood levels provided in Table 50 were found to be insensitive to blockage at this location, increasing by 0.01 m in the case of 50% blockage.

Table 50: Tressider Avenue – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Tressider Ave	Level	4.63	4.66	4.68	4.70	4.72	4.74	5.08
	Depth	0.14	0.17	0.18	0.21	0.23	0.25	0.58

11. PUBLIC EXHIBITION

Marrickville Council resolved to place the Draft Hawthorne Canal Flood Study on public exhibition at their December 2013 meeting. The flood study was placed on public exhibition during August and September 2014. No submissions were made in direct response to the Hawthorne Canal Flood Study during this exhibition process. The Hawthorne Canal Flood Study was subsequently adopted by Marrickville Council in February 2015.

12. ACKNOWLEDGEMENTS

WMAwater wish to acknowledge the assistance of Ashfield and Marrickville Council staff in carrying out this study as well as the NSW Government (Office of Environment and Heritage) and the residents of the Hawthorne Canal Catchment. This study was jointly funded by Ashfield and Marrickville councils, Sydney Water Corporation and the NSW Government.

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Figures

FIGURE 01

LOCALITY MAP

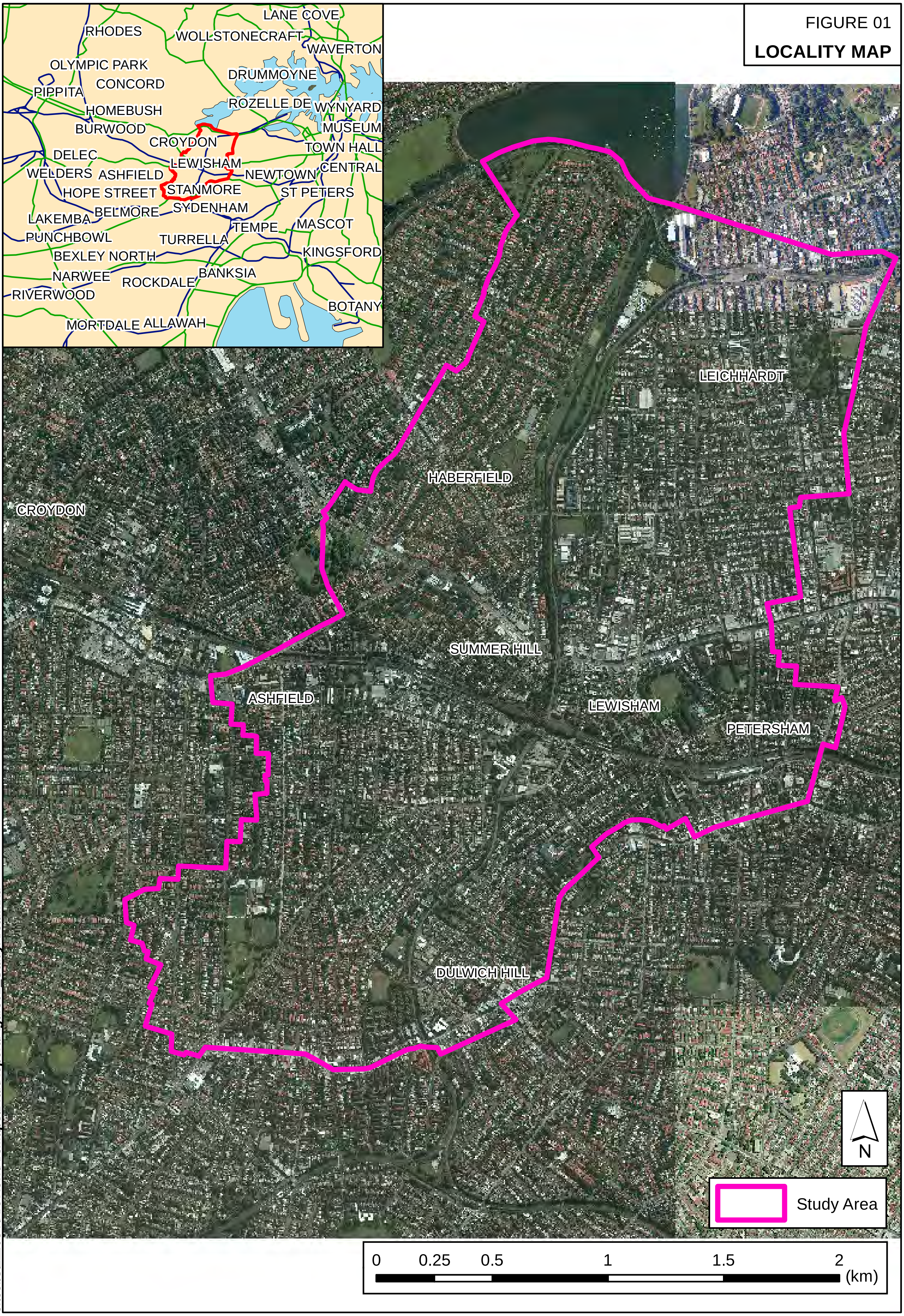

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FIGURE 2
STUDY AREA

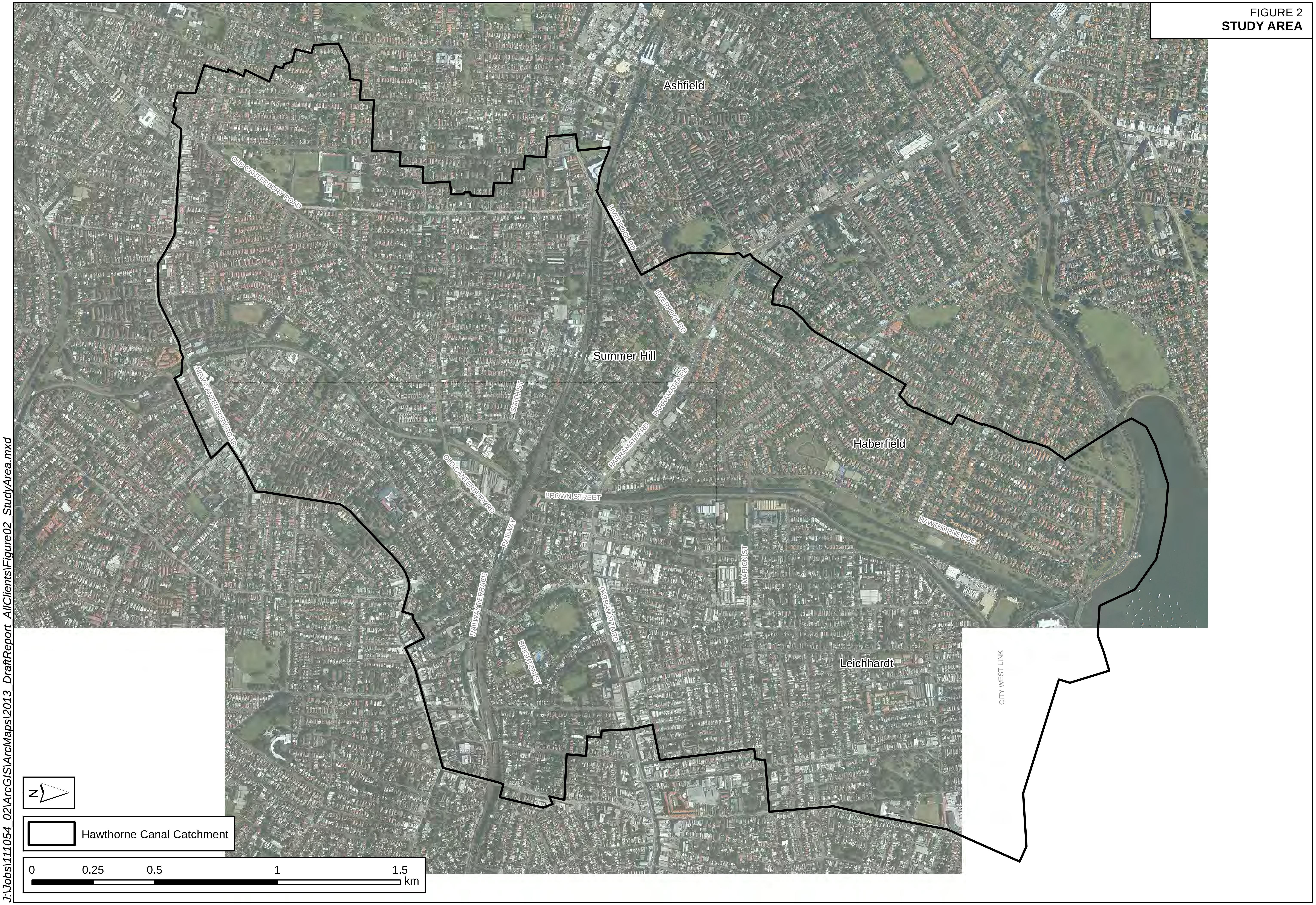

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FIGURE 3
LiDAR SURVEY DATA

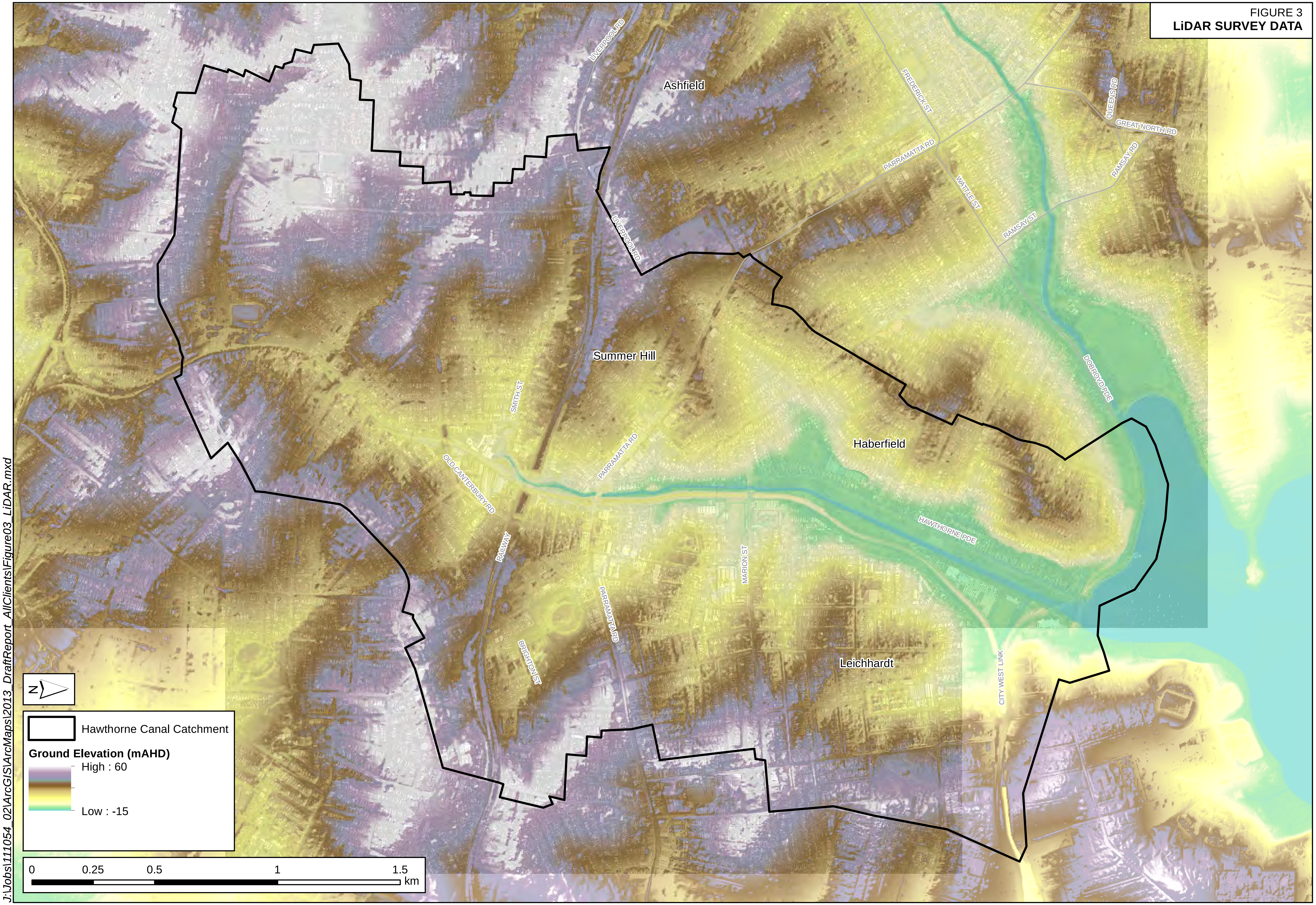

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FIGURE 4
CBH SURVEY DATA

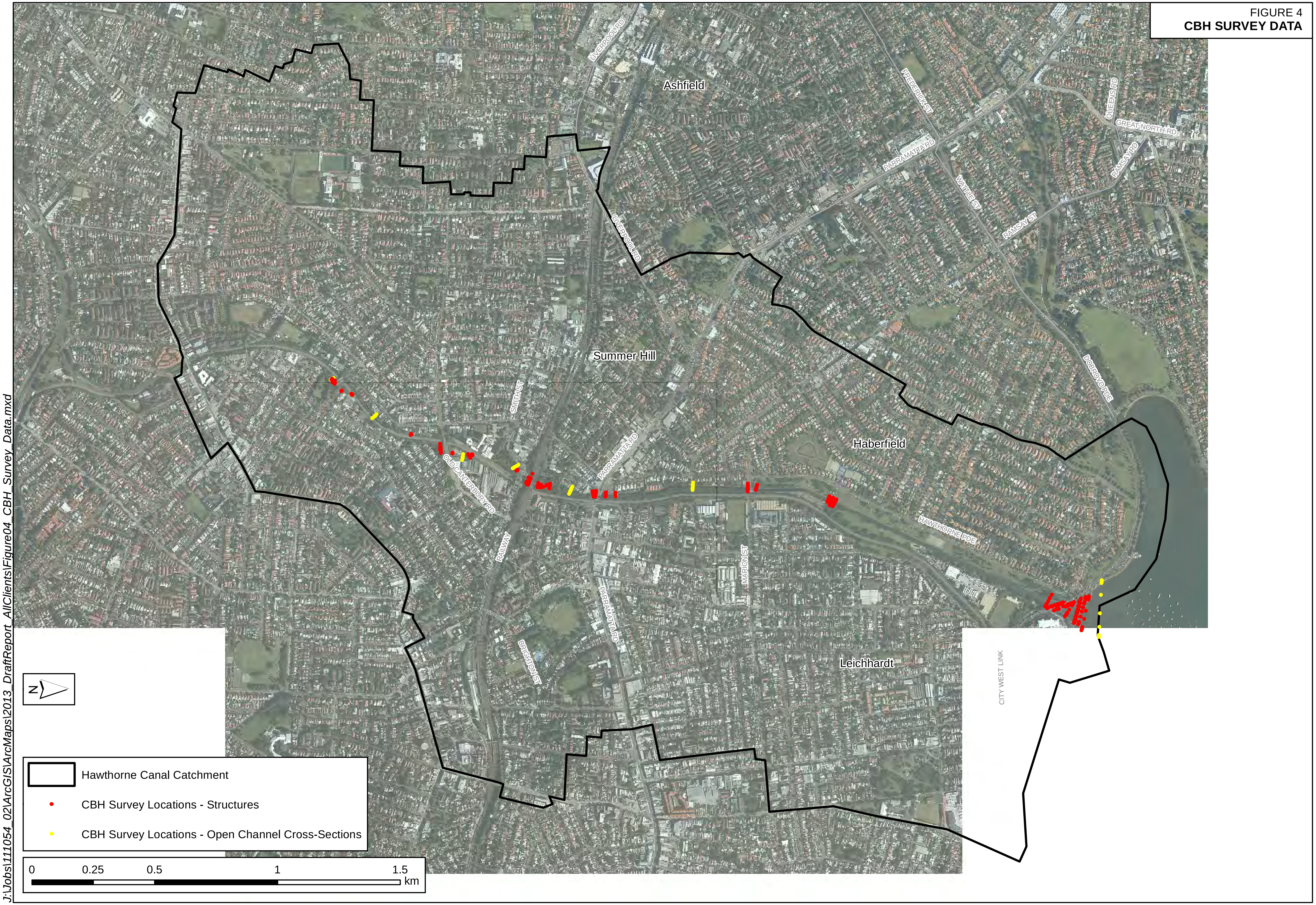

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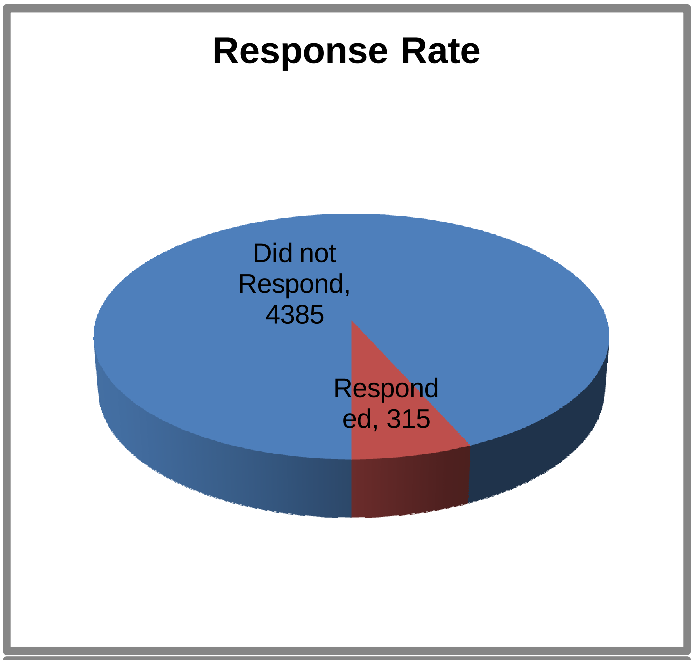
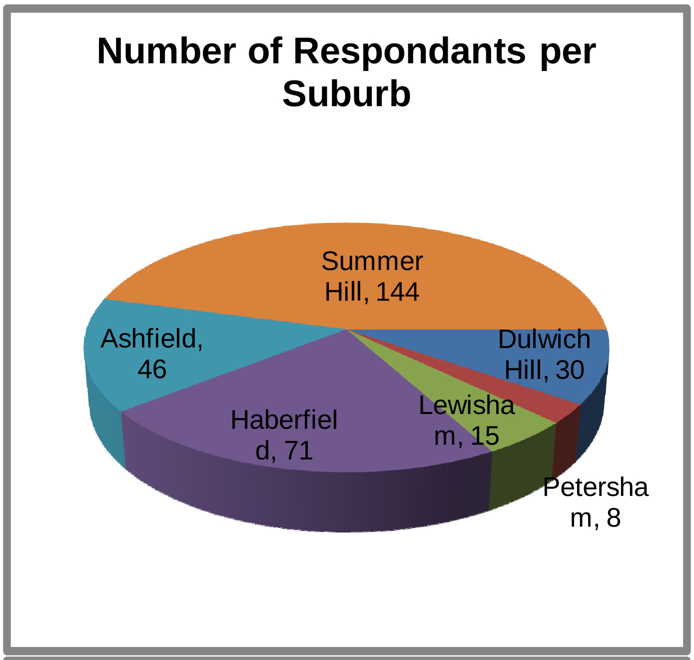
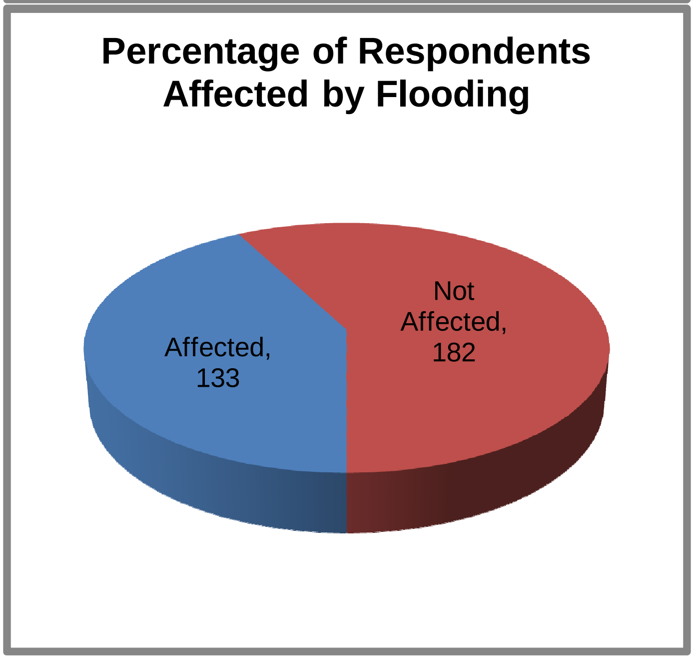
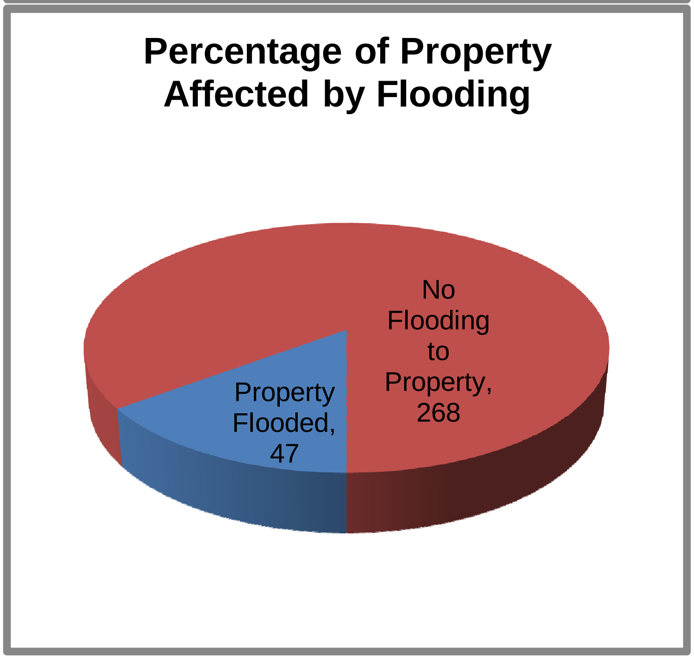
FIGURE 5
COMMUNITY CONSULTATION STATISTICS









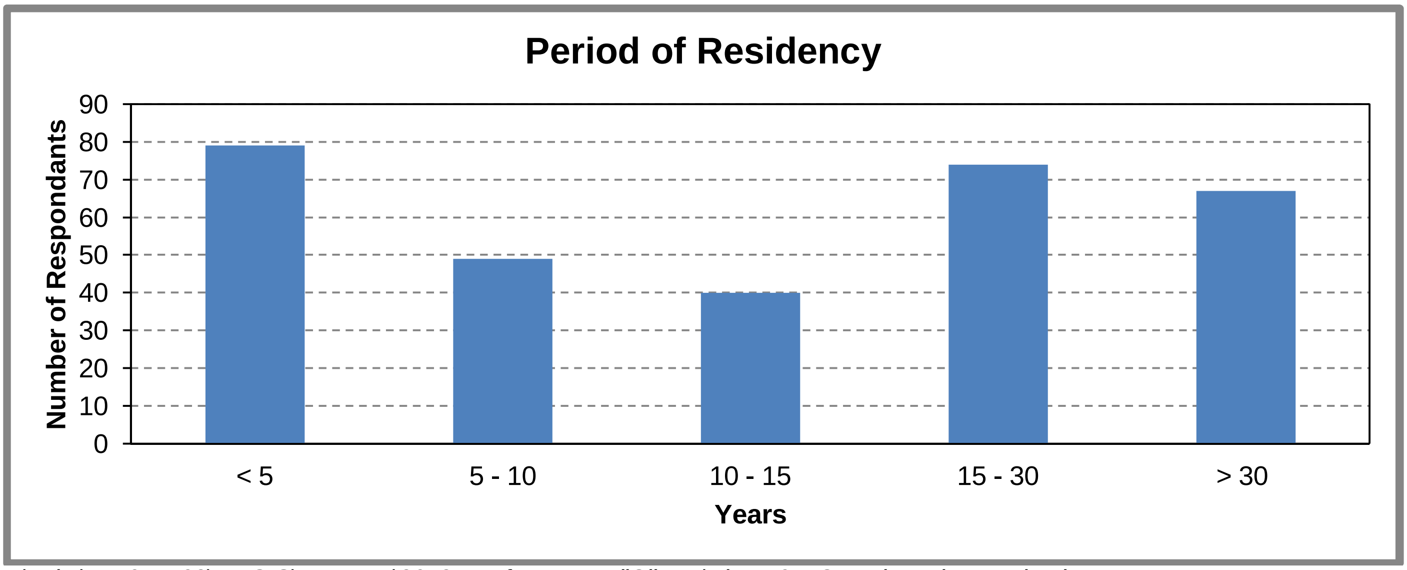


FIGURE 6
COMMUNITY CONSULTATION RESPONDANTS

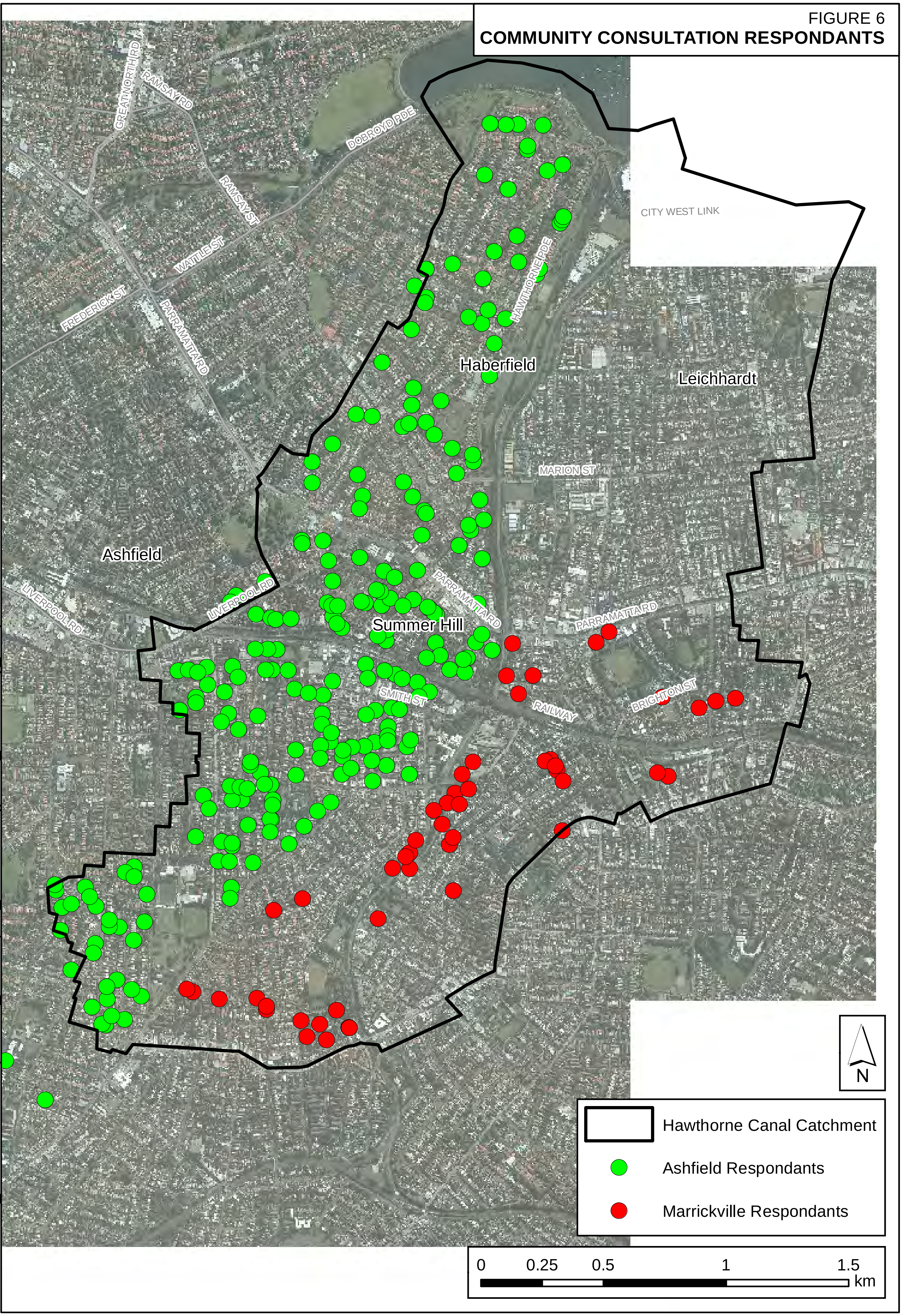


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FIGURE 7
HISTORIC FLOOD LEVEL LOCATIONS

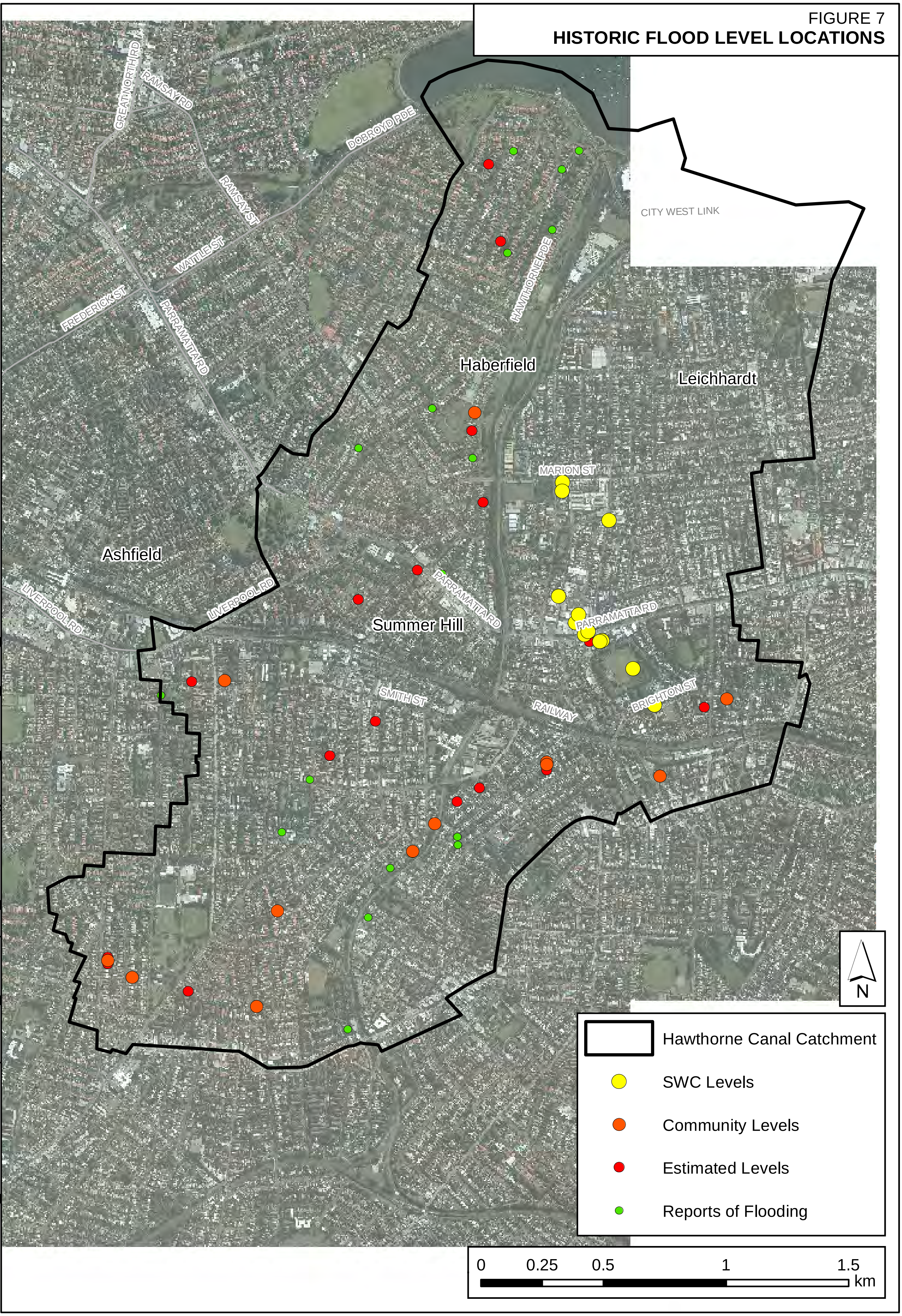

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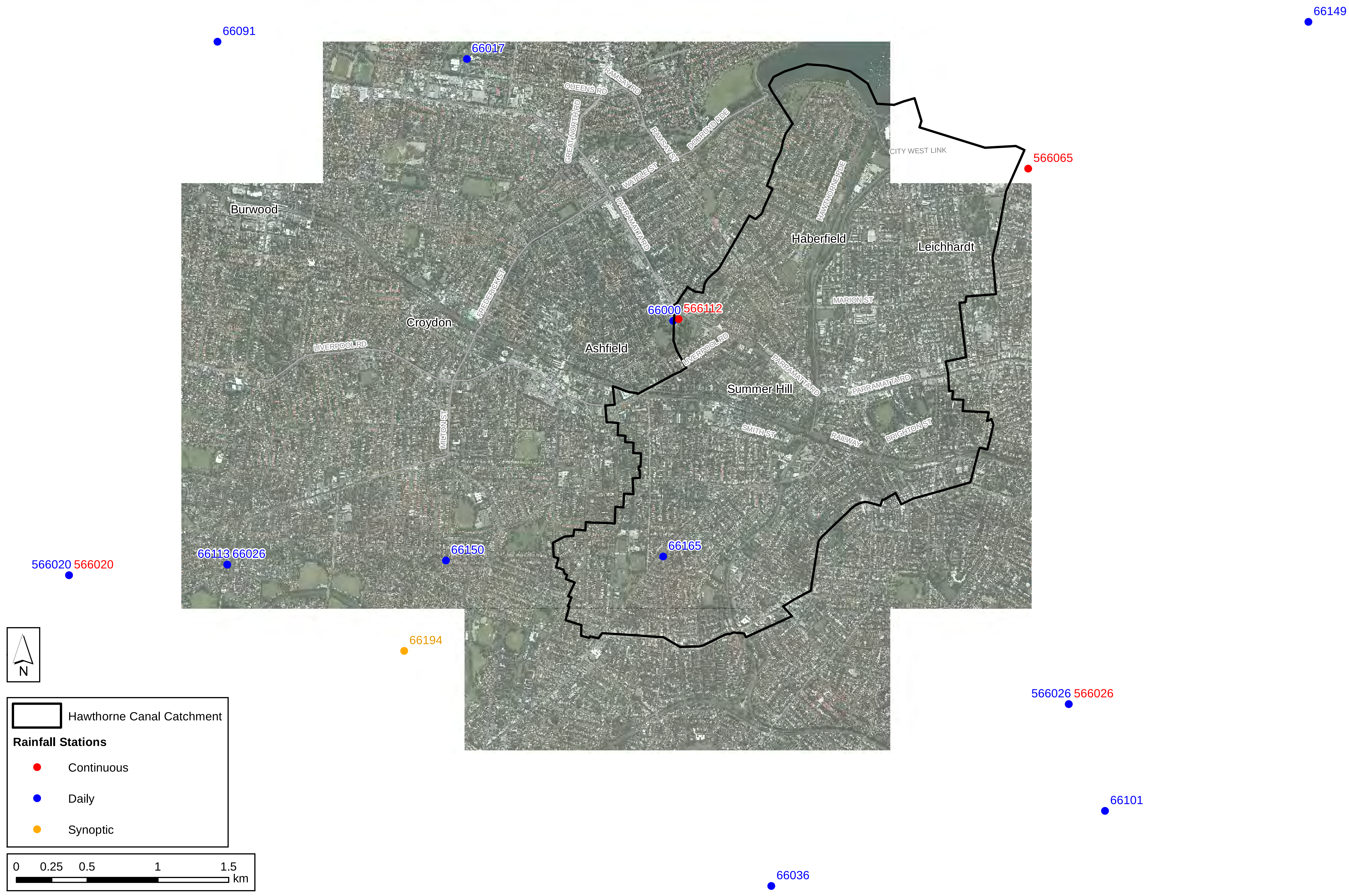
FIGURE 8
GAUGE LOCATIONS



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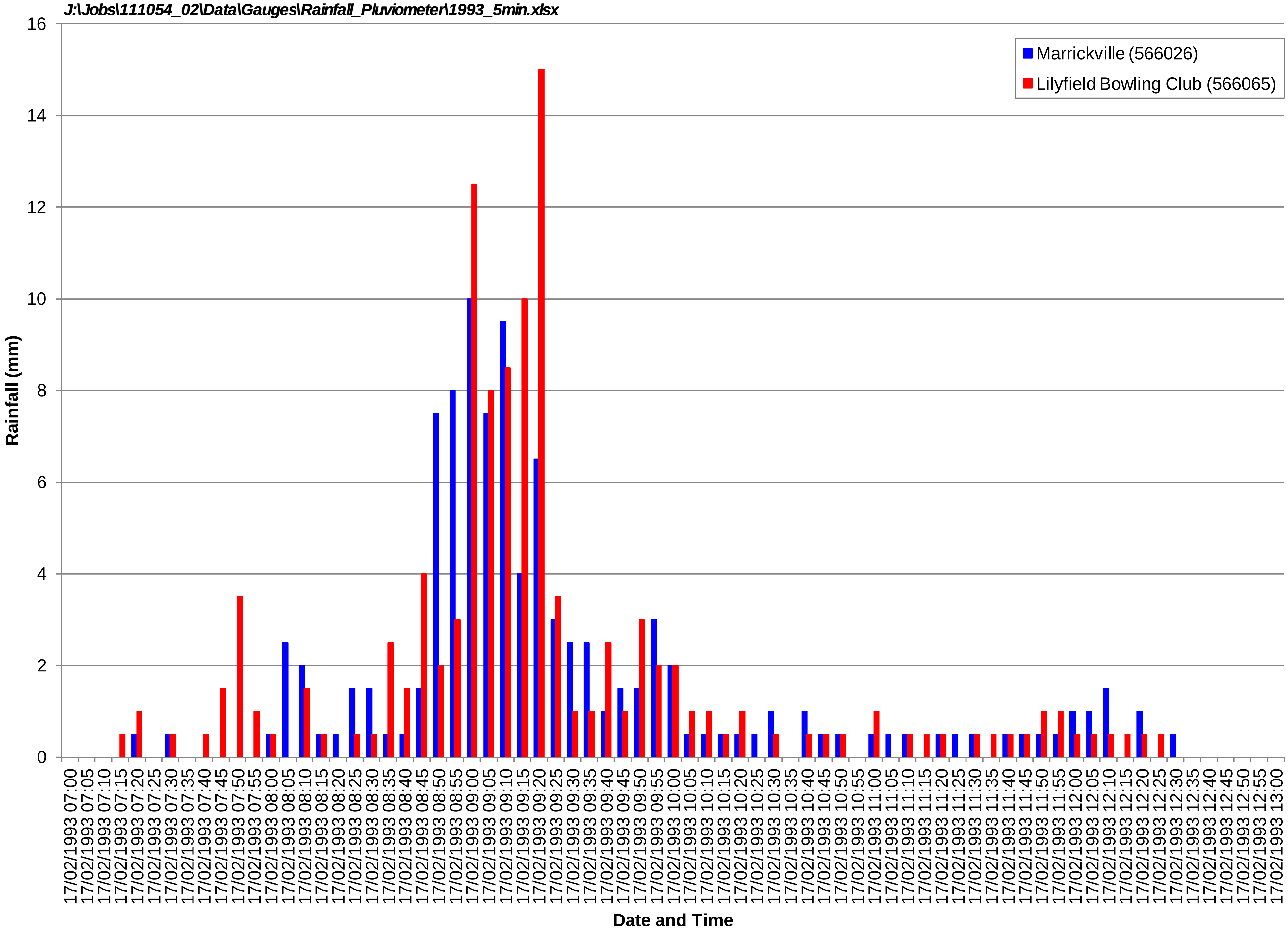

FIGURE 9
RAINFALL HYETOGRAPHS
FEBRUARY 1993

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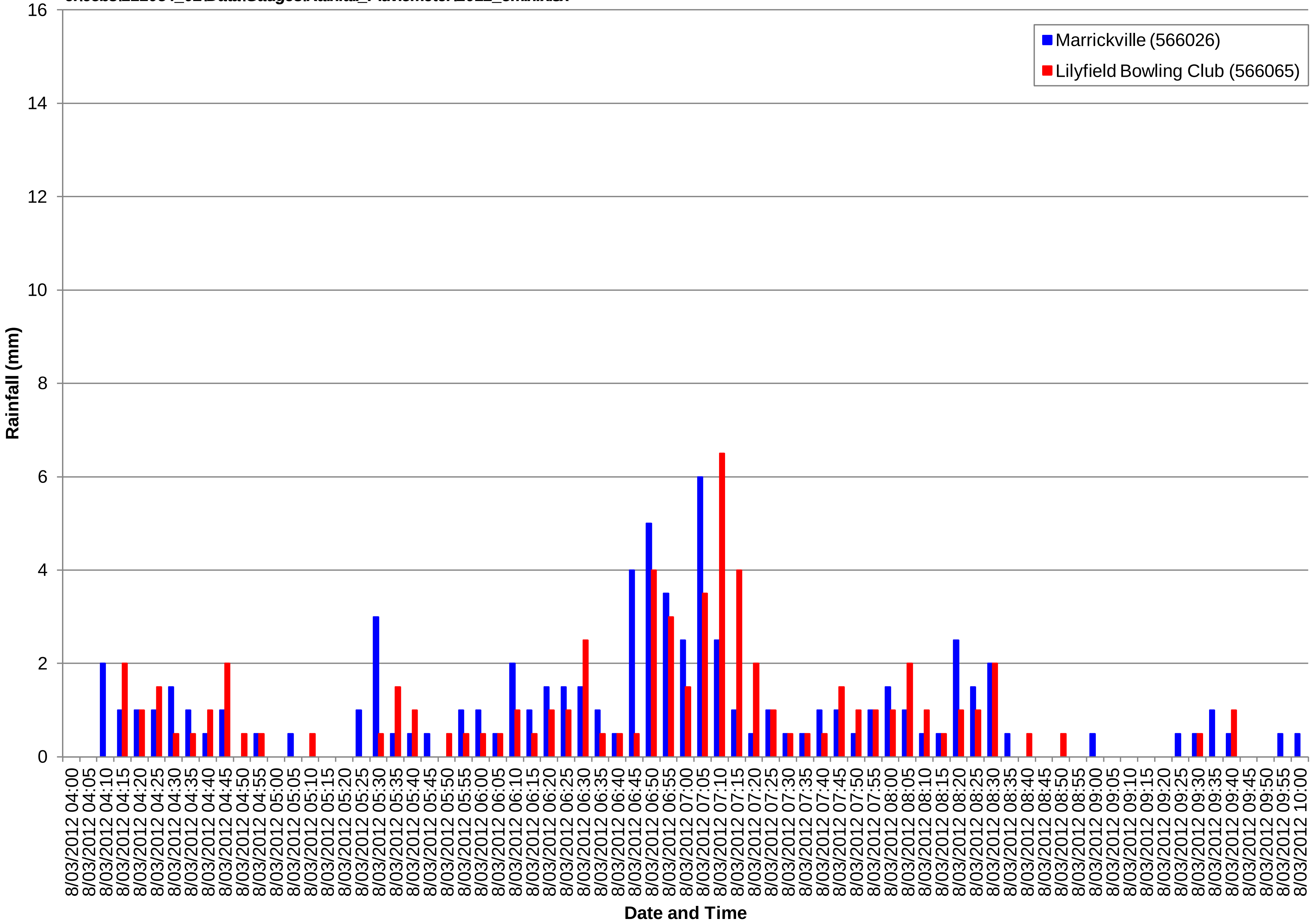

FIGURE 10
RAINFALL HYETOGRAPHS
MARCH 2012

FIGURE 11

IFD Data and February 1993 Event Comparison
Marrickville and Lilyfield Gauges

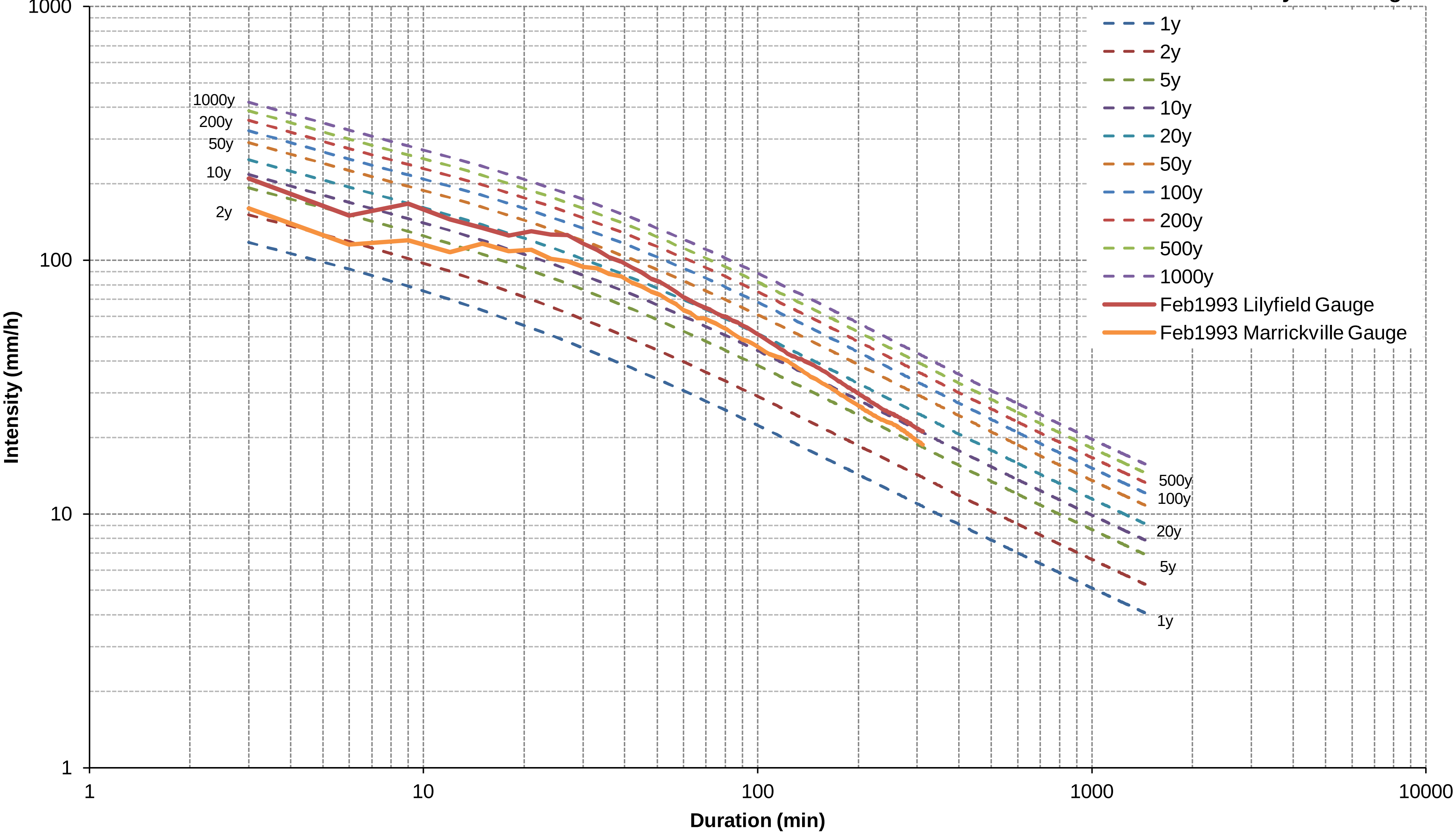

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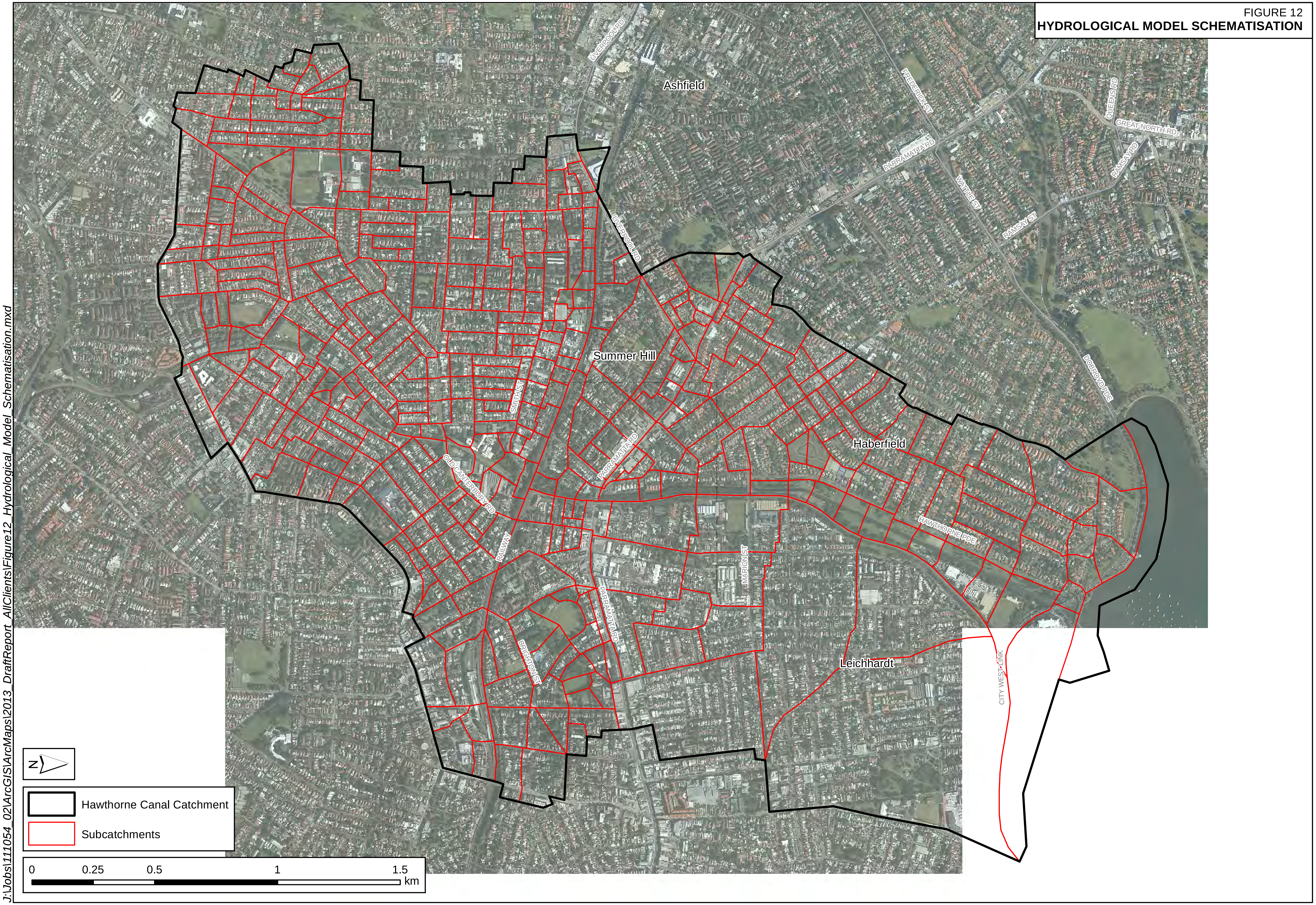
FIGURE 12
HYDROLOGICAL MODEL SCHEMATISATION



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HYDRAULIC MODEL SCHEMATISATION

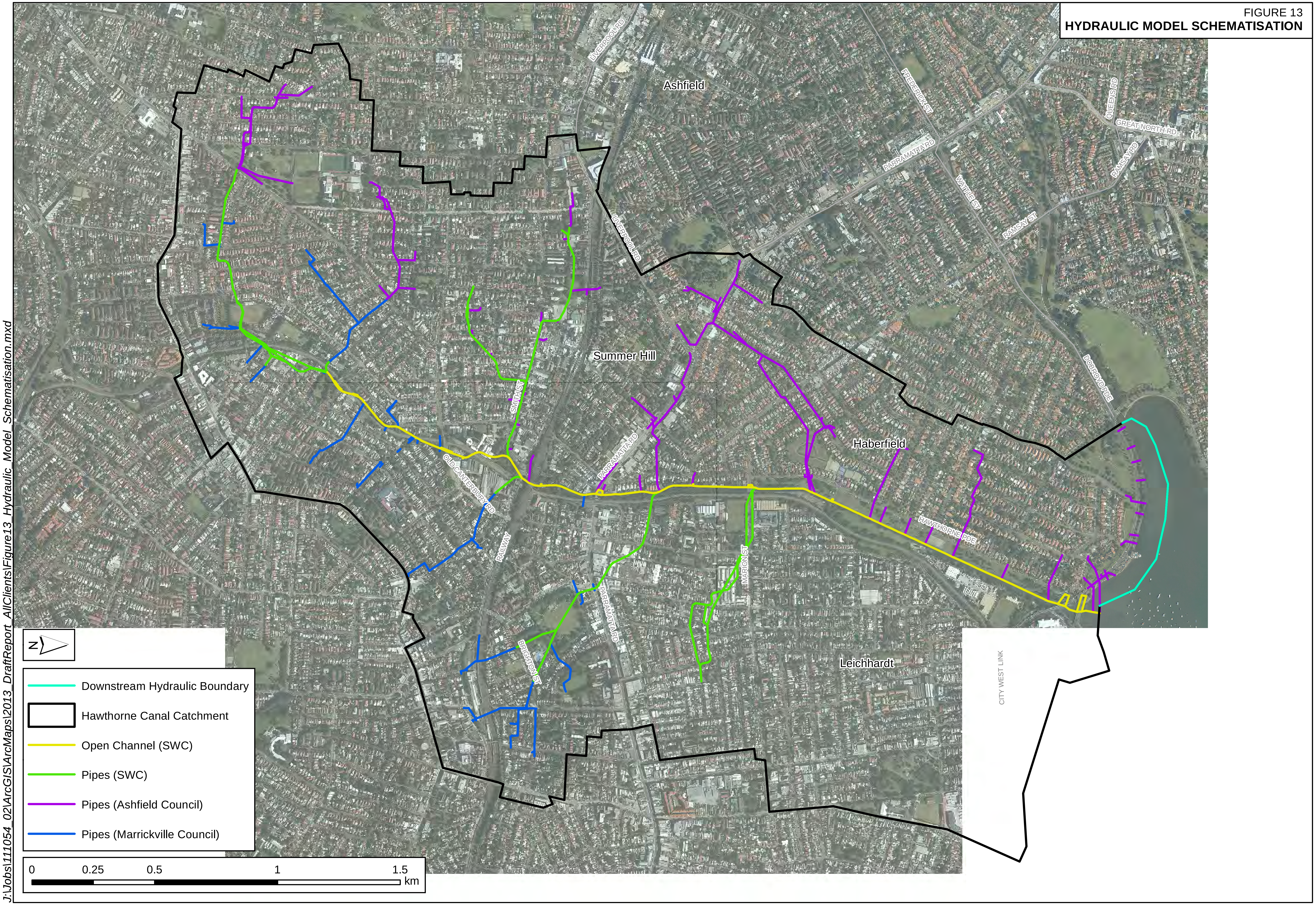

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FIGURE 14
HYDRAULIC MODEL ROUGHNESS LAYOUT

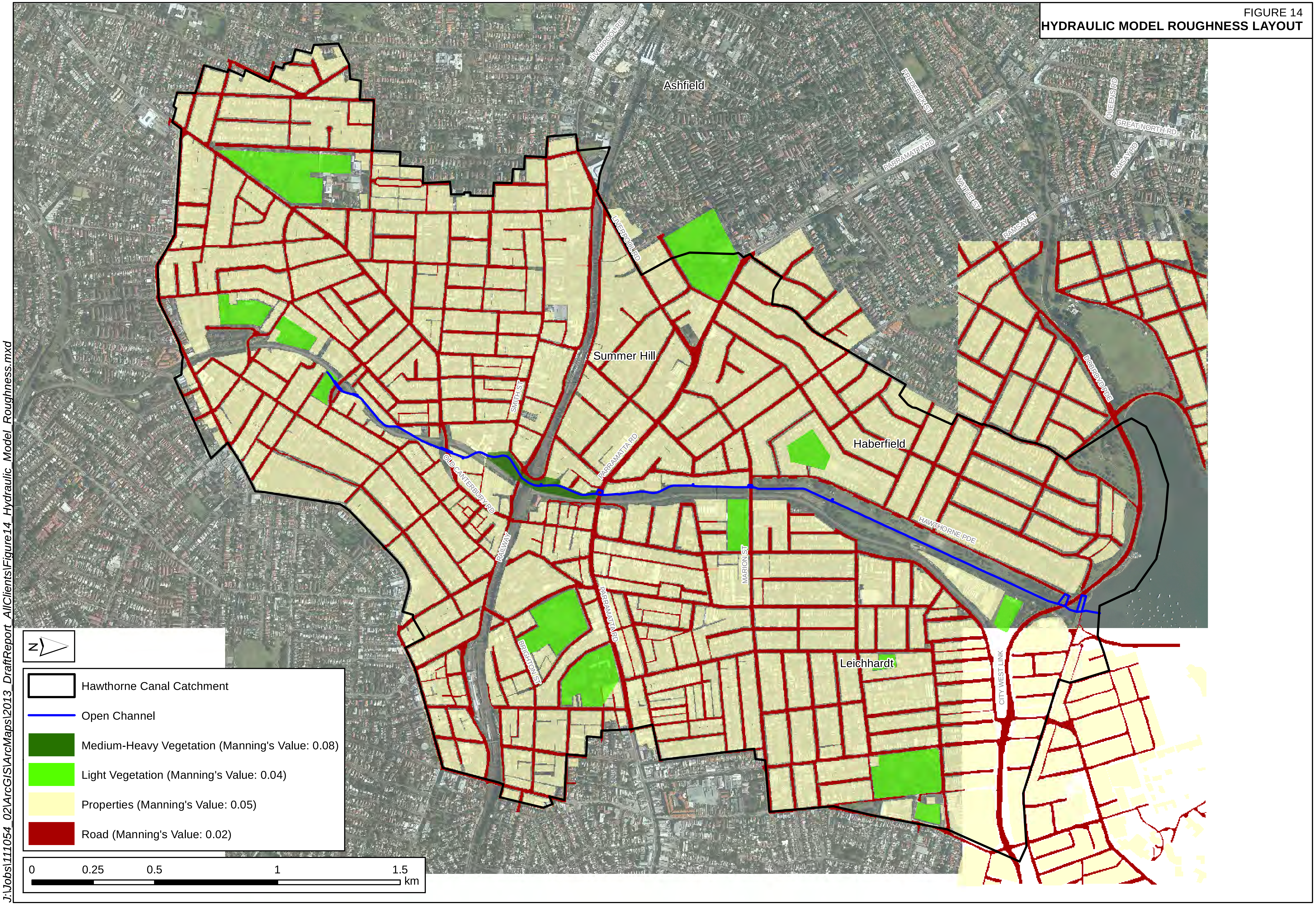

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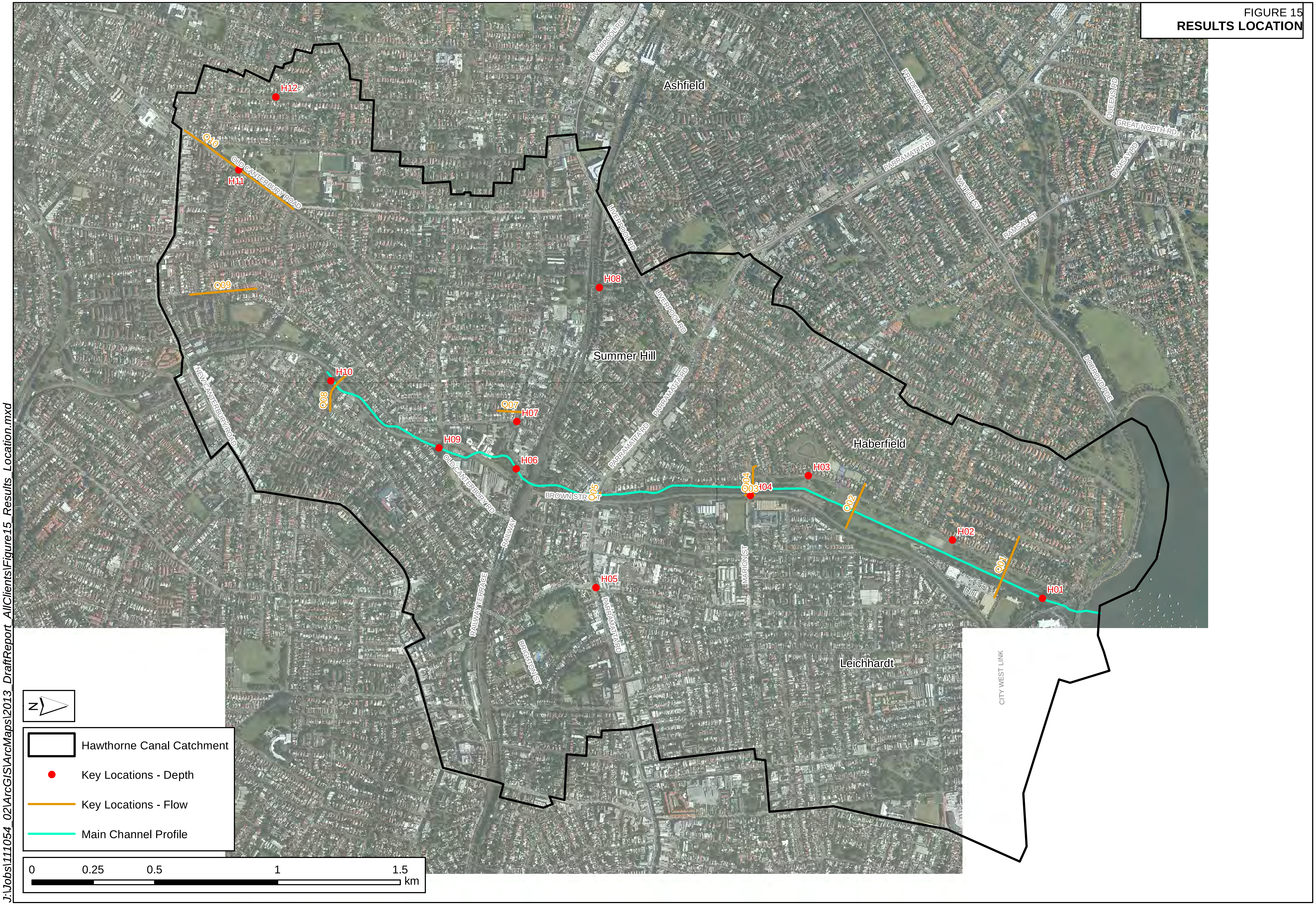
FIGURE 15
RESULTS LOCATION



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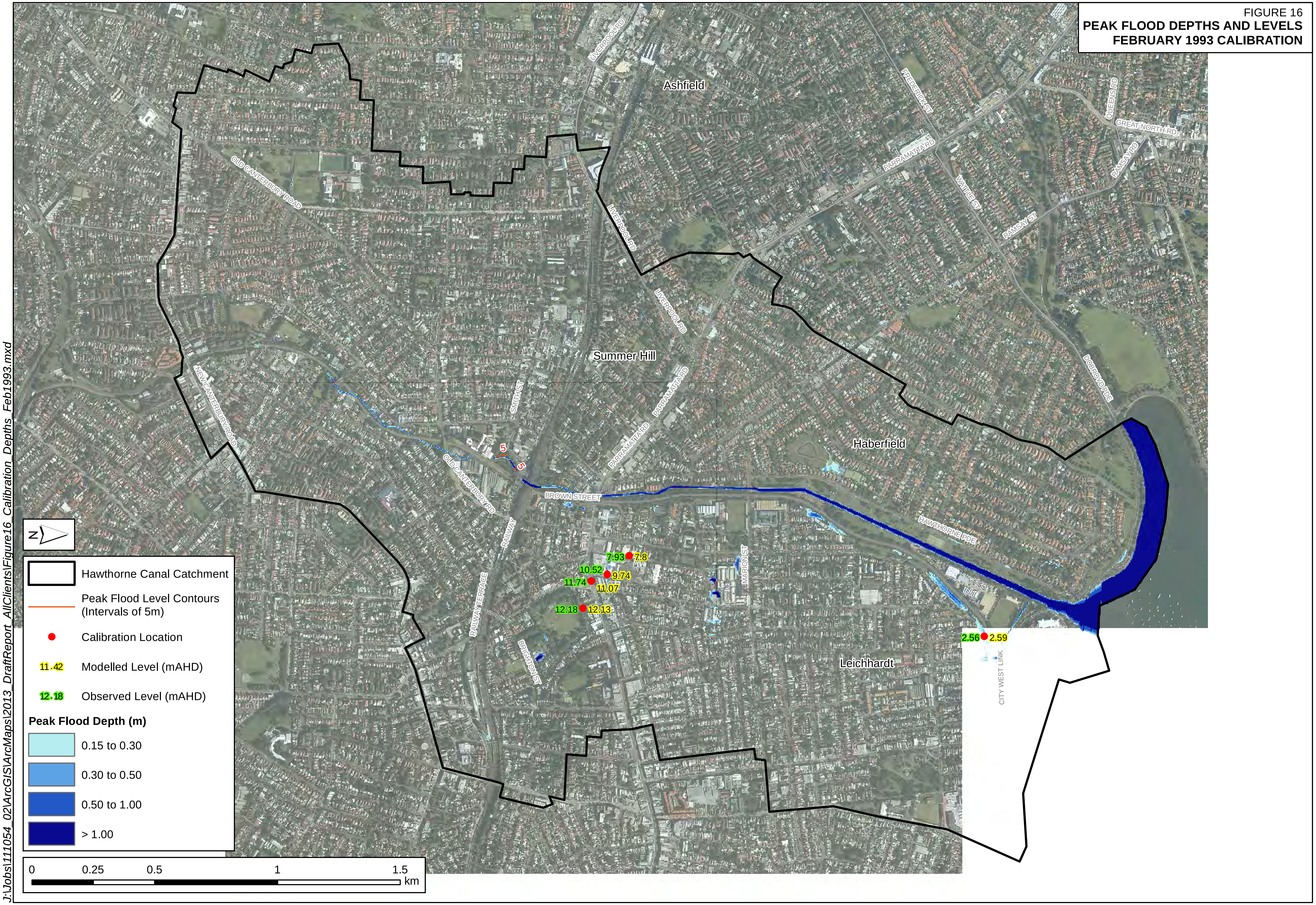

FIGURE 16
PEAK FLOOD DEPTHS AND LEVELS FEBRUARY 1993 CALIBRATION

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FIGURE 17
PEAK FLOOD DEPTHS AND LEVELS
MARCH 2012 VALIDATION

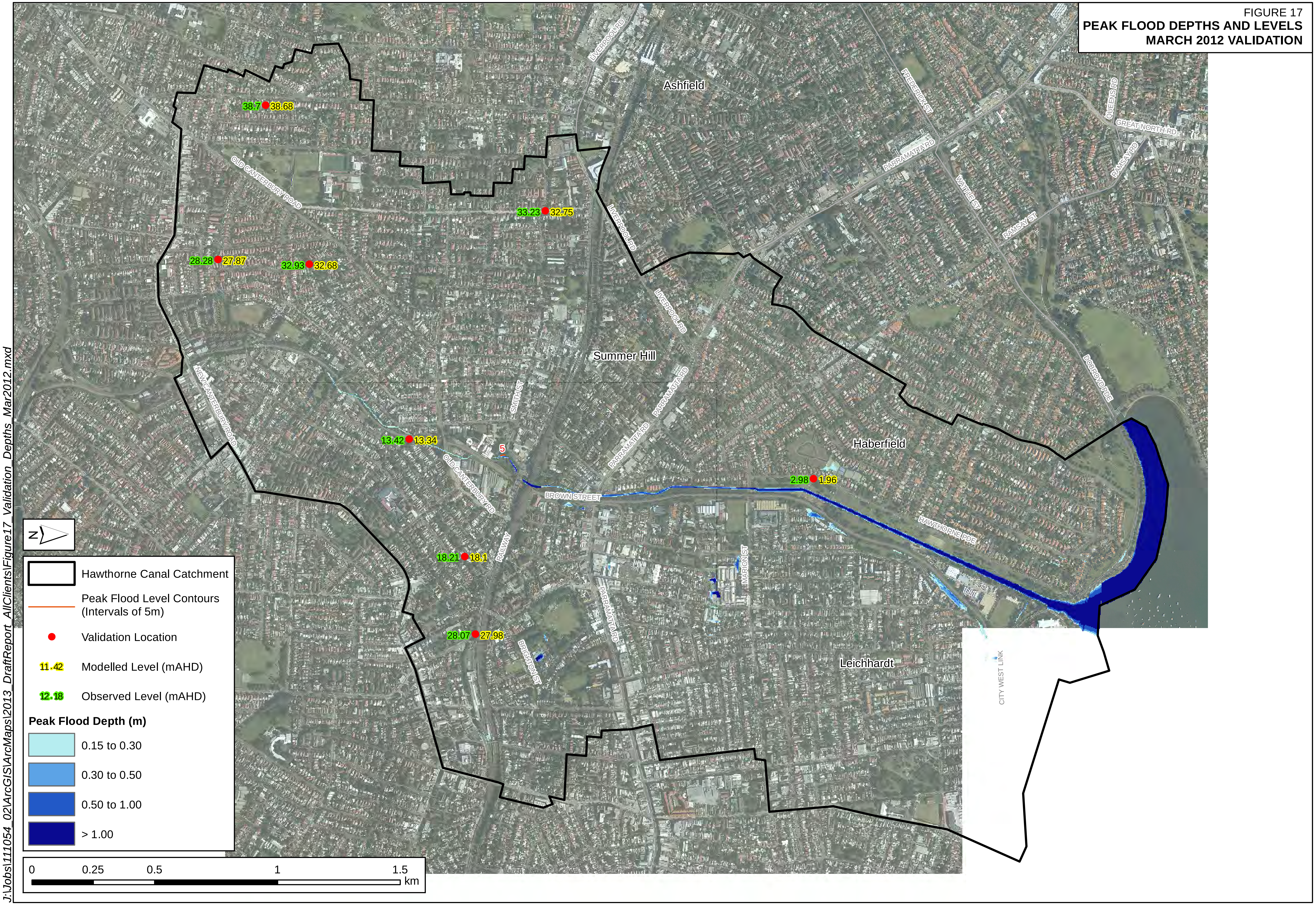

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FIGURE 18
PEAK FLOOD DEPTHS AND LEVELS
5 YEAR ARI VERIFICATION

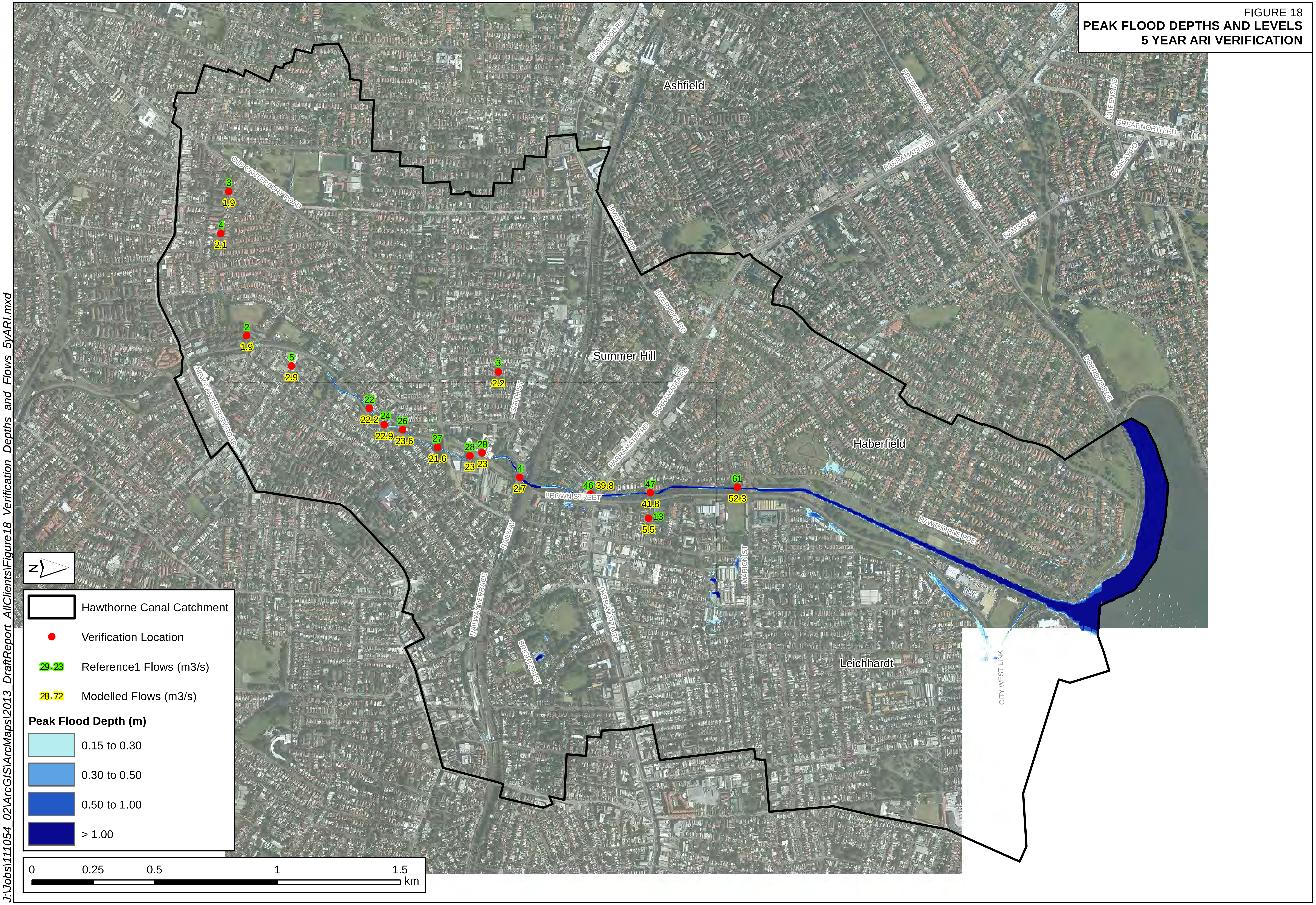

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FIGURE 19
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
2 YEAR ARI

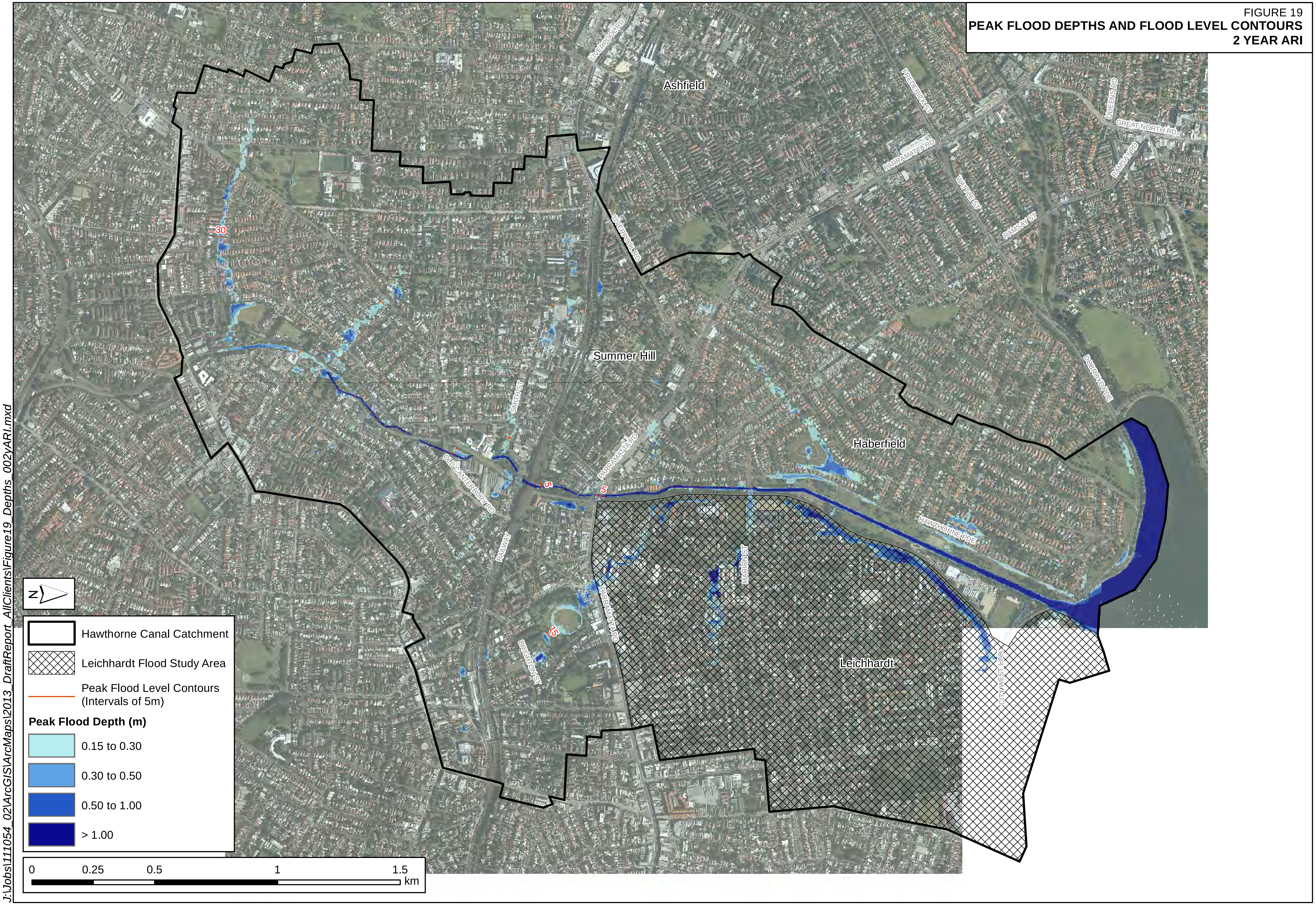

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FIGURE 20
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
5 YEAR ARI

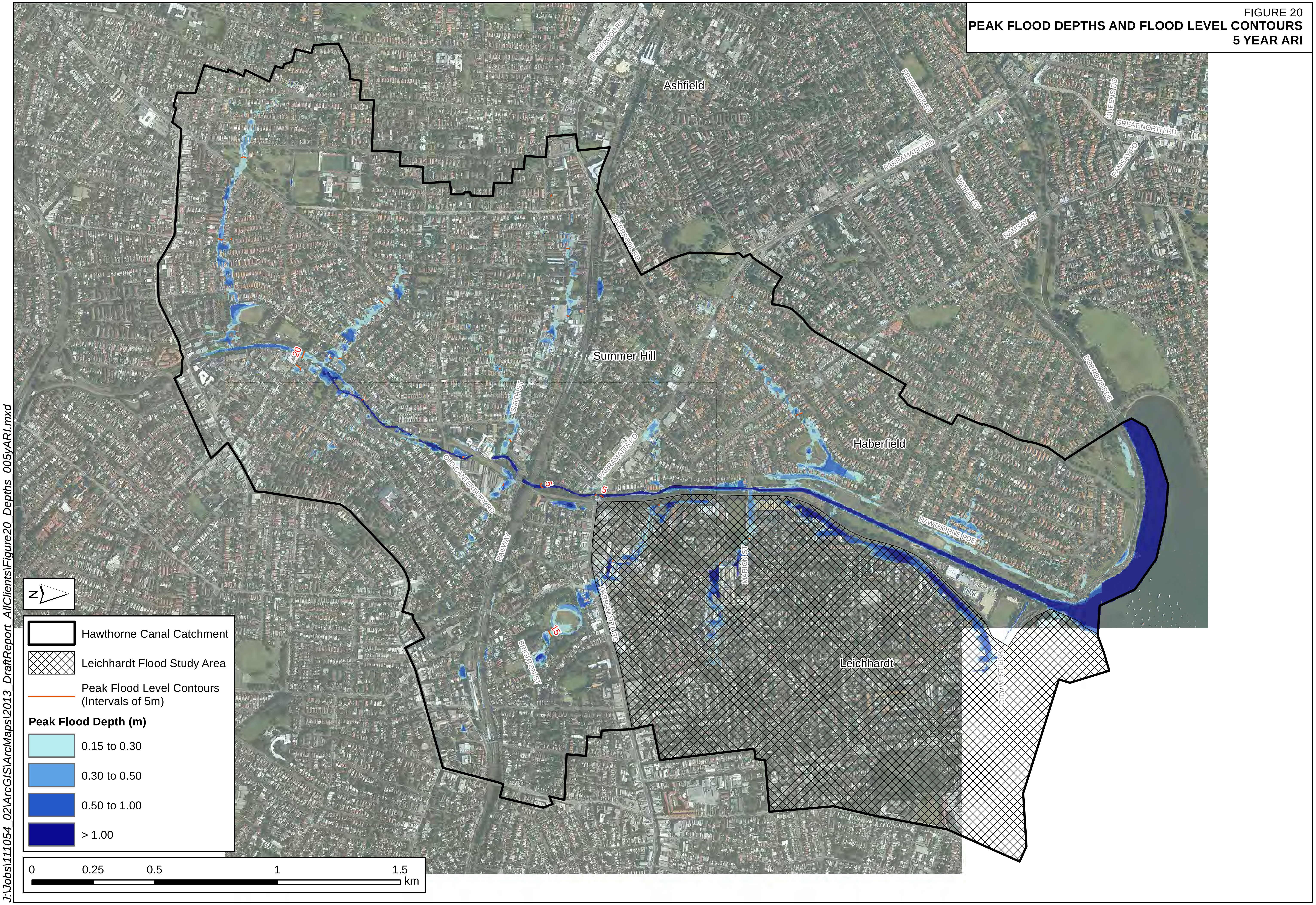

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FIGURE 21
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
10 YEAR ARI

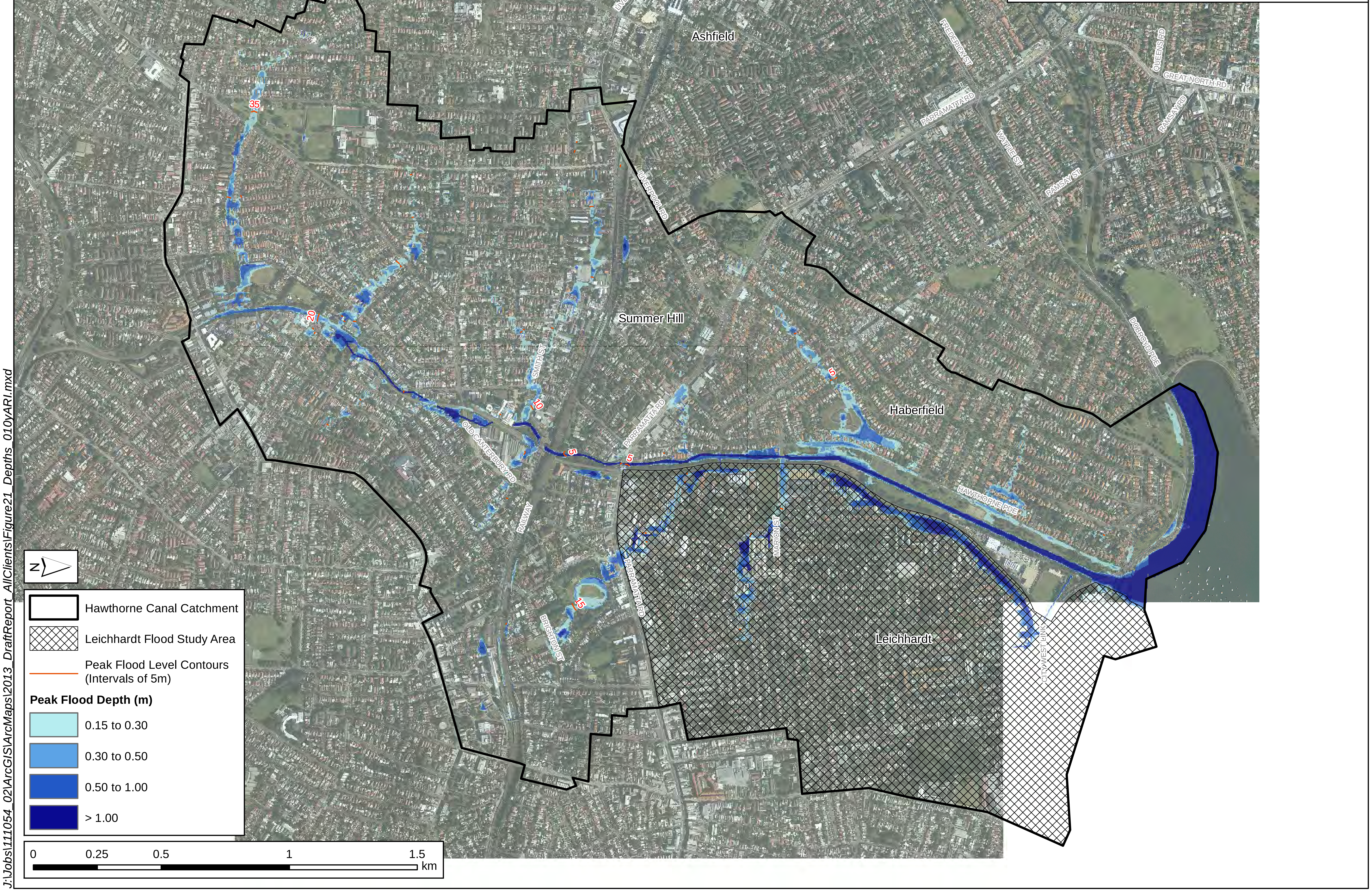

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FIGURE 22
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
20 YEAR ARI

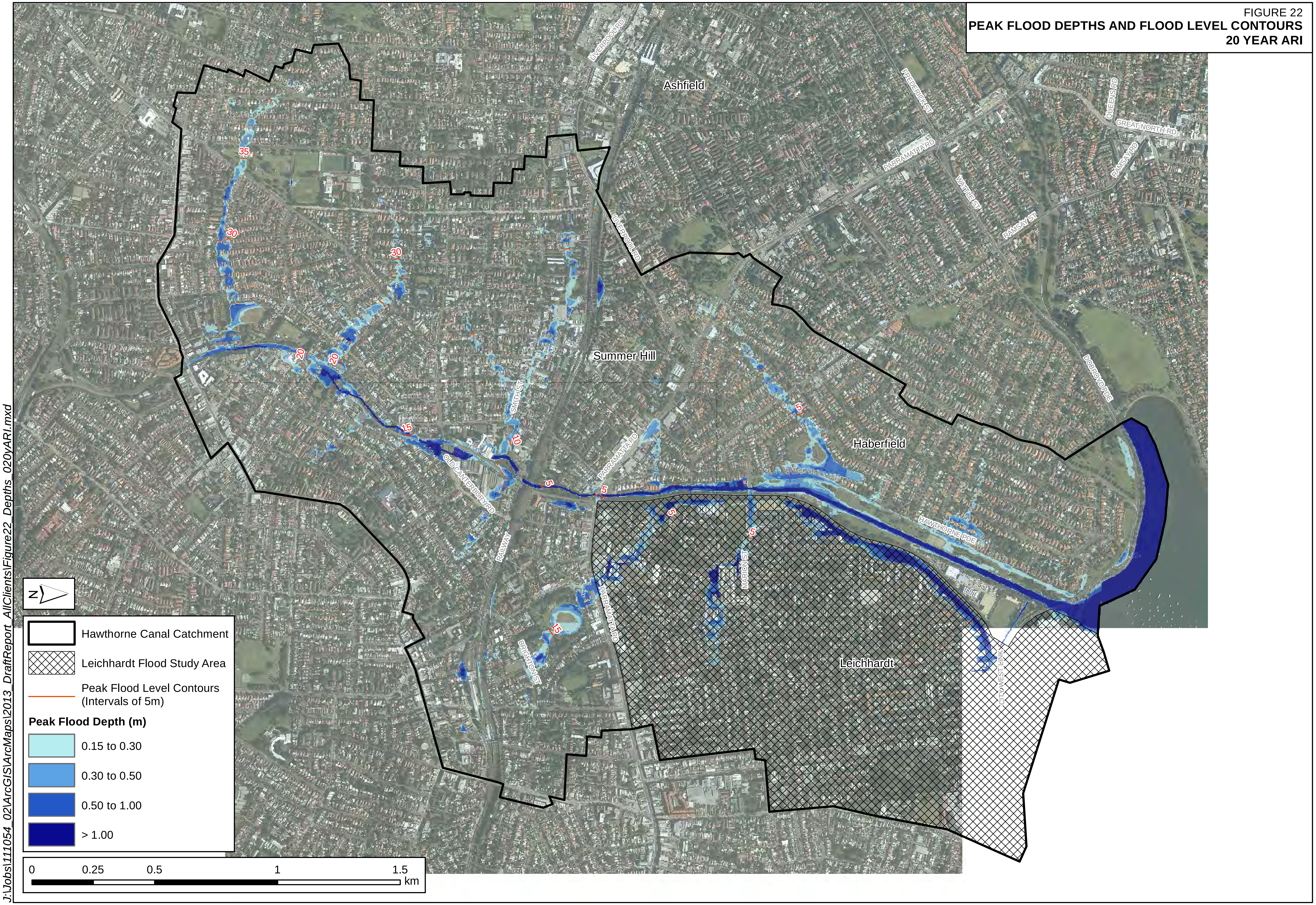

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FIGURE 23
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
50 YEAR ARI

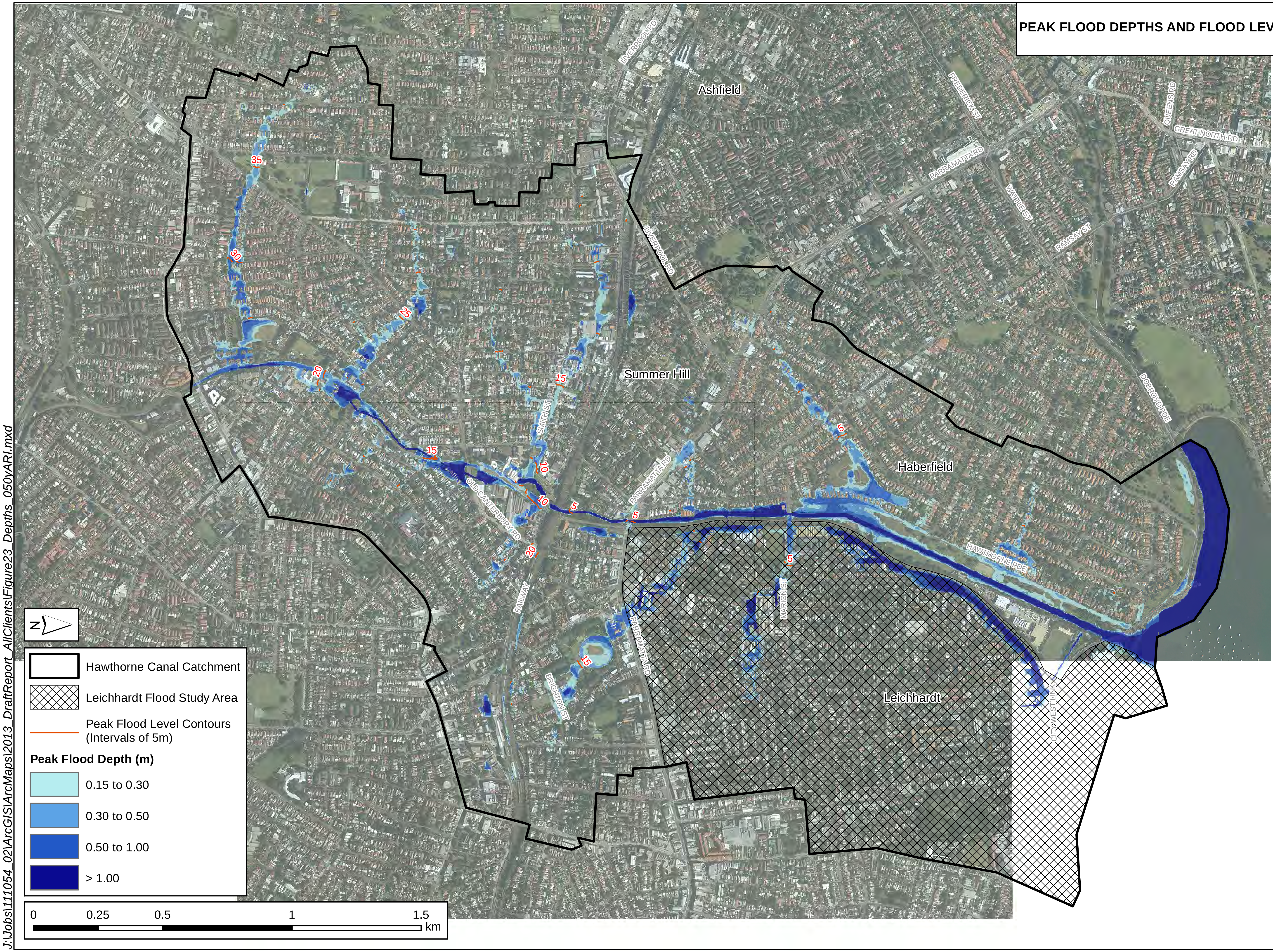

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FIGURE 24
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
100 YEAR ARI

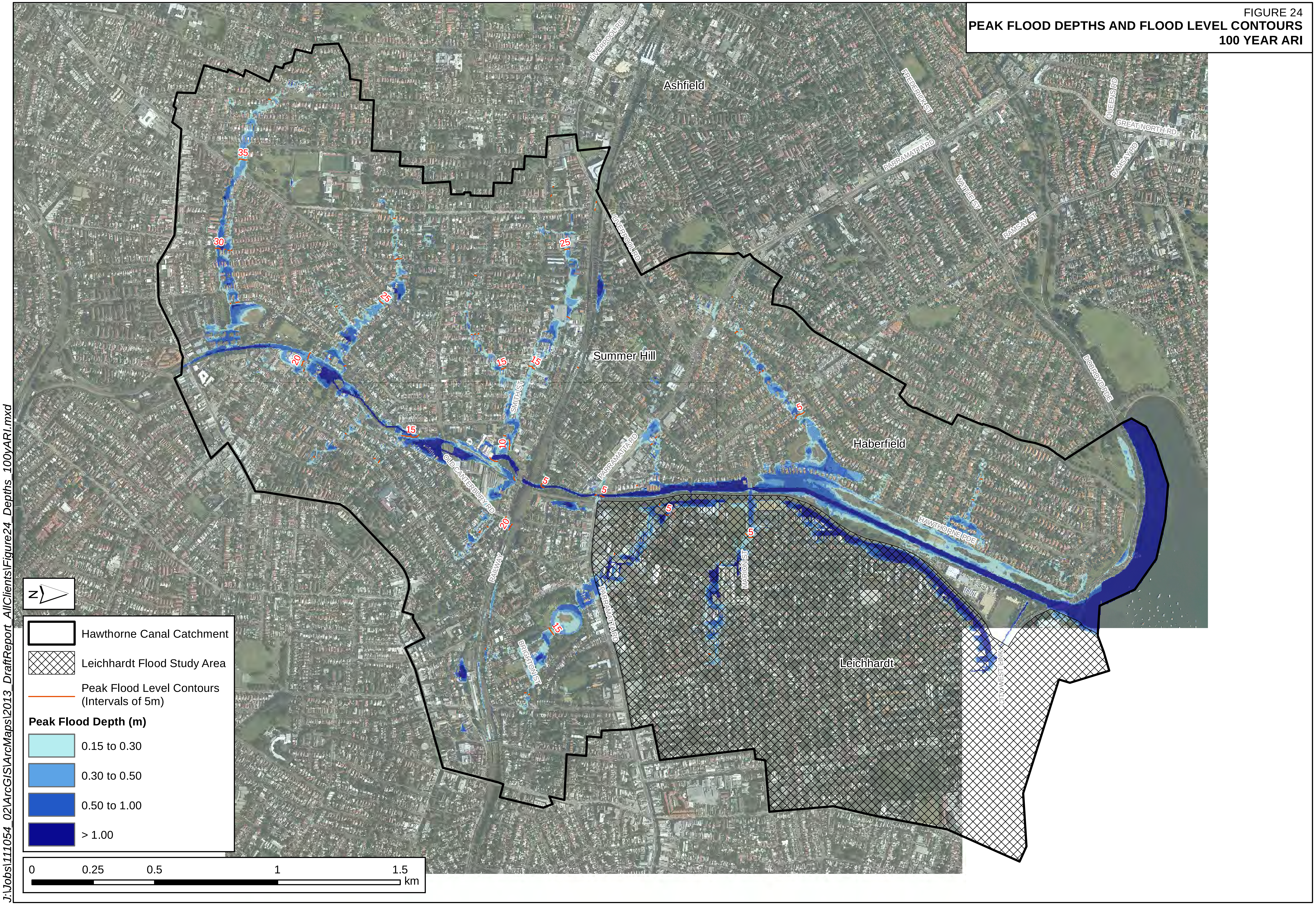

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FIGURE 25

PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
PMF

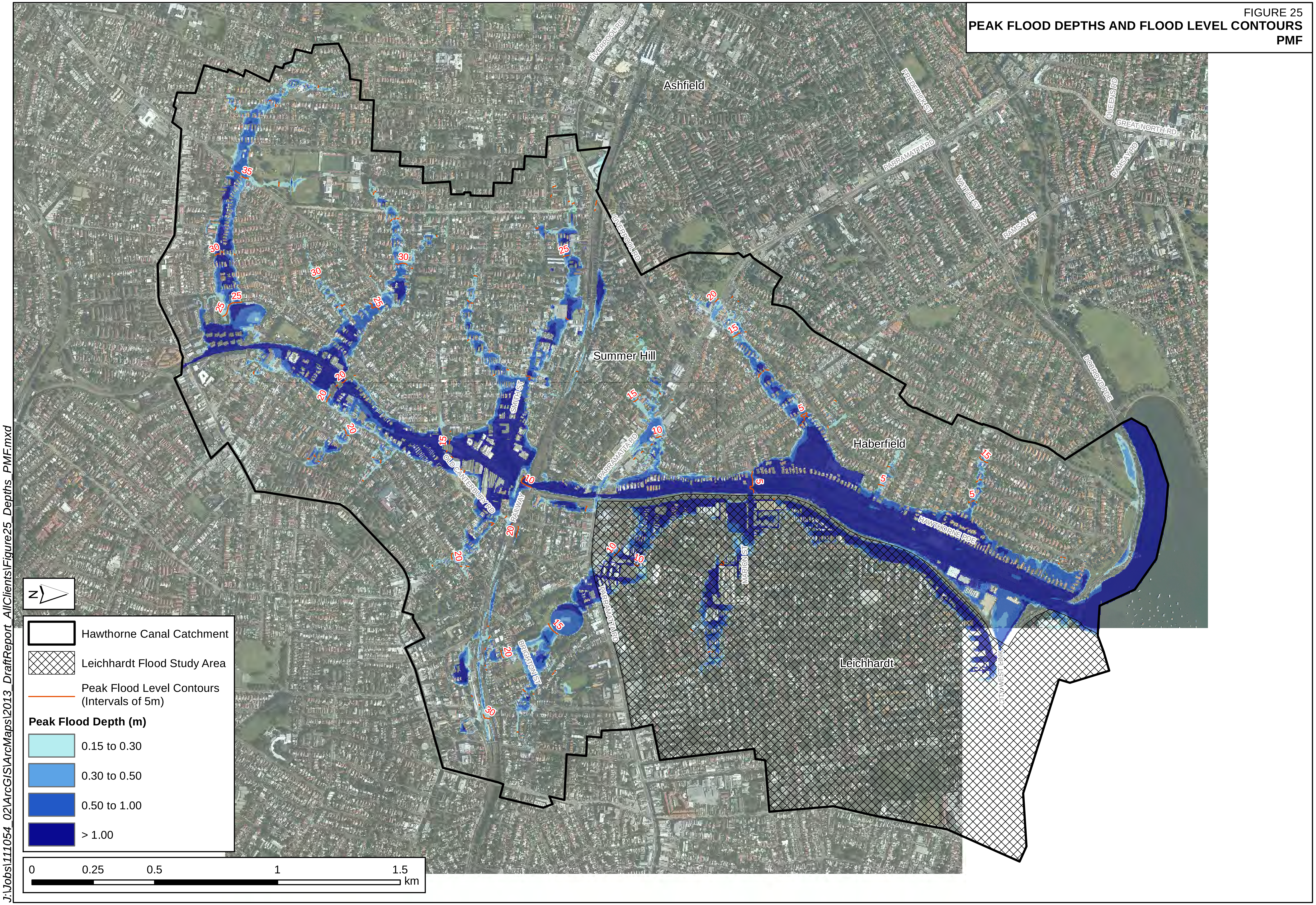

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FIGURE 26A
PEAK FLOOD LEVEL PROFILES
DESIGN EVENTS

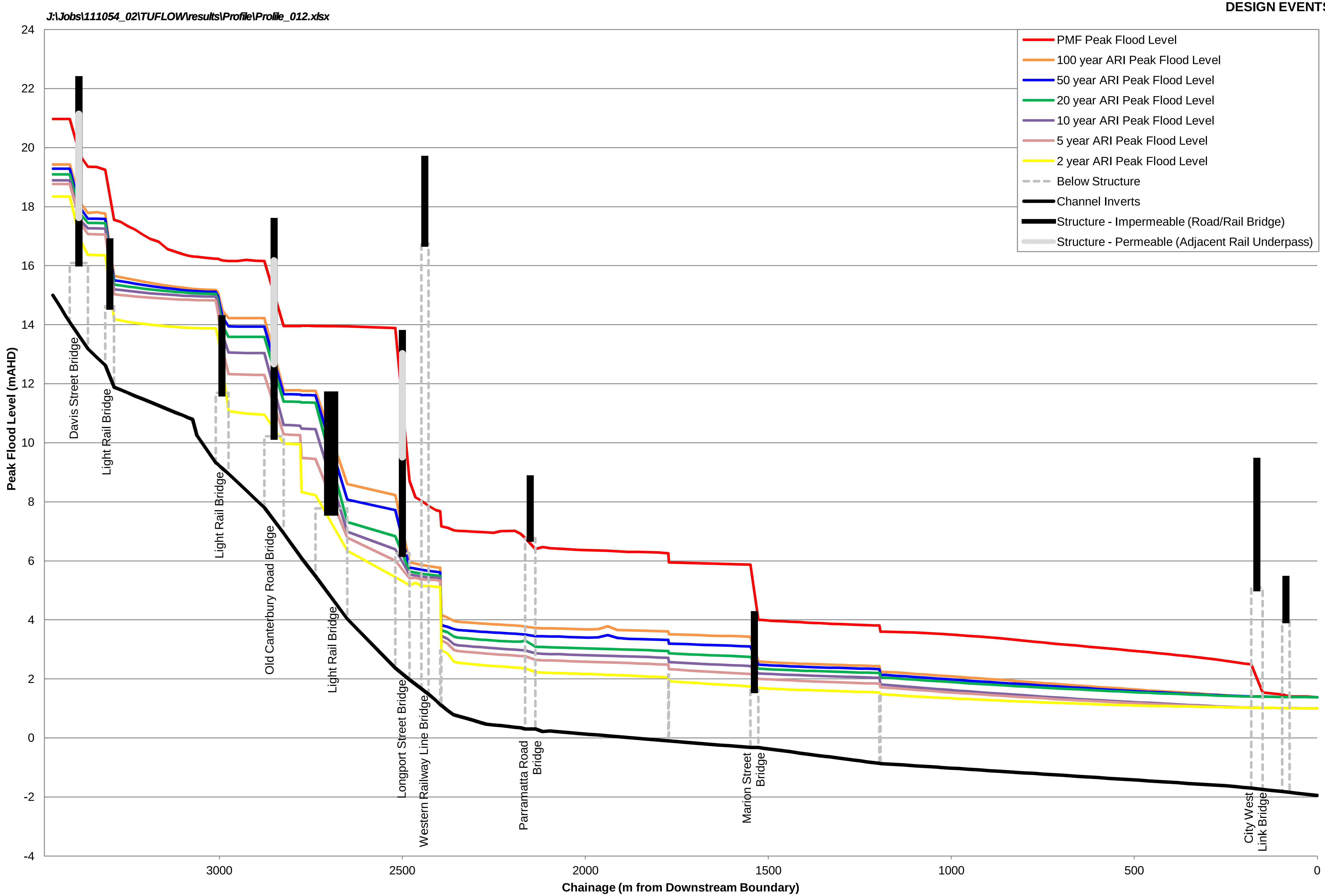

FIGURE 26B

PEAK FLOOD LEVEL PROFILES
SEA LEVEL RISE EVENTS

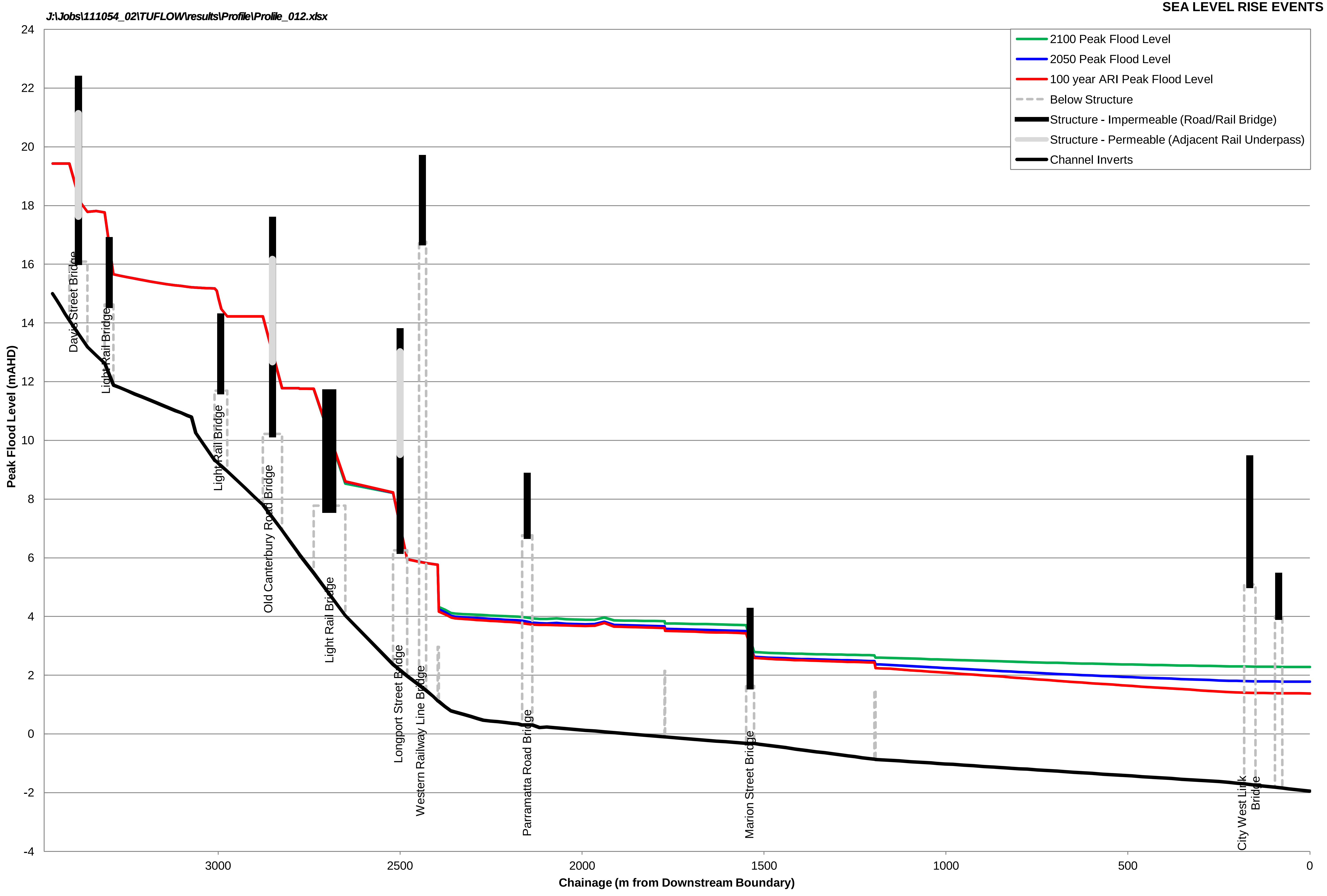

FIGURE 27
PEAK FLOOD VELOCITY
100 YEAR ARI

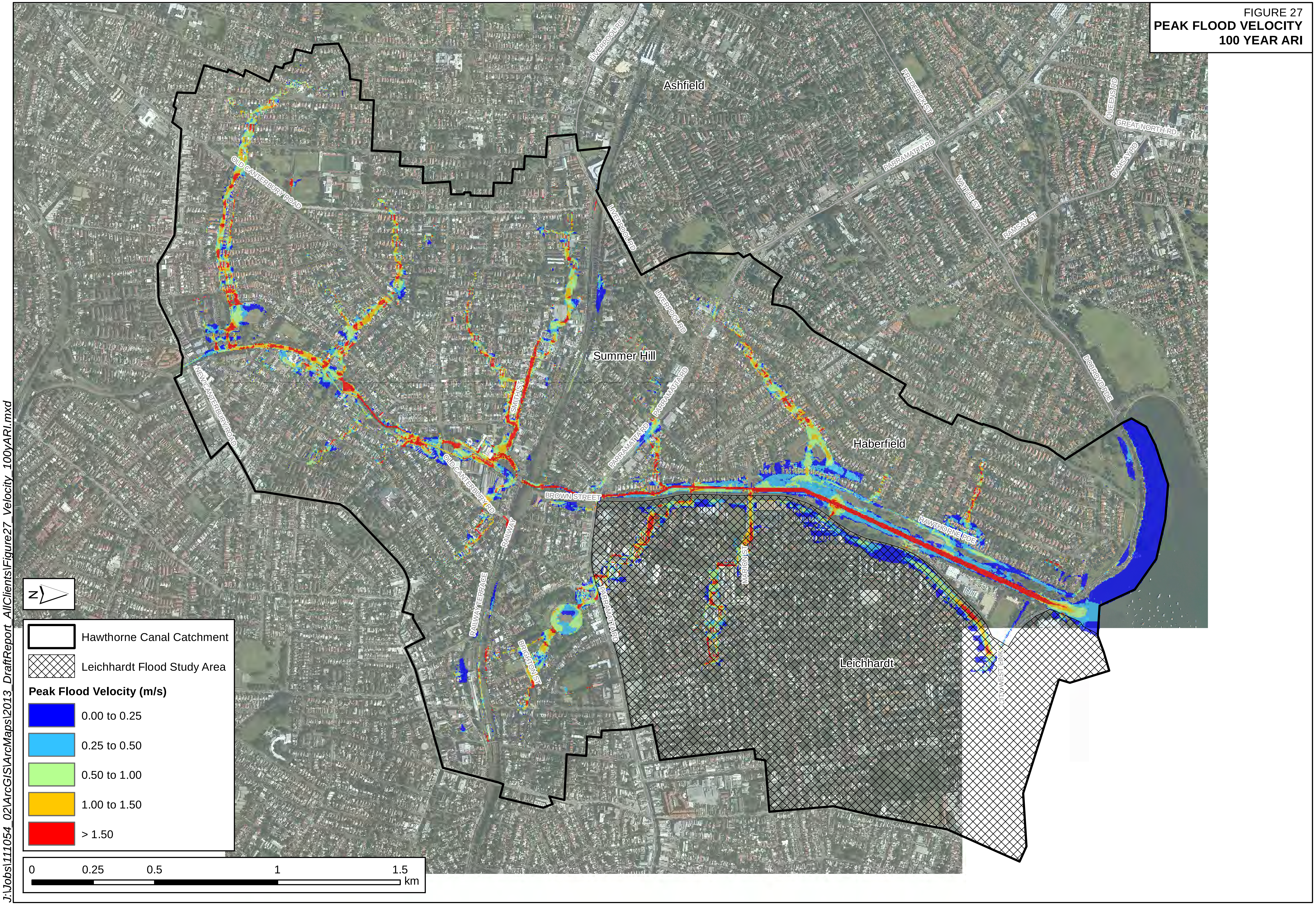

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FIGURE 28
PROVISIONAL HYDRAULIC HAZARD
5 YEAR ARI

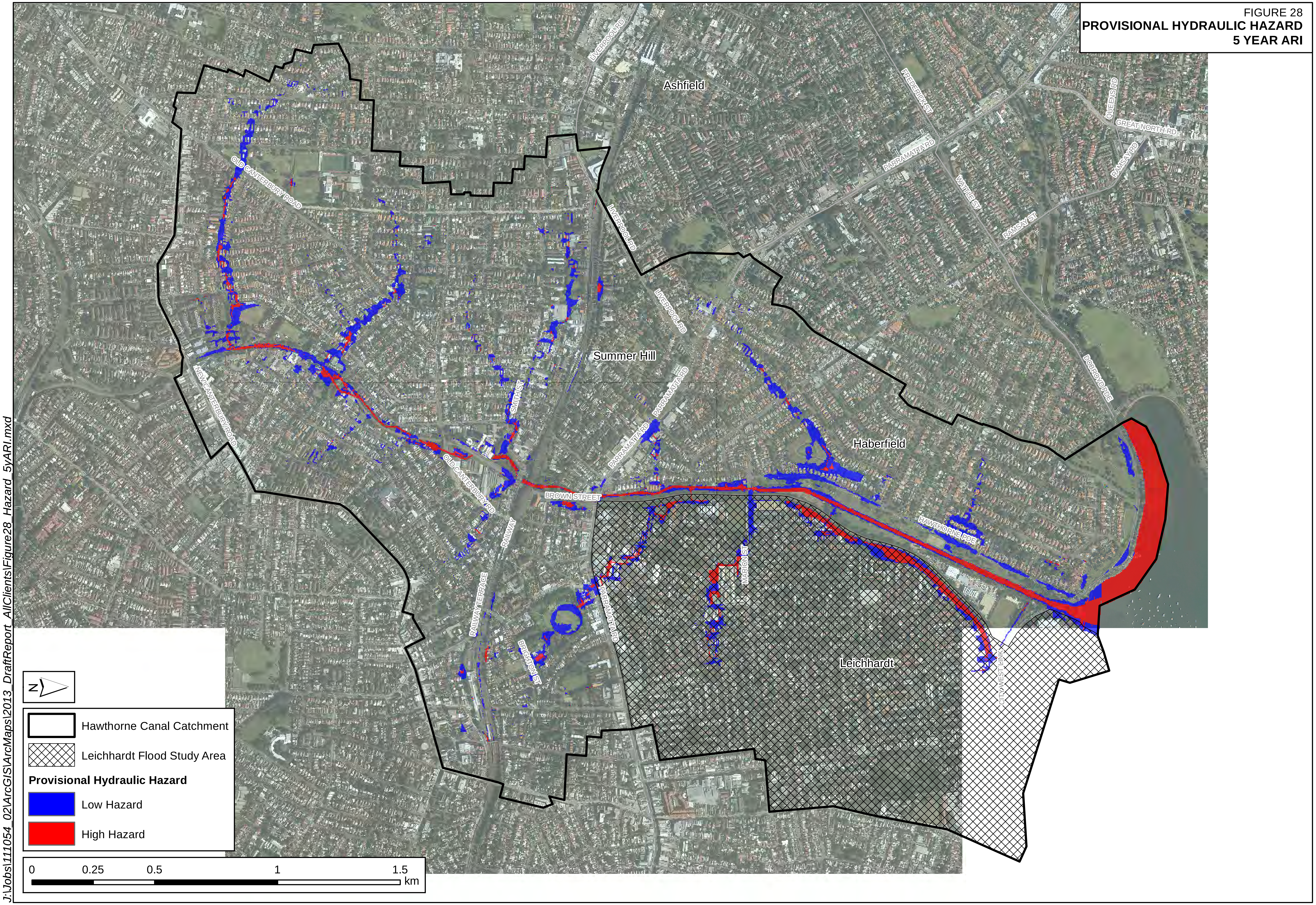

FIGURE 29
PROVISIONAL HYDRAULIC HAZARD
20 YEAR ARI

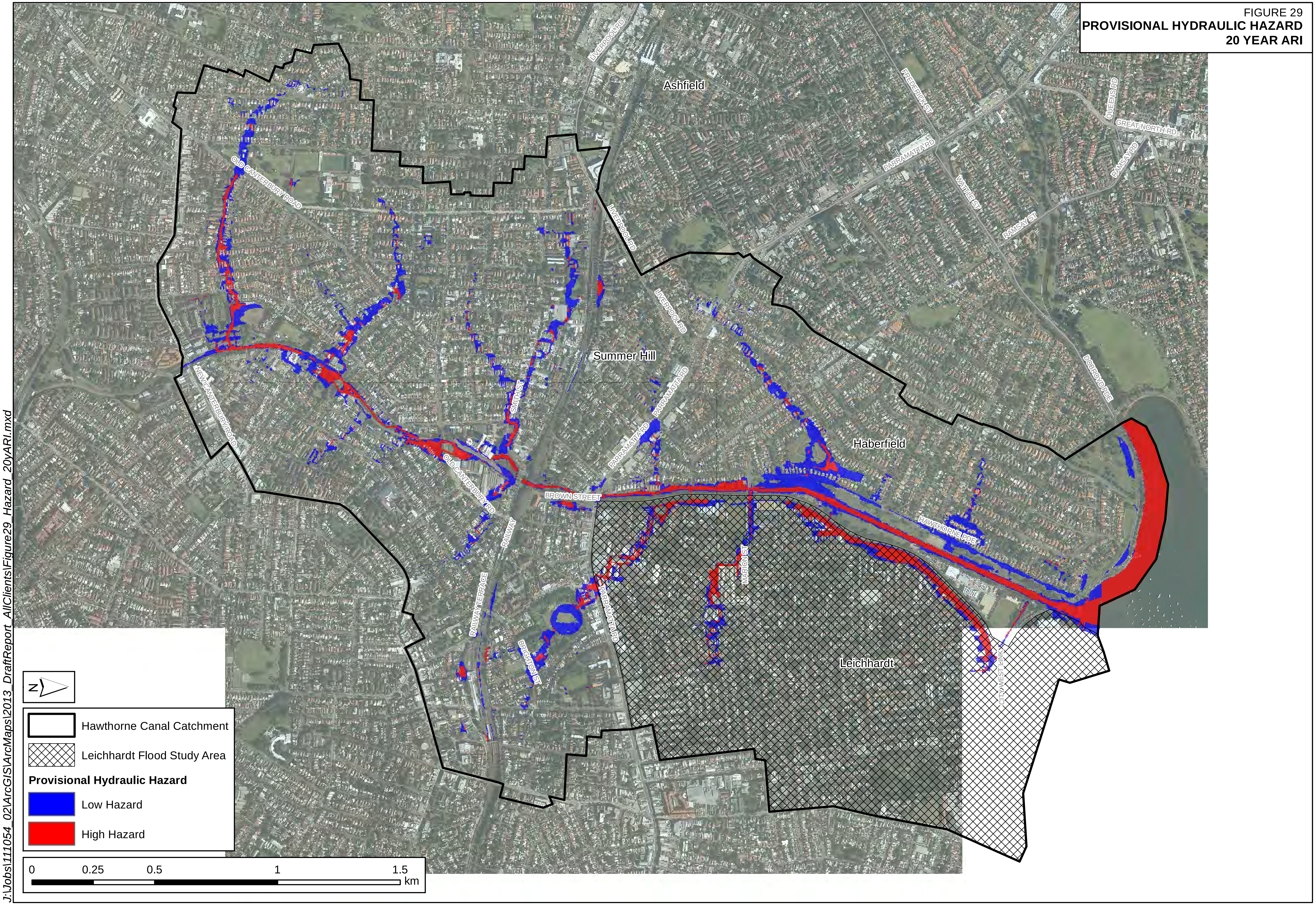

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FIGURE 30
PROVISIONAL HYDRAULIC HAZARD
100 YEAR ARI

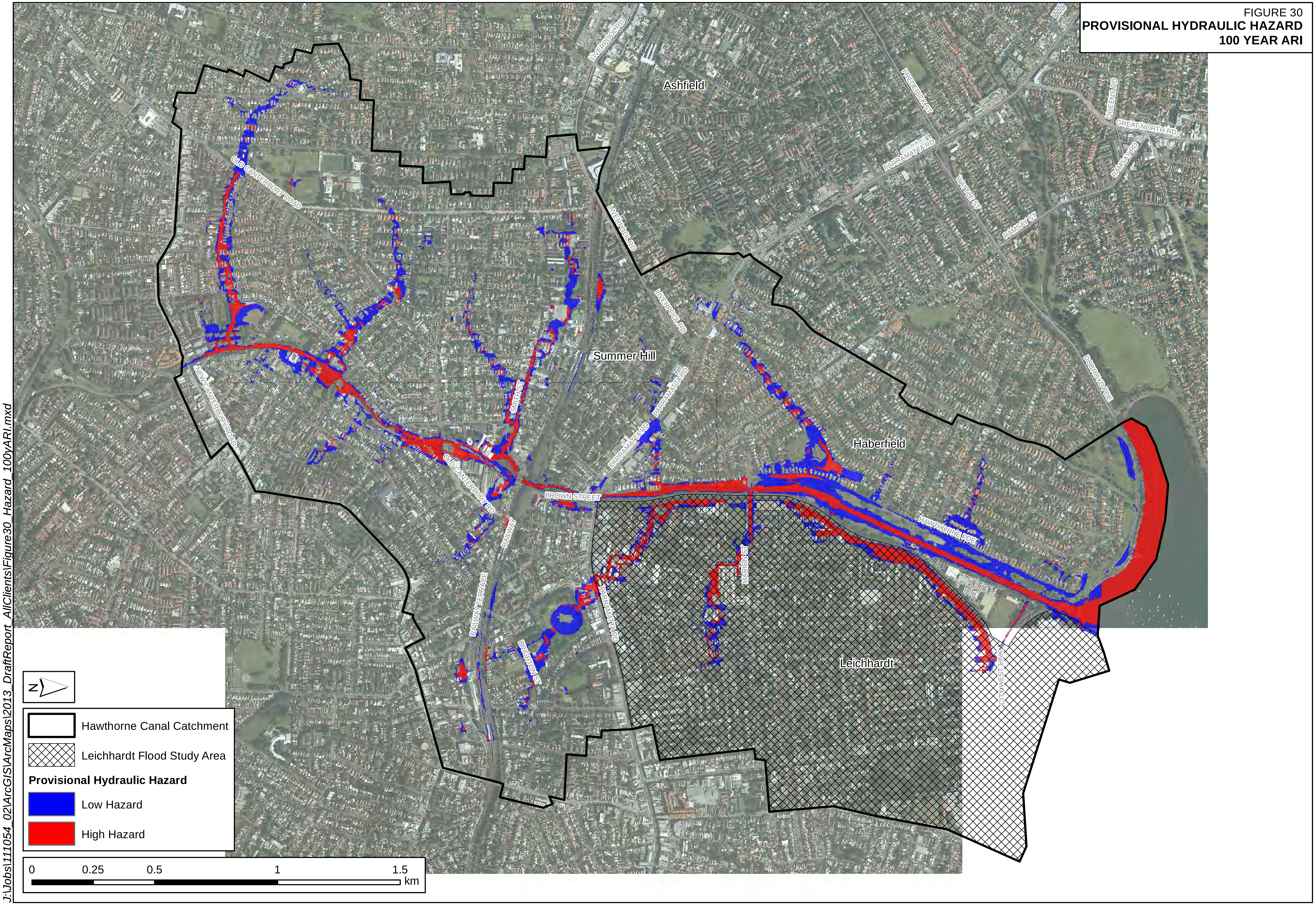

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FIGURE 31
PROVISIONAL HYDRAULIC HAZARD PMF

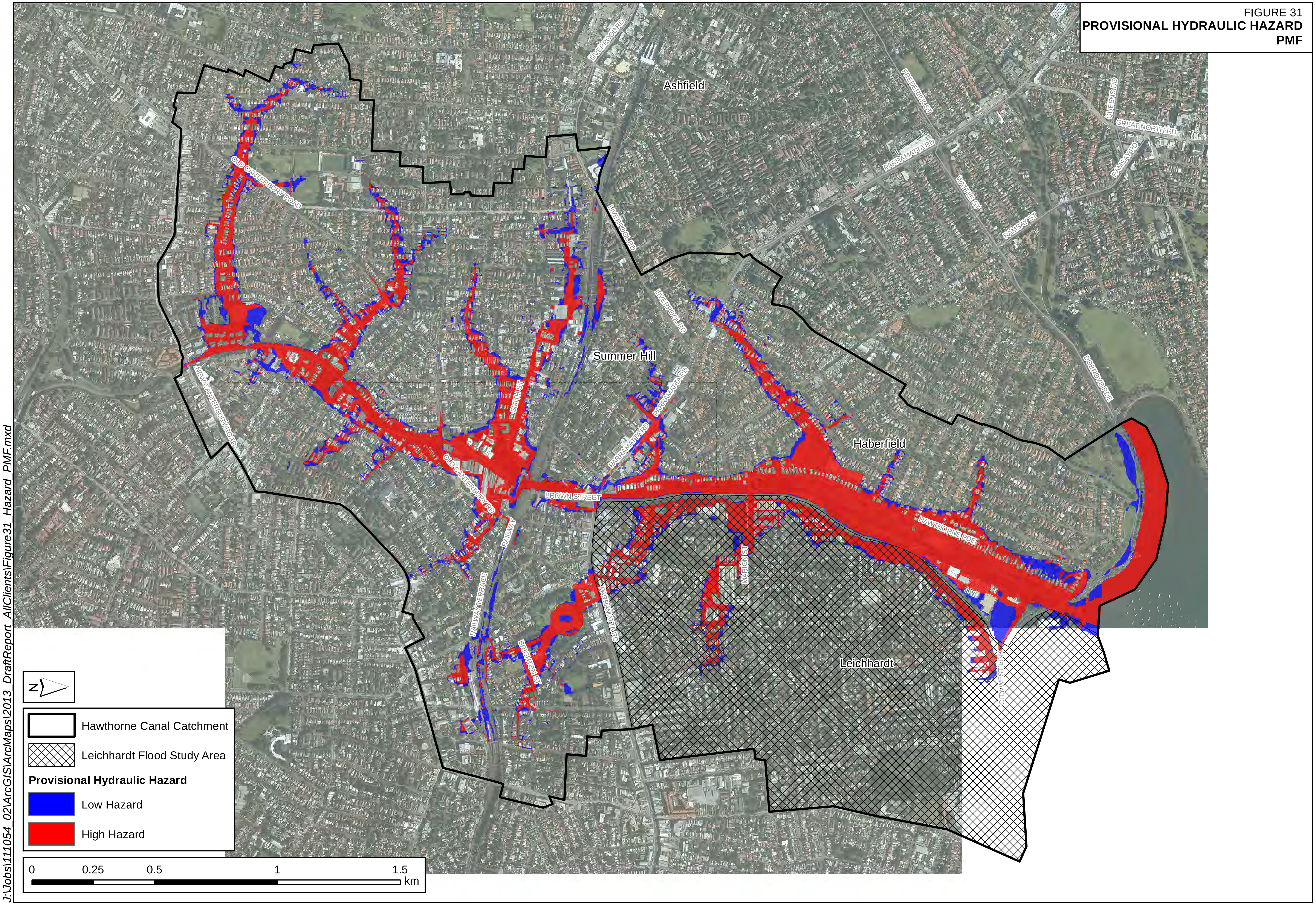

FIGURE 32
PRELIMINARY HYDRAULIC CLASSIFICATION
5 YEAR ARI

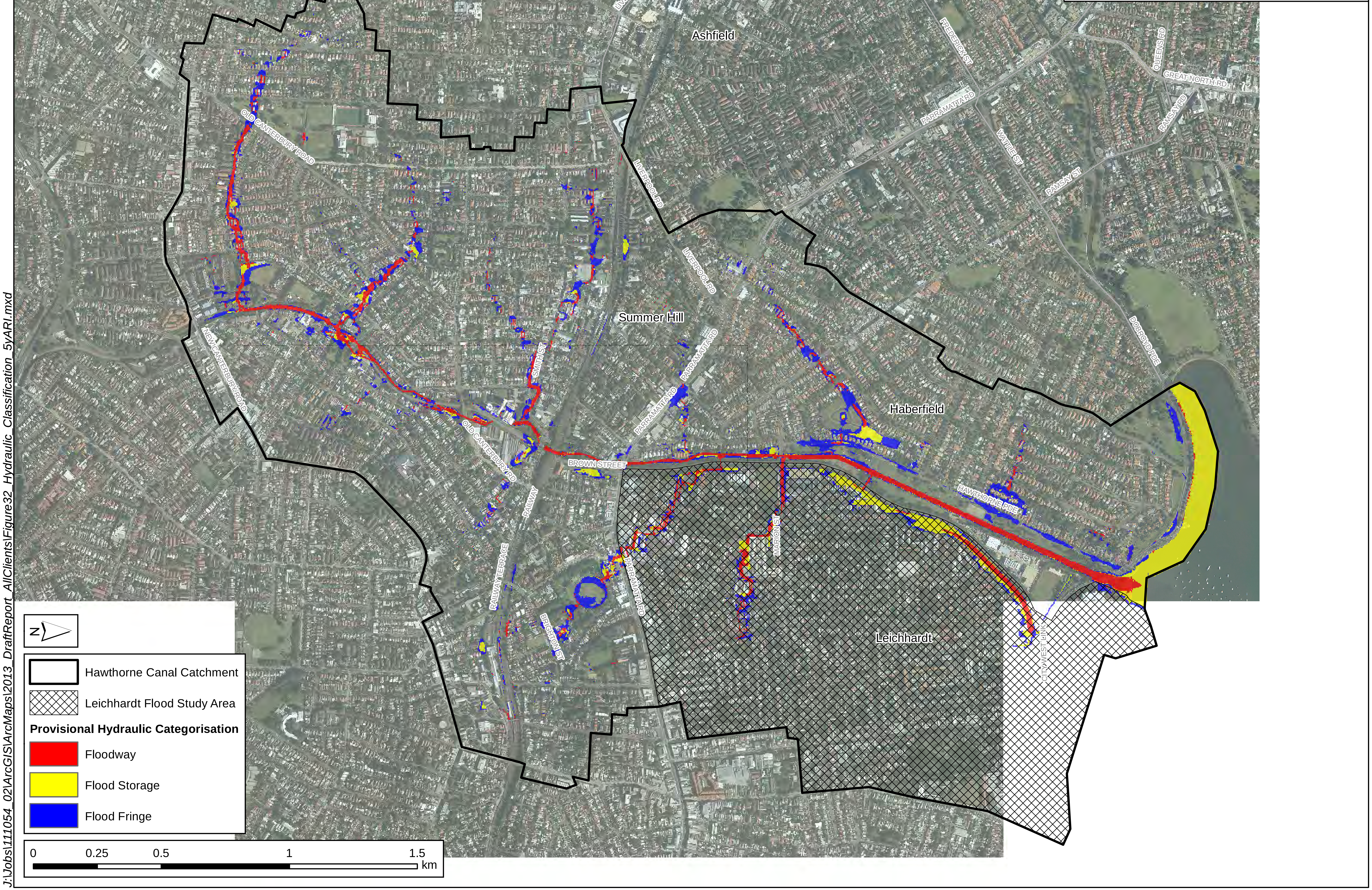

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FIGURE 33
PRELIMINARY HYDRAULIC CLASSIFICATION
20 YEAR ARI

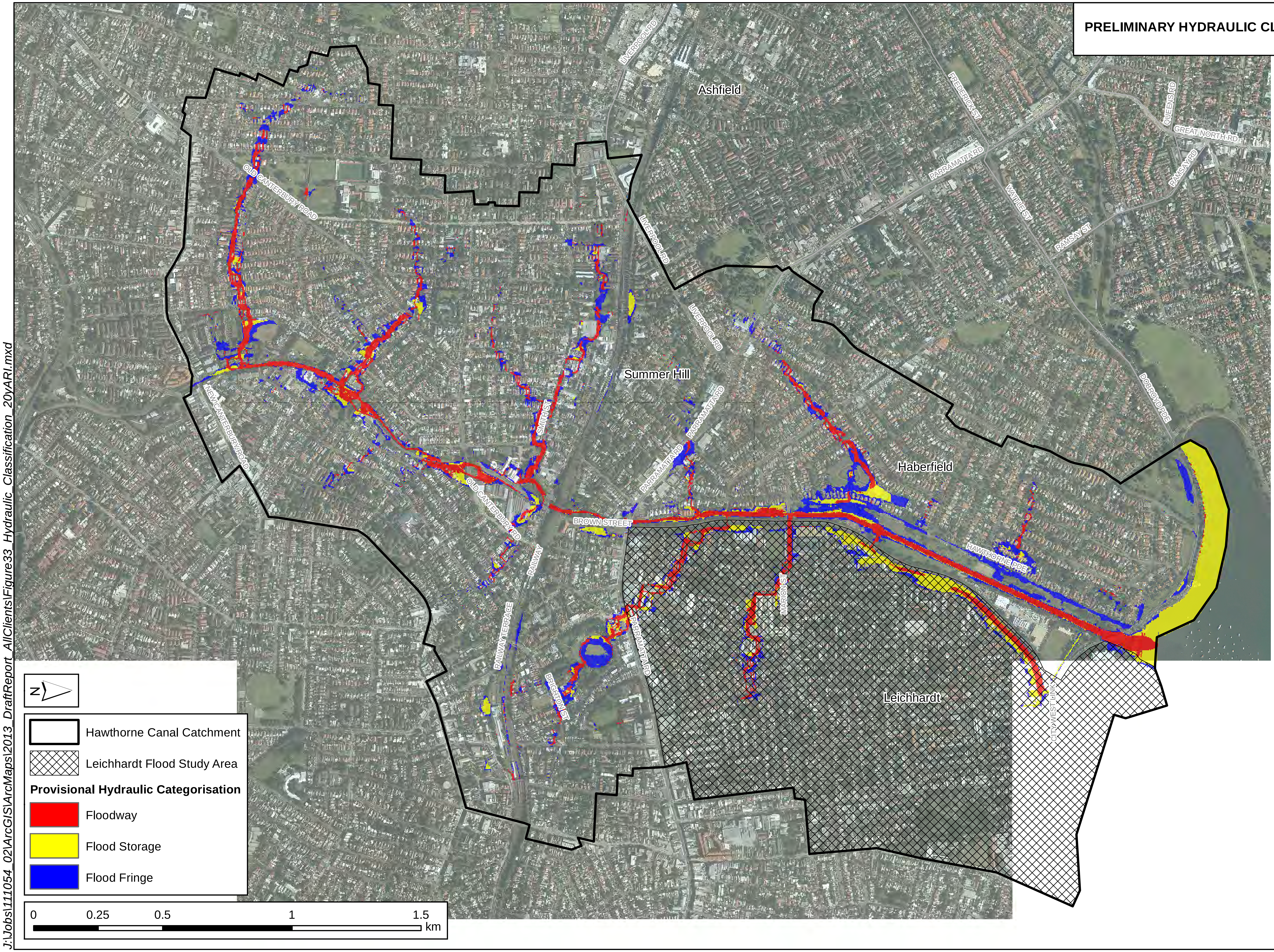

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FIGURE 34
PRELIMINARY HYDRAULIC CLASSIFICATION
100 YEAR ARI

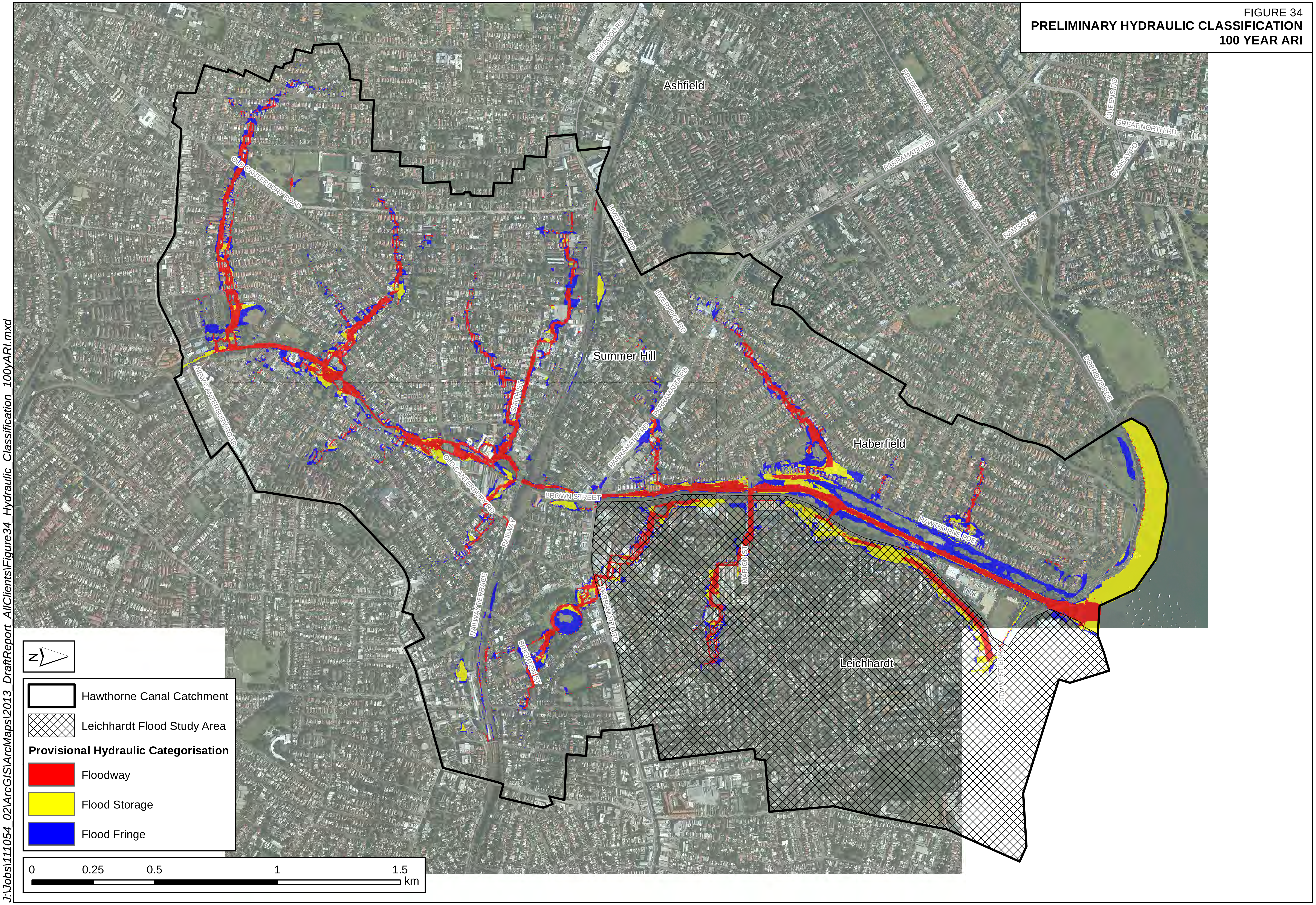

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FIGURE 35
PRELIMINARY HYDRAULIC CLASSIFICATION PMF

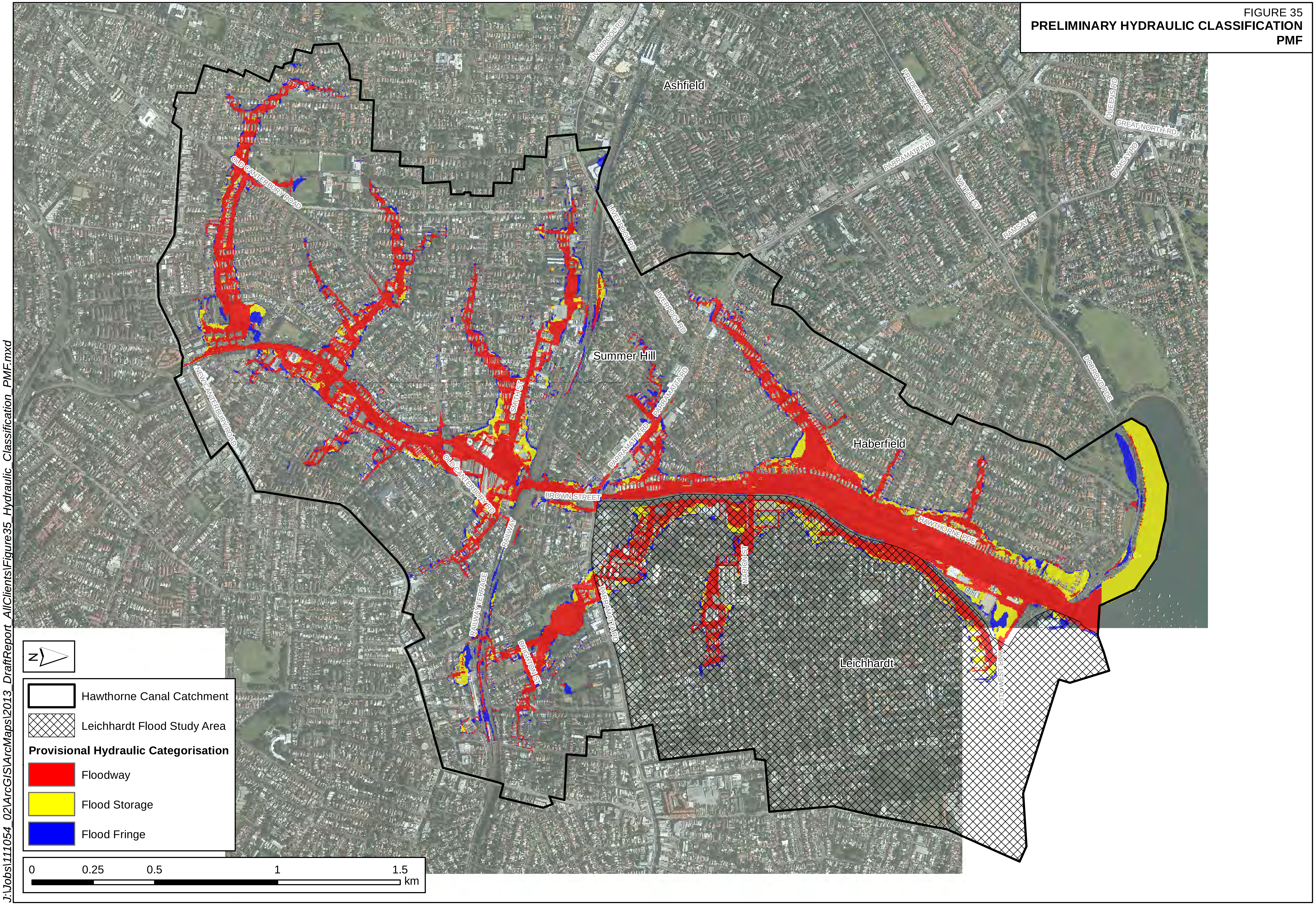

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FIGURE 36
PRELIMINARY HYDRAULIC CLASSIFICATION (HIGH/LOW RISK)
100 YEAR ARI

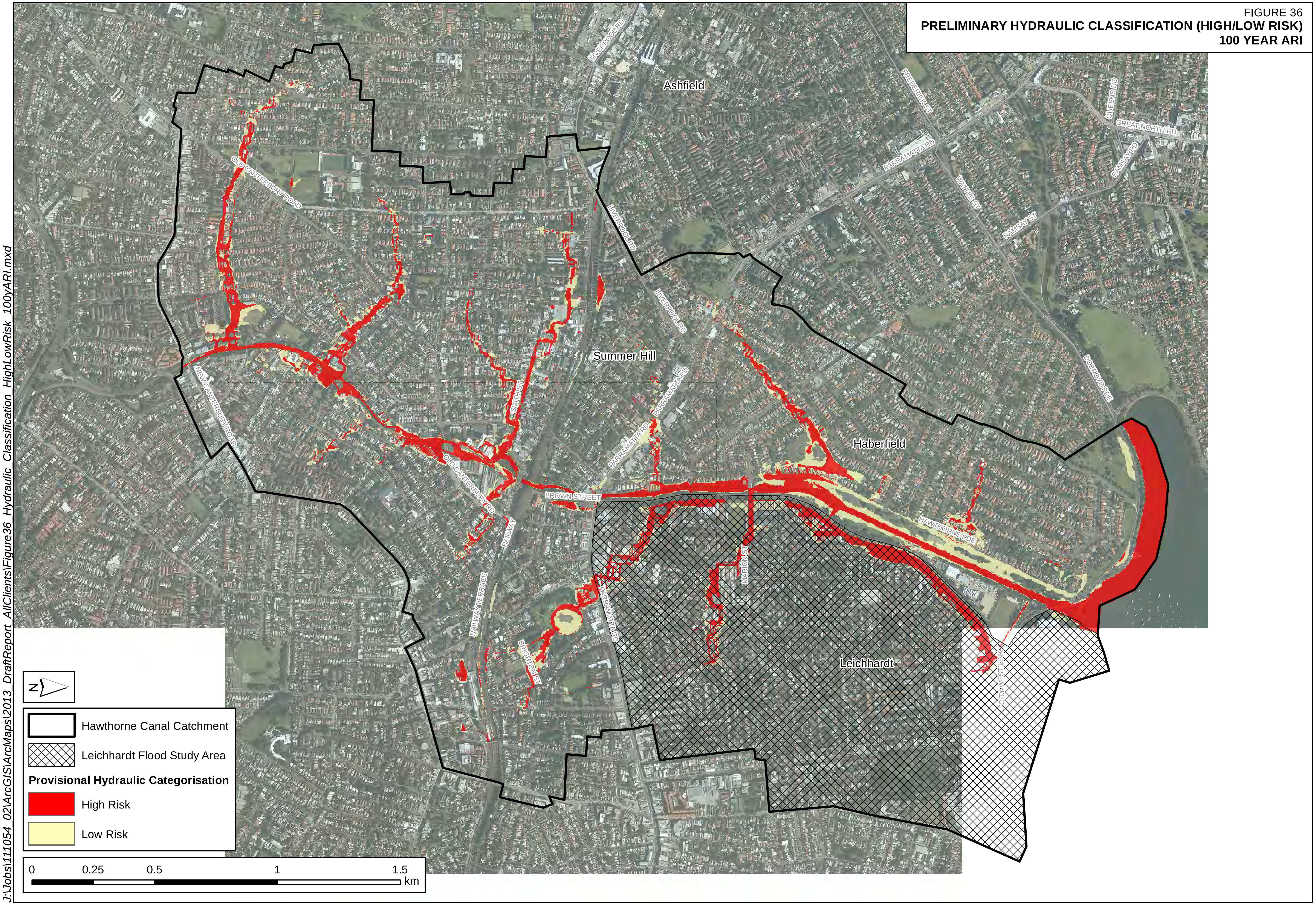

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FIGURE 37
PRELIMINARY FLOOD EMERGENCY RESPONSE CLASSIFICATION OF COMMUNITIES

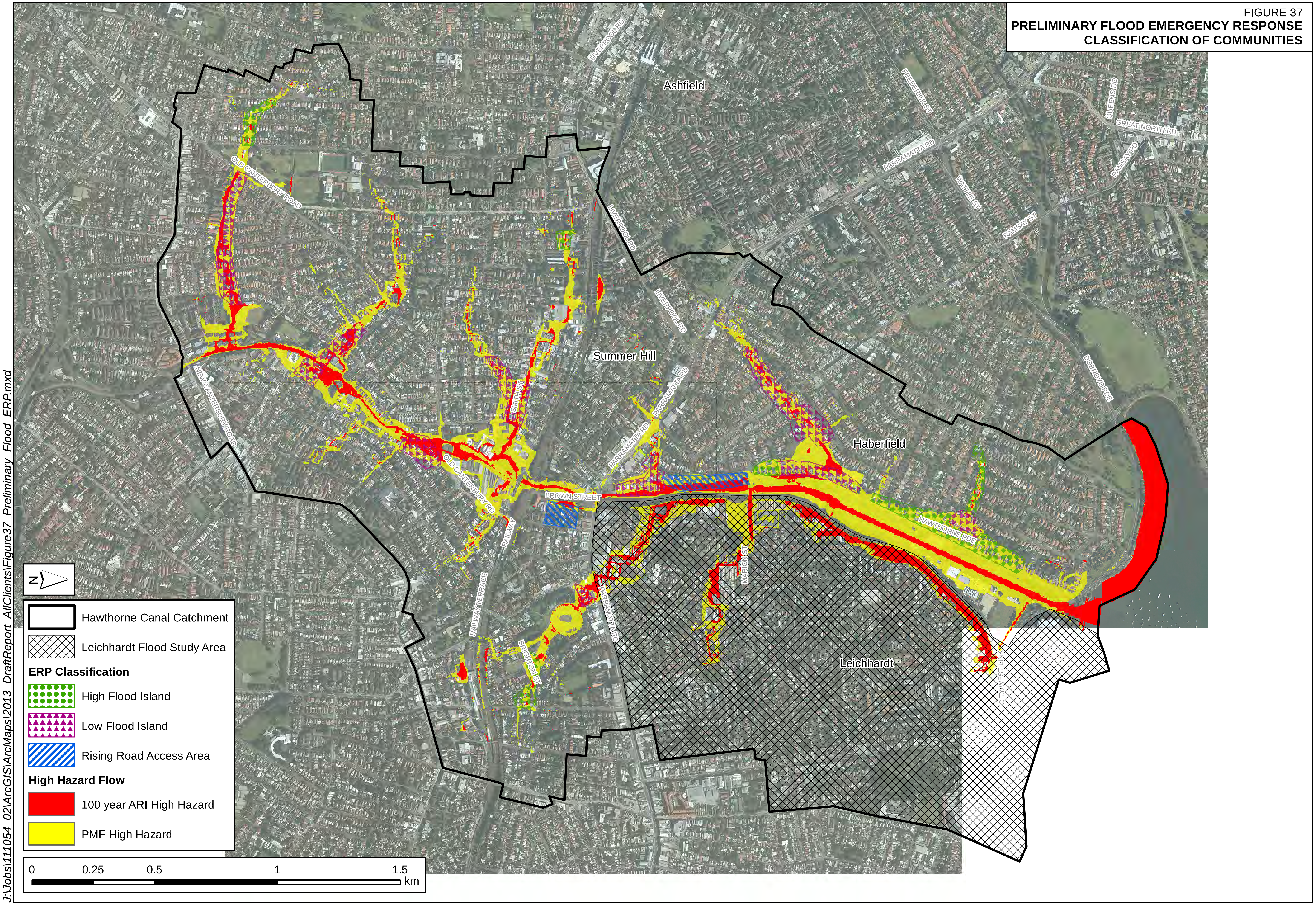

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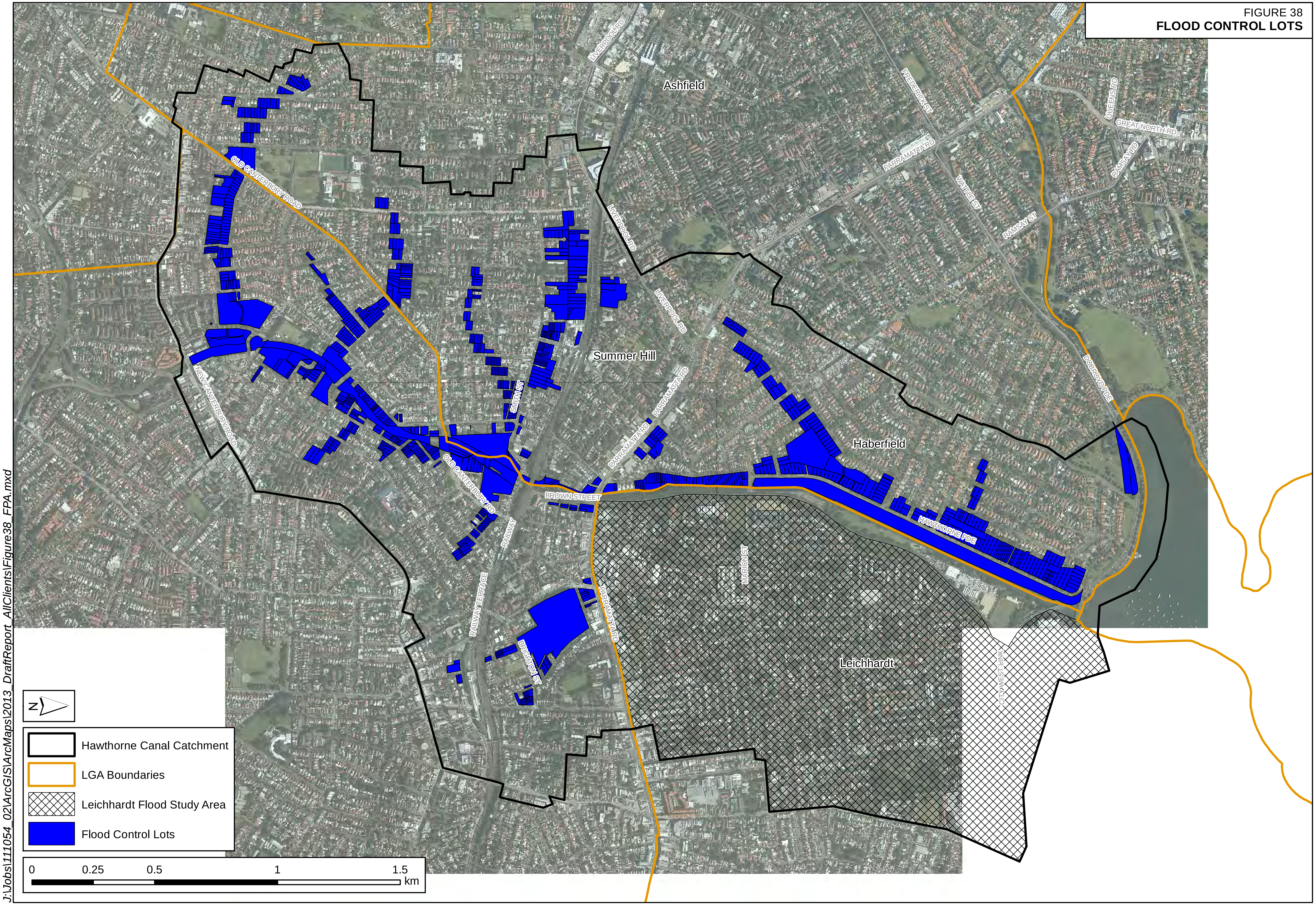
FIGURE 38
FLOOD CONTROL LOTS



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Appendix A

APPENDIX A: GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

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

redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.

disaster plan (DISPLAN) A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.

discharge The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).

ecologically sustainable development (ESD) Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.

effective warning time The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.

emergency management A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.

flash flooding Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.

flood Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.

flood awareness Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.

flood education Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.

flood fringe areas The remaining area of flood prone land after floodway and flood storage areas have been defined.

flood liable land Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).

flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the Aflood liable land@ concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL=s are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the Astandard flood event@ in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.

Those parts of the floodplain that are important for the temporary storage of

flood storage areas floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.

floodway areas Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.

freeboard Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.

habitable room **in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom.

in an industrial or commercial situation: an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.

hazard A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.

hydraulics Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.

hydrograph A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.

hydrology Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.

local overland flooding Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.

local drainage Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.

mainstream flooding Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.

major drainage Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves:

$ the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or

$ water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or

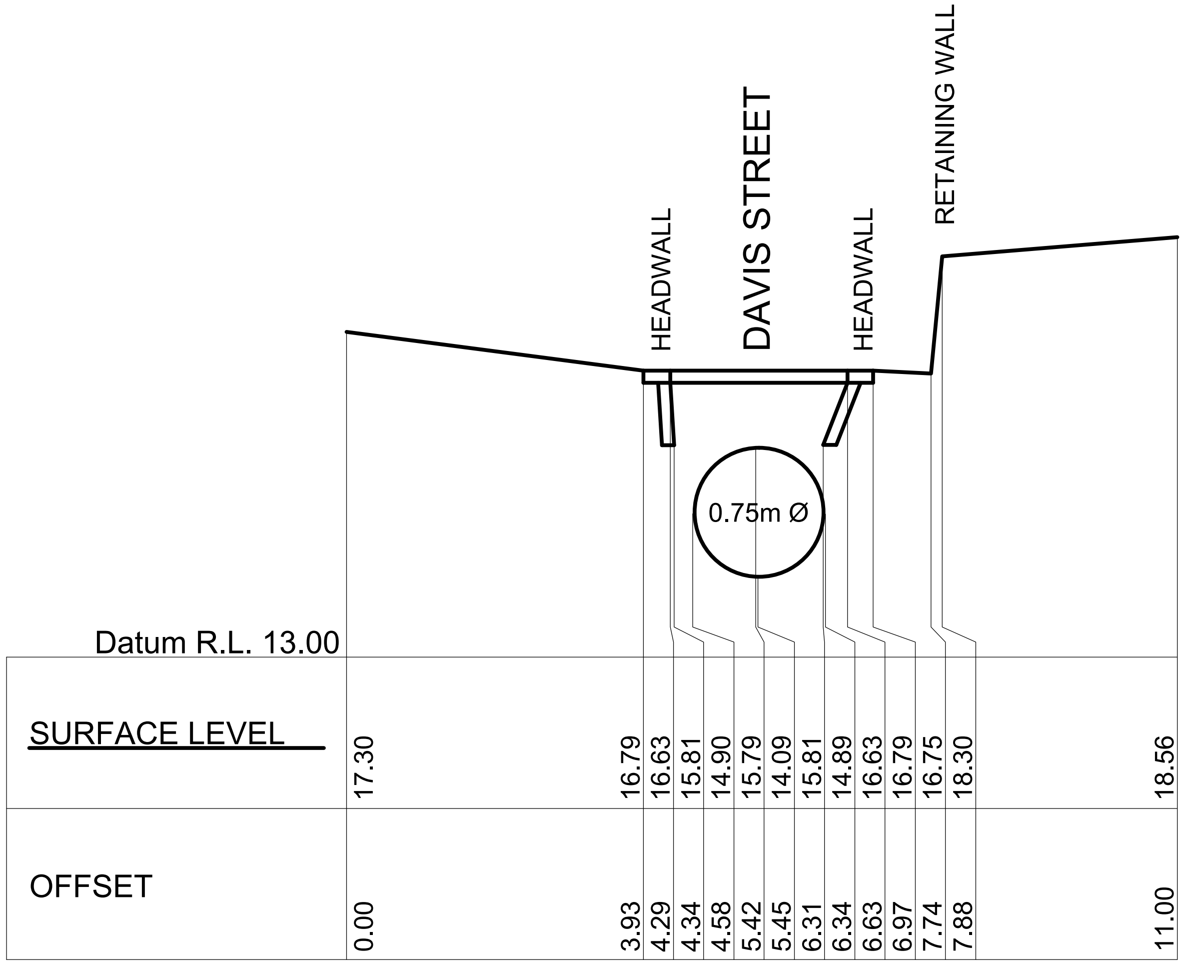
$ major overland flow paths through developed areas outside of defined drainage reserves; and/or

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

mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State=s rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.

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

Appendix B



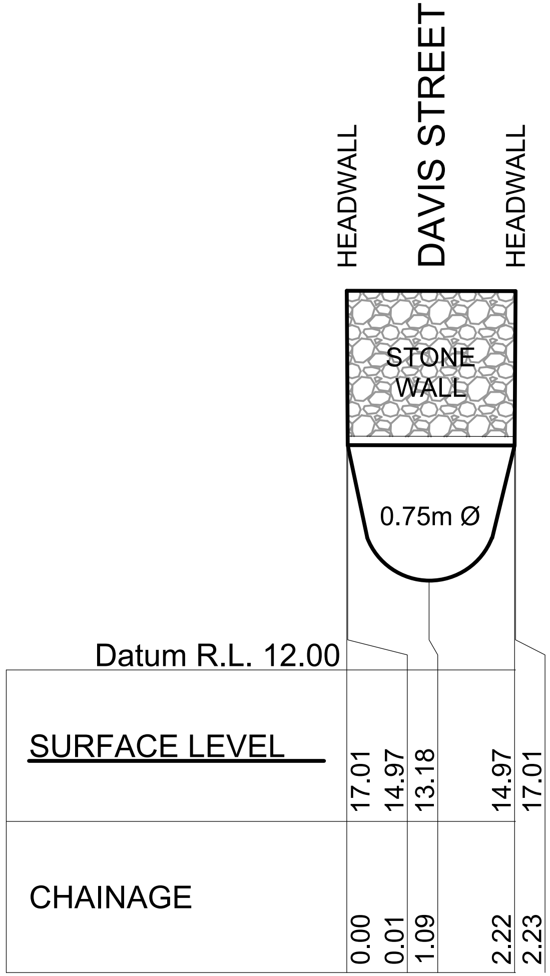
H_St001 (UPSTREAM END)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CULVERT CROSS SECTION H_St001
DAVIS STREET, DULWICH HILL

PIPE DIAMETER	1.95m
PIPE LENGTH	49.85m
UPSTREAM INVERT	RL 14.09
DOWNSTREAM INVERT	RL 13.18



H_St001 (DOWNSTREAM END)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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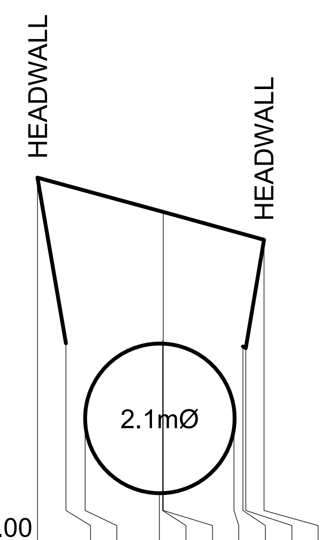
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CULVERT CROSS SECTION H_St001
DAVIS STREET, DULWICH HILL

PIPE DIAMETER	1.95m
PIPE LENGTH	49.85m
UPSTREAM INVERT	RL 14.09
DOWNSTREAM INVERT	RL 13.18



SURFACE LEVEL	16.80	14.61	13.65	12.63	14.64	16.35	13.74	14.57	14.54	15.98
CHAINAGE	0.00	0.38	0.63	1.61	1.66	1.66	2.60	2.72	2.76	3.00

H_St002 (UPSTREAM END)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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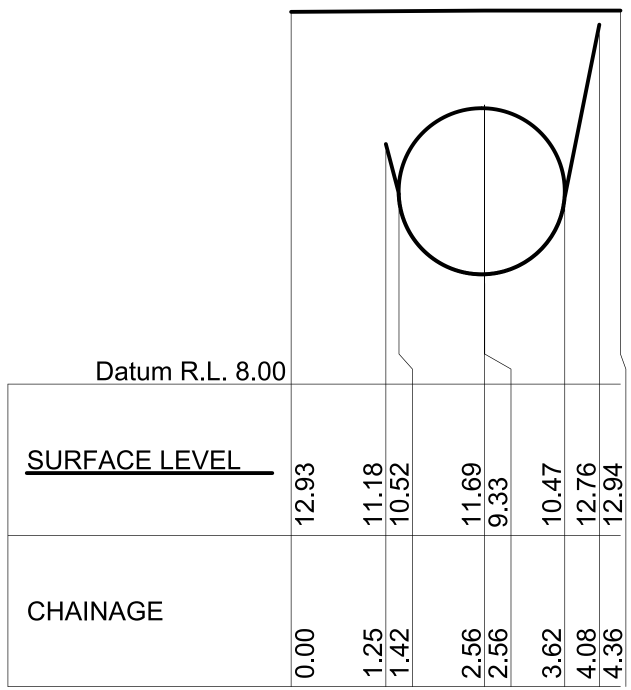
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CULVERT CROSS SECTION H_St002
RAILWAY LINE, DULWICH HILL

PIPE DIAMETER	2.1m
PIPE LENGTH	23.6m
UPSTREAM INVERT	RL 12.63
DOWNSTREAM INVERT	RL 11.88



H_St003 (UPSTREAM END)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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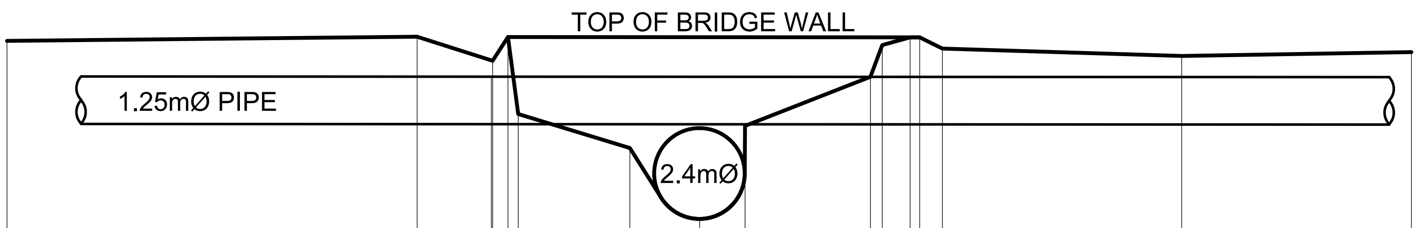
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CULVERT CROSS SECTION H_St003
RAILWAY LINE, DULWICH HILL

PIPE DIAMETER	2.1m
PIPE LENGTH	38.4m
UPSTREAM INVERT	RL 9.33
DOWNSTREAM INVERT	RL 8.95



SURFACE LEVEL	12.50	12.61	11.56	11.98	12.61	10.58	9.68	7.80	10.26	11.55	12.39	12.60	12.60	12.31	12.11	12.21
CHAINAGE	0.00	10.86	12.82	12.86	13.27	13.54	16.49	18.33	19.54	22.85	23.17	23.90	24.16	24.76	31.10	37.18

H_St004 (UPSTREAM END)

SCALE HORIZONTAL 1:200 VERTICAL 1:200

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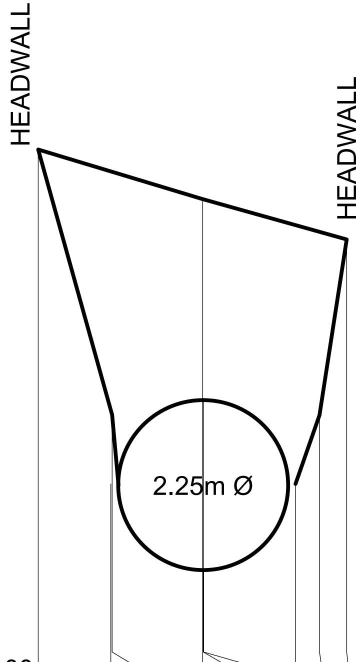
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
BRIDGE CROSS SECTION H_St004
OLD CANTERBURY ROAD, DULWICH HILL

PIPE DIAMETER	2.4m
PIPE LENGTH	51.9m
UPSTREAM INVERT	RL 7.801
DOWNSTREAM INVERT	RL 6.940



SURFACE LEVEL	11.05	6.62	7.54	10.39	7.72	5.49	6.62	7.53	9.86
CHAINAGE	0.00	0.96	0.98	2.18	2.18	2.19	3.40	3.72	4.08

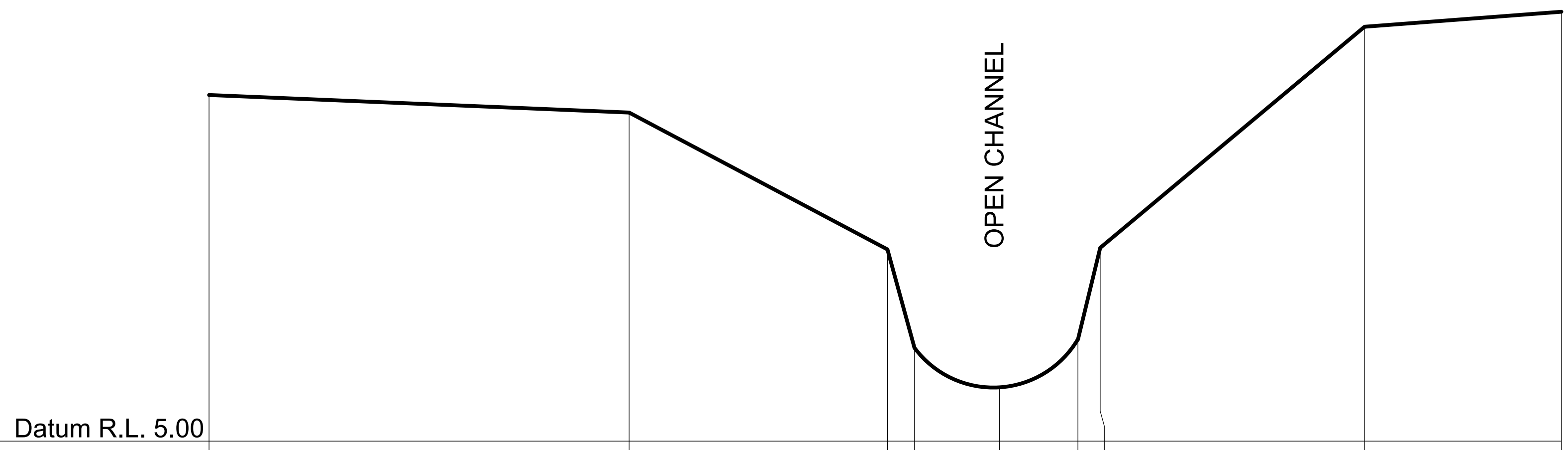
H_St005 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CULVERT CROSS SECTION H_St005
RAILWAY LINE, SUMMER HILL

PIPE DIAMETER	2.25m
PIPE LENGTH	80.92m
UPSTREAM INVERT	RL 5.490
DOWNSTREAM INVERT	RL 4.035



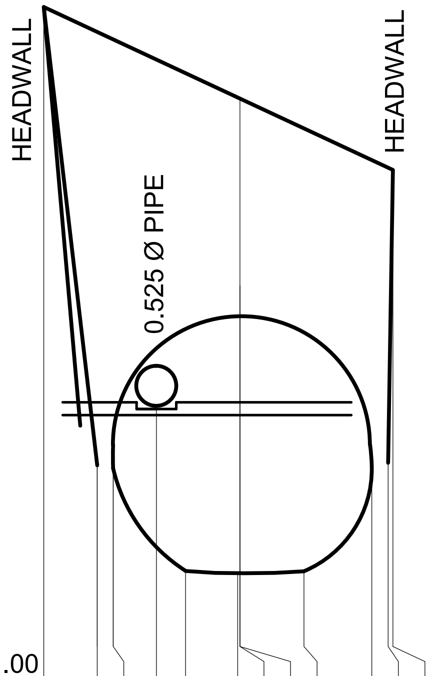
SURFACE LEVEL	9.60	9.36	7.55	6.24	5.71	6.35	7.57	10.50	10.70
CHAINAGE	0.00	5.59	9.02	9.38	10.51	11.55	11.85	15.36	17.97

H_St005 (10m UPSTREAM)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CROSS SECTION 10m UPSTREAM FROM H_St005
RAILWAY LINE, SUMMER HILL



SURFACE LEVEL	9.86	3.80	3.75	4.58	2.40	2.37	6.17	8.65	2.39	3.77	3.83	7.70
CHAINAGE	0.00	0.71	0.92	1.49	1.87	2.56	2.59	2.60	3.44	4.34	4.55	4.62

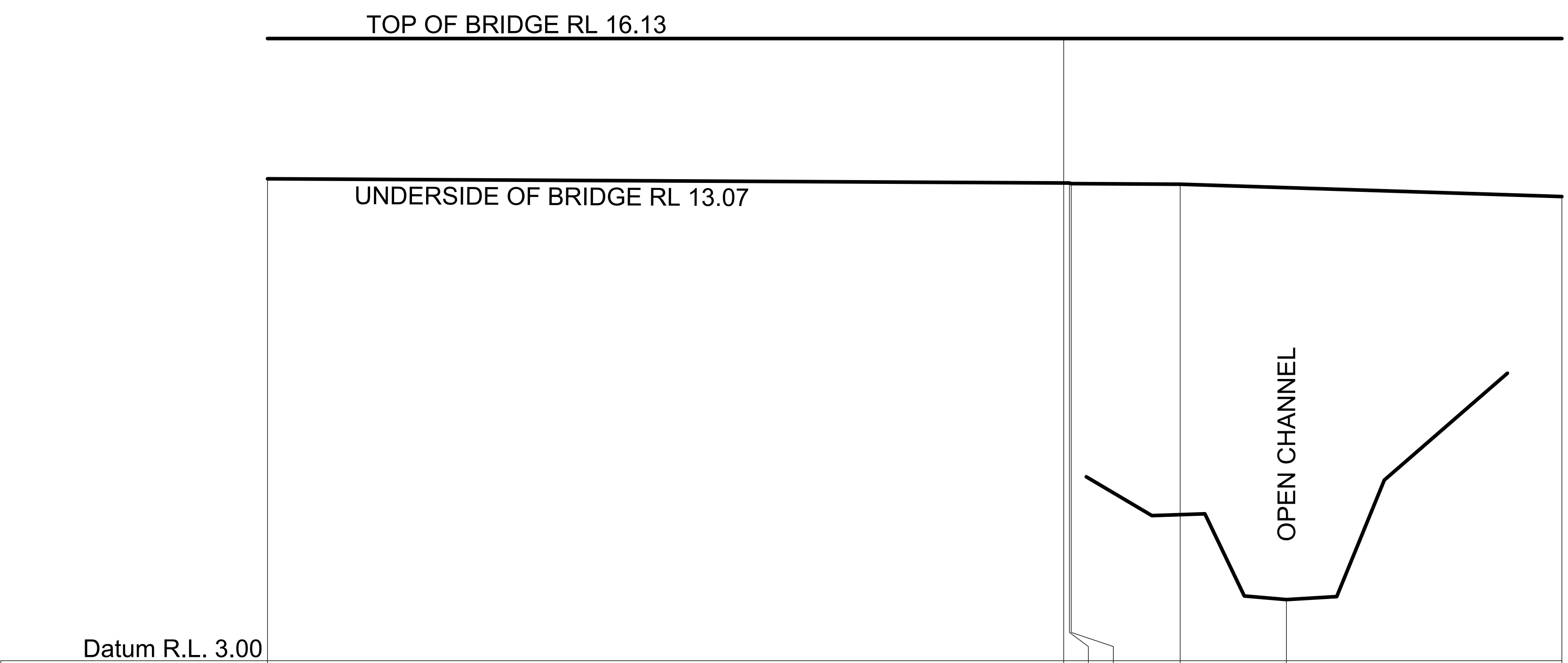
H_St006 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CULVERT CROSS SECTION H_St006
LONGPORT ROAD, SUMMER HILL

PIPE DIAMETER	3.4m
PIPE LENGTH	39.81m
UPSTREAM INVERT	RL 2.367
DOWNSTREAM INVERT	RL 1.972



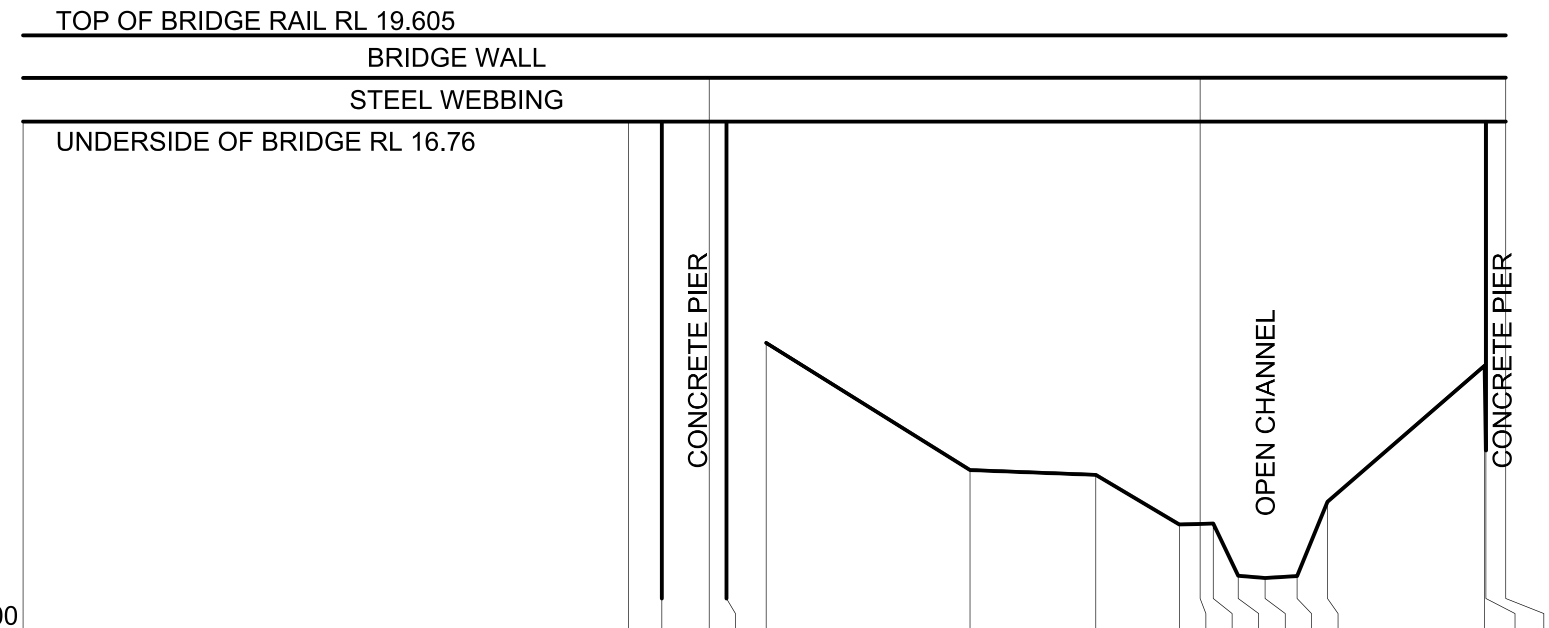
SURFACE LEVEL	13.17	16.13	13.08	13.07	13.05	4.29	12.80
CHAINAGE	1.39	18.19	18.31	18.35	20.65	22.89	28.70

H_St007 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
VIADUCT CROSS SECTION H_St007
RAILWAY LINE, LEWISHAM



Datum R.L. 0.00

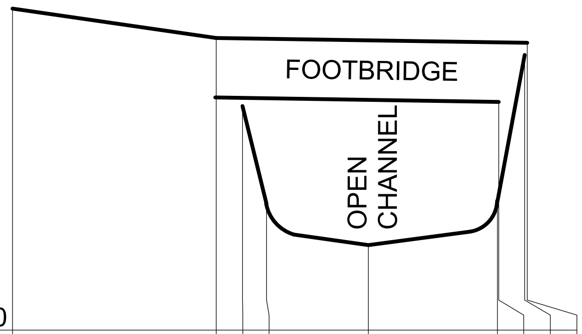
SURFACE LEVEL	16.76	16.76	12.19	18.20	10.14	9.44	5.24	5.09	3.44	18.21	3.48	1.75	1.67	1.73	4.19	8.36	5.89	18.21
CHAINAGE	0.00	20.02	21.13	22.69	23.26	24.57	31.32	35.47	38.24	38.93	39.36	40.19	41.08	42.14	43.14	48.34	48.38	49.04

H_St008 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:250 VERTICAL 1:250

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
BRIDGE CROSS SECTION H_St008
RAILWAY LINE, LEWISHAM



Datum R.L. 0.00

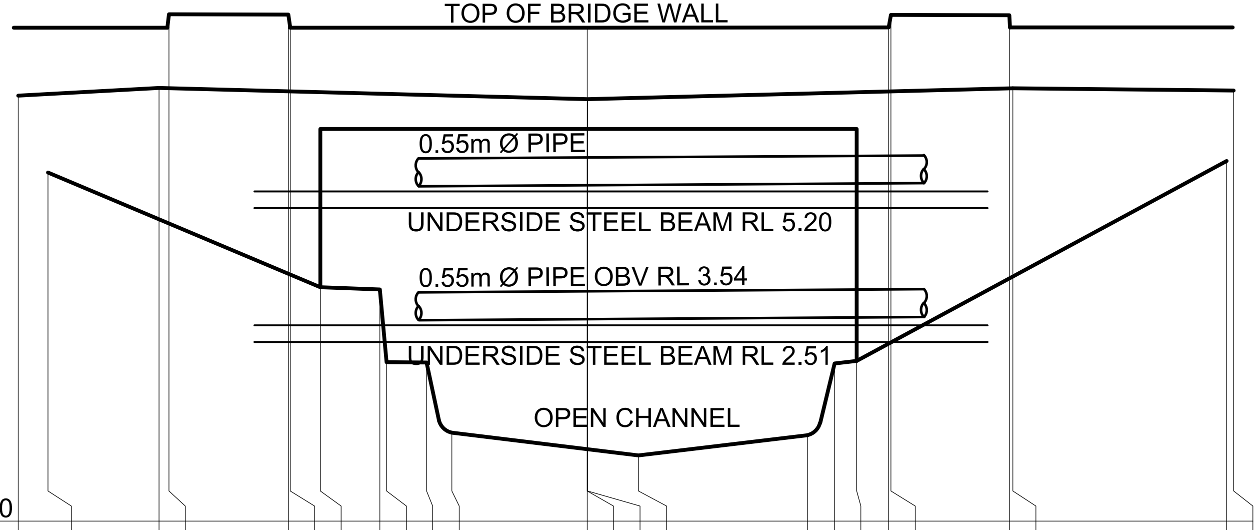
SURFACE LEVEL	4.26	3.87	2.97	1.67	1.13	1.71	3.02	3.65	3.81	
CHAINAGE	0.00	2.69	3.04	3.36	4.70	6.41	6.43	6.77	6.81	

H_St009 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
FOOTBRIDGE BRIDGE CROSS SECTION H_St009
GROSVENOR ROAD, LEWISHAM



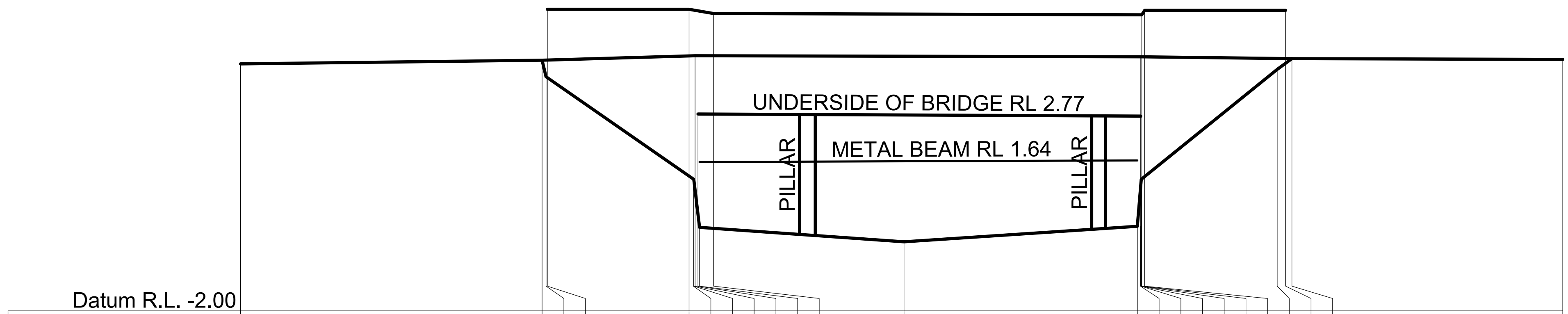
SURFACE LEVEL	7.44	5.91	7.59	9.05	9.04	8.78	3.63	3.60	2.15	2.15	0.75	6.77	8.78	7.37	0.31	0.70	2.13	2.18	8.79	9.04	9.03	7.58	6.14	7.54
CHAINAGE	0.00	0.59	2.79	2.99	5.36	5.40	6.00	7.18	7.31	8.10	8.60	11.29	11.29	11.30	12.31	15.66	16.20	16.64	17.27	17.32	19.67	19.74	23.98	24.12

H_St011 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
BRIDGE CROSS SECTION H_St011
PARRAMATTA ROAD, HABERFIELD



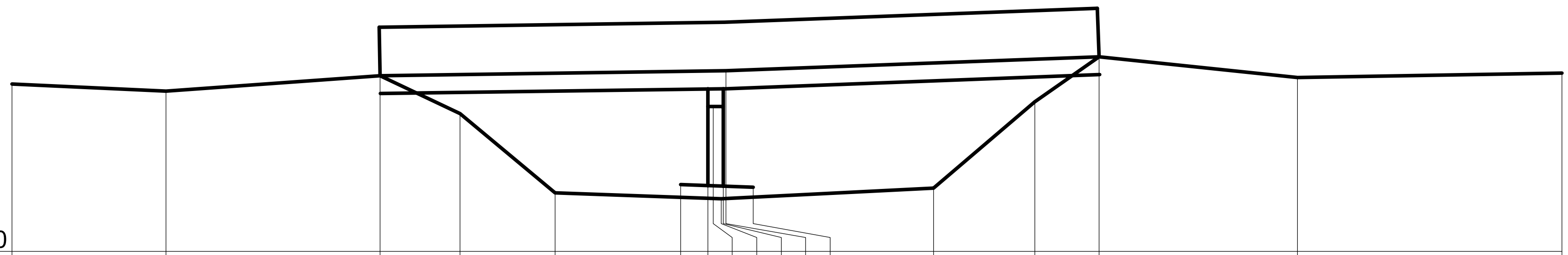
SURFACE LEVEL	3.97	4.07	3.66	5.30	5.30	1.18	1.18	4.18	2.77	0.02	5.20	-0.33	0.04	0.05	2.72	4.15	1.18	5.16	5.27	3.85	5.27	4.11	4.09
CHAINAGE	0.00	7.30	7.40	7.43	10.86	10.95	10.97	11.00	11.07	11.11	11.45	16.06	21.70	21.78	21.78	21.79	21.80	21.81	21.88	25.09	25.29	25.44	32.00

H_St012 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
BRIDGE CROSS SECTION H_St012
MARION STREET, HABERFIELD



Datum R.L. -2.00

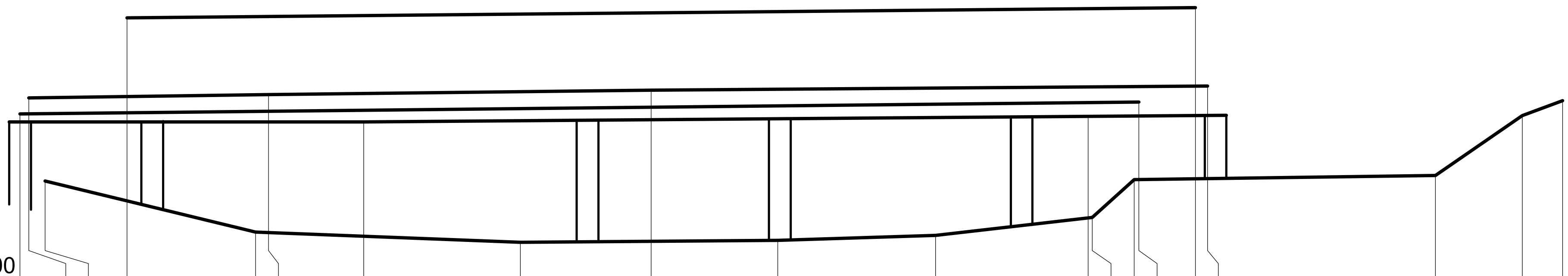
SURFACE LEVEL	1.59	1.44	1.78	0.96	-0.74	-0.56	1.11	1.11	-0.87	1.12	1.88	-0.62	-0.64	1.22	2.18	1.74	1.83
CHAINAGE	0.00	3.30	7.89	9.61	11.64	14.34	14.92	15.04	15.21	15.25	15.31	15.89	19.76	21.94	23.31	27.57	33.24

H_St013 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
FOOTBRIDGE CROSS SECTION H_St013
HAWTHORNE PARADE, HABERFIELD



Datum R.L. -3.00

SURFACE LEVEL	4.48	5.23	1.39	8.90	-0.95	5.38	4.11	-1.43	5.58	-1.33	-1.11	4.37	-0.28	1.46	5.03	9.37	5.76	1.65	4.41	5.09
CHAINAGE	0.00	0.40	1.15	4.92	10.84	11.44	15.82	23.04	29.06	34.88	42.15	49.18	49.36	51.30	51.52	54.12	54.68	65.17	69.16	71.02

H_St014 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:300 VERTICAL 1:300

SURVEYING | CIVIL | DEVELOPMENT

Erina: (02) 4367 7334

Hornsby: (02) 9482 9498

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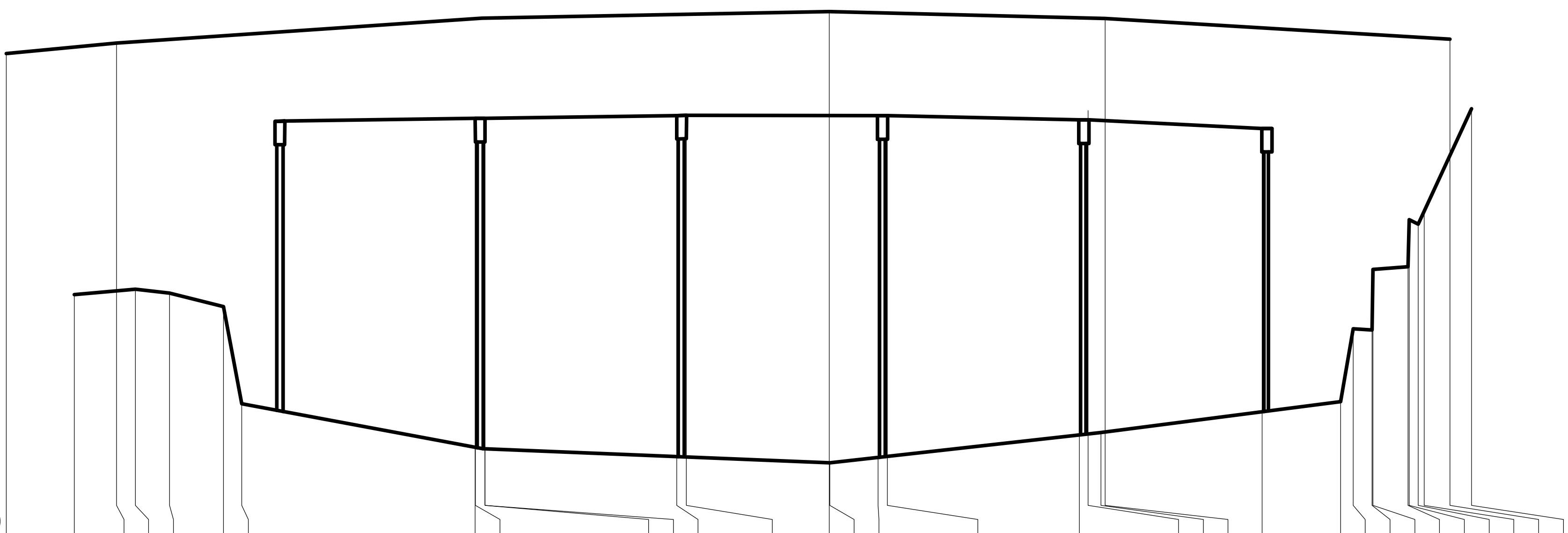
CHASE BURKE HARVEY

HAWTHORNE CHANNEL

BRIDGE CROSS SECTION H_St014

M4 CITY WEST LINK, HABERFIELD



Datum R.L. -2.00

CHAINAGE	SURFACE LEVEL
0.00	4.78
4.81	1.37
7.79	4.93
9.12	1.45
11.53	1.40
15.35	1.21
16.64	-0.17
33.14	3.86
33.14	3.53
33.82	3.56
33.83	3.87
47.38	3.90
48.04	3.91
58.16	5.37
58.20	-1.00
61.60	3.90
62.26	3.90
75.83	3.84
76.45	3.97
77.36	-0.57
77.64	5.28
88.75	3.72
94.28	-0.14
95.18	0.89
96.51	0.88
96.58	1.73
99.04	1.77
99.13	2.43
99.77	2.37
100.19	2.54
102.00	4.98
103.54	4.00

H_St015 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:500 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT

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Hornsby: (02) 9482 9498

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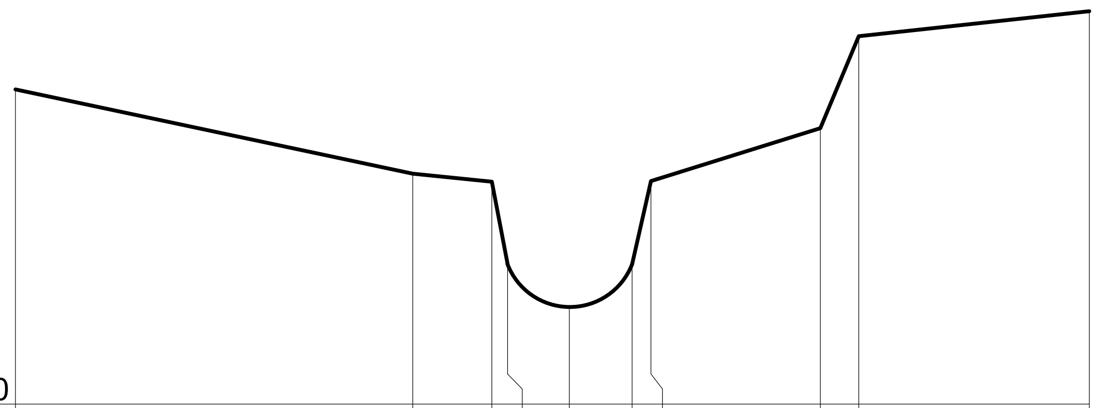
CHASE BURKE HARVEY

HAWTHORNE CHANNEL

BRIDGE CROSS SECTION H_St015

LILYFIELD ROAD, HABERFIELD



Datum R.L. 13.00

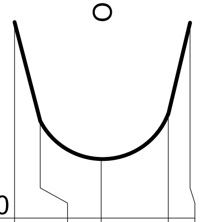
SURFACE LEVEL	17.16	16.05	15.94	14.85	14.28	14.85	15.95	16.64	17.86	18.19
CHAINAGE	0.00	5.25	6.30	6.50	7.32	8.15	8.40	10.64	11.15	14.20

H_CS001

SCALE HORIZONTAL 1:100 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

CROSS SECTION H_CS001
NEAR DAVIS STREET, DULWICH HILL
HAWTHORNE CHANNEL



Datum R.L. 12.00

SURFACE LEVEL	14.59	13.28	12.78	13.37	14.58
CHAINAGE	0.00	0.34	1.14	2.03	2.32

H_CS002

SCALE HORIZONTAL 1:100 VERTICAL 1:100

SURVEYING | CIVIL | DEVELOPMENT
Erina: (02) 4367 7334
Hornsby: (02) 9482 9498
www.cbhsurvey.com.au
CHASE BURKE HARVEY

HAWTHORNE CHANNEL
CROSS SECTION H_CS002
NEAR RAILWAY CROSSING, DULWICH HILL

Appendix C

Ashfield Council

HAWTHORNE and DOBROYD CANAL

FLOOD STUDIES

Ashfield Council is carrying out flood studies within its local government area. The flood studies are carried out under the NSW Floodplain Risk Management Program, which discharges Councils responsibility for management of flood risk under the Local Government Act 1993 (Section 733).

Two separate catchments within Ashfield are being investigated, these being the Hawthorne Canal and Dobroyd Canal catchments (see map). The other Councils that share the catchments with Ashfield Council are Burwood Council in the Dobroyd Canal catchment to the west and Marrickville Council in the Hawthorne Canal catchment to the east. Both Burwood Council and Marrickville Council are engaged in the present flood studies with Ashfield Council.

The primary objective of the Policy is to reduce the impact of flooding and flood liability on owners and occupants of flood prone land and to reduce losses from flooding. The Policy provides for technical and financial support by the Government through four sequential stages:

1. Flood Study

- Determine the nature and extent of flood problem

2. Floodplain Risk Management

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

3. Floodplain Risk Management Plan

- Formal adoption by Council of a plan of management for the floodplain

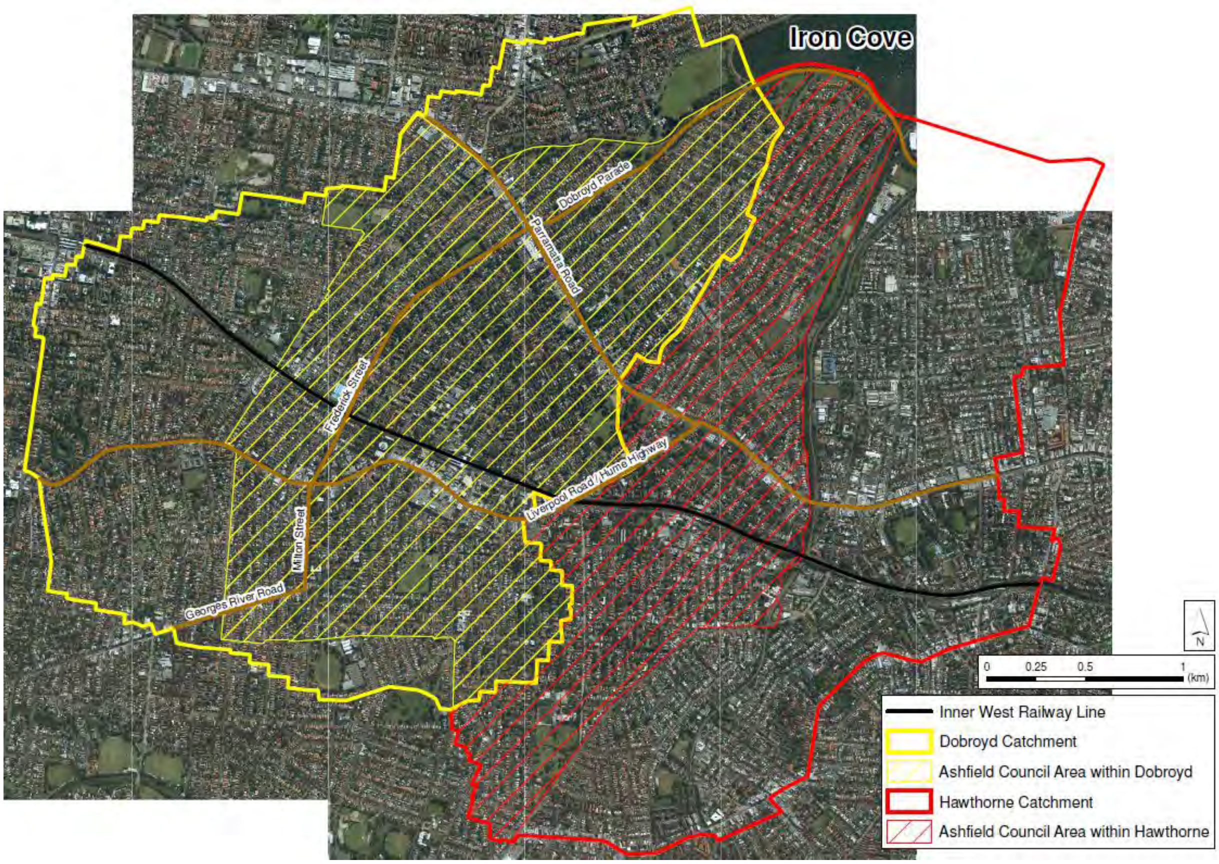
4. Implementation of the Plan

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

The flood studies just started will define the flood behaviour over a range of flood magnitudes within the two catchments. As part of the flood studies, computer models describing the flooding behaviour will be built. In order to establish the accuracy of such models, observations from the public on observed flooding behaviour are sought.

WMAwater is carrying out the study for Council and would like information about your experiences of flooding. Please return the completed questionnaire before 30/06/2012 by:

- **Prepaid self-addressed envelope provided** *or*
- Fax to 9262 6208
- Scan and email to gray@wmawater.com.au



If you have any photographs of flooding in your area, please email them to gray@wmawater.com.au or include them with the questionnaire in the prepaid envelope. All photos will be copied and returned.

Your Name: ______________________ Tel No: ______________
Property Address: ______________________ E-Mail: ______________

☐ Residential Property ☐ Non-Residential Property

Flood Information

How long have you lived or worked at this address? _____ years

If you have experienced any flood events, please specify below.

Date of Event	__/__/__	__/__/__	__/__/__
Was the water above the floor level?	House ☐ Other Buildings ☐	House ☐ Other Buildings ☐	House ☐ Other Buildings ☐
What level did the floodwater reach on the rest of this or other properties? (see examples)			

What other floods have you experienced? ____

HAWTHORNE CANAL FLOOD STUDY

QUESTIONNAIRE

Marrickville Council is carrying out a drainage and flood study for the Hawthorne Canal catchment within Council's local government area. The purpose of this study is to determine where flooding occurs, and to what extent, so that Council can identify strategies to reduce the impact of flooding in the local area. This study will ensure future flood management planning for Marrickville is based on accurate information.

WMAwater is carrying out the study for Council and would like information about your experiences of flooding. Please return the completed questionnaire before 31/07/2012 by:

- **Prepaid self-addressed envelope provided** *or*
- Fax to 9262 6208
- Scan and email to gray@wmawater.com.au

If you have any photographs of flooding in your area, please email them to gray@wmawater.com.au or include them with the questionnaire in the prepaid envelope. All photos will be copied and returned.

Your Name: ______________________________ Tel No: ________________
Property Address: ______________________________ E-Mail: ________________

☐ Residential Property ☐ Non-Residential Property

Flood Information

How long have you lived or worked at this address? _____ years

Have you experienced any of the following flood events?

Date of Event	__/__/__	__/__/__	__/__/__
Was the water above the floor level?	House ☐ Other Buildings ☐	House ☐ Other Buildings ☐	House ☐ Other Buildings ☐
What level did the floodwater reach on the rest of this or other properties? (see examples)			

What other floods have you experienced? ____________________________________

Appendix D

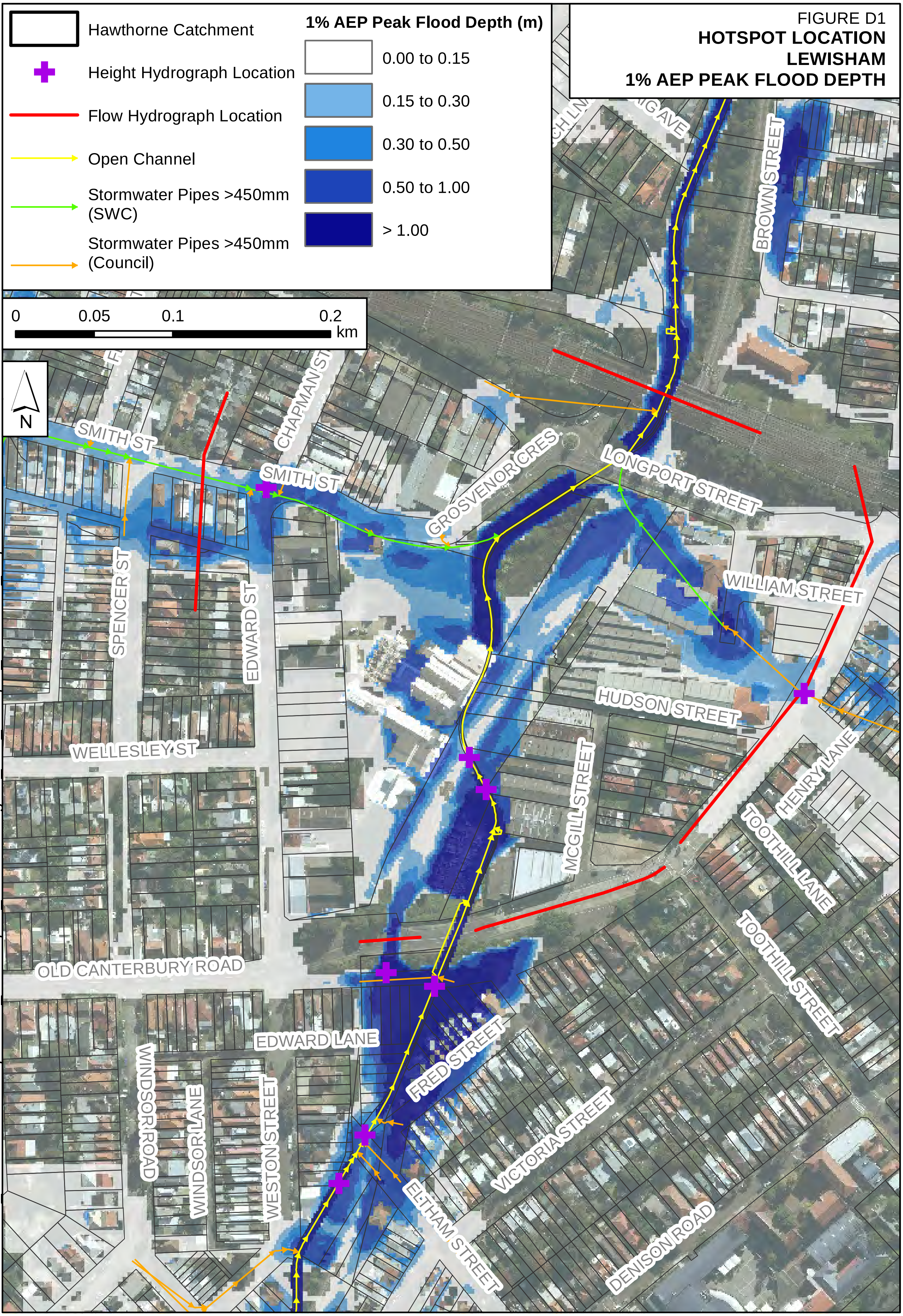


FIGURE D1
HOTSPOT LOCATION
LEWISHAM
1% AEP PEAK FLOOD DEPTH

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FIGURE D2
HOTSPOT LOCATION
GROSVENOR CRESCENT
1% AEP PEAK FLOOD DEPTH

Hawthorne Catchment

Height Hydrograph Location

Flow Hydrograph Location

Open Channel

Stormwater Pipes >450mm (SWC)

Stormwater Pipes >450mm (Ashfield Council)

1% AEP Peak Flood Depth (m)

0.00 to 0.15

0.15 to 0.30

0.30 to 0.50

0.50 to 1.00

> 1.00

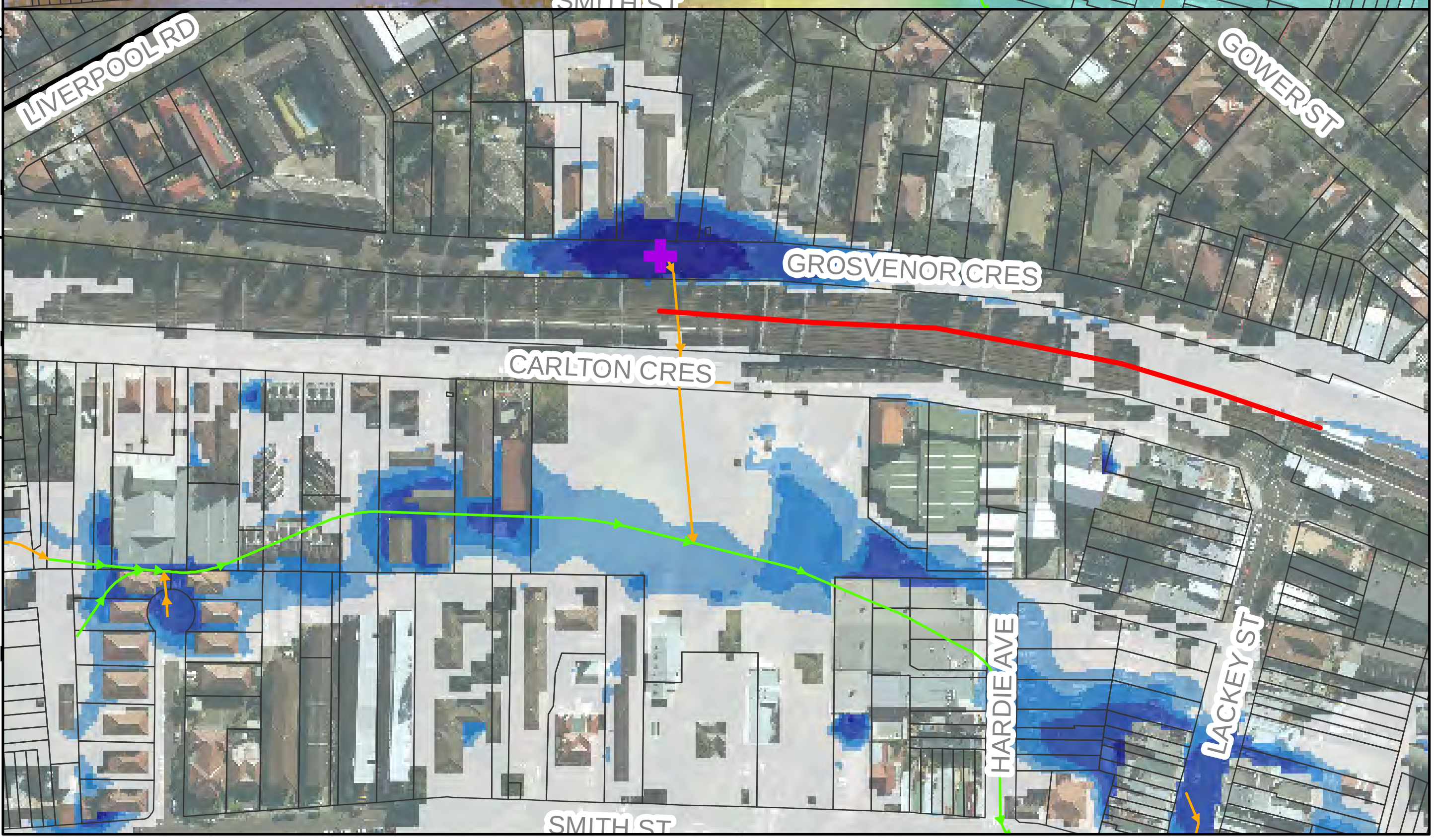
Ground Elevation (mAHD)

High : 40

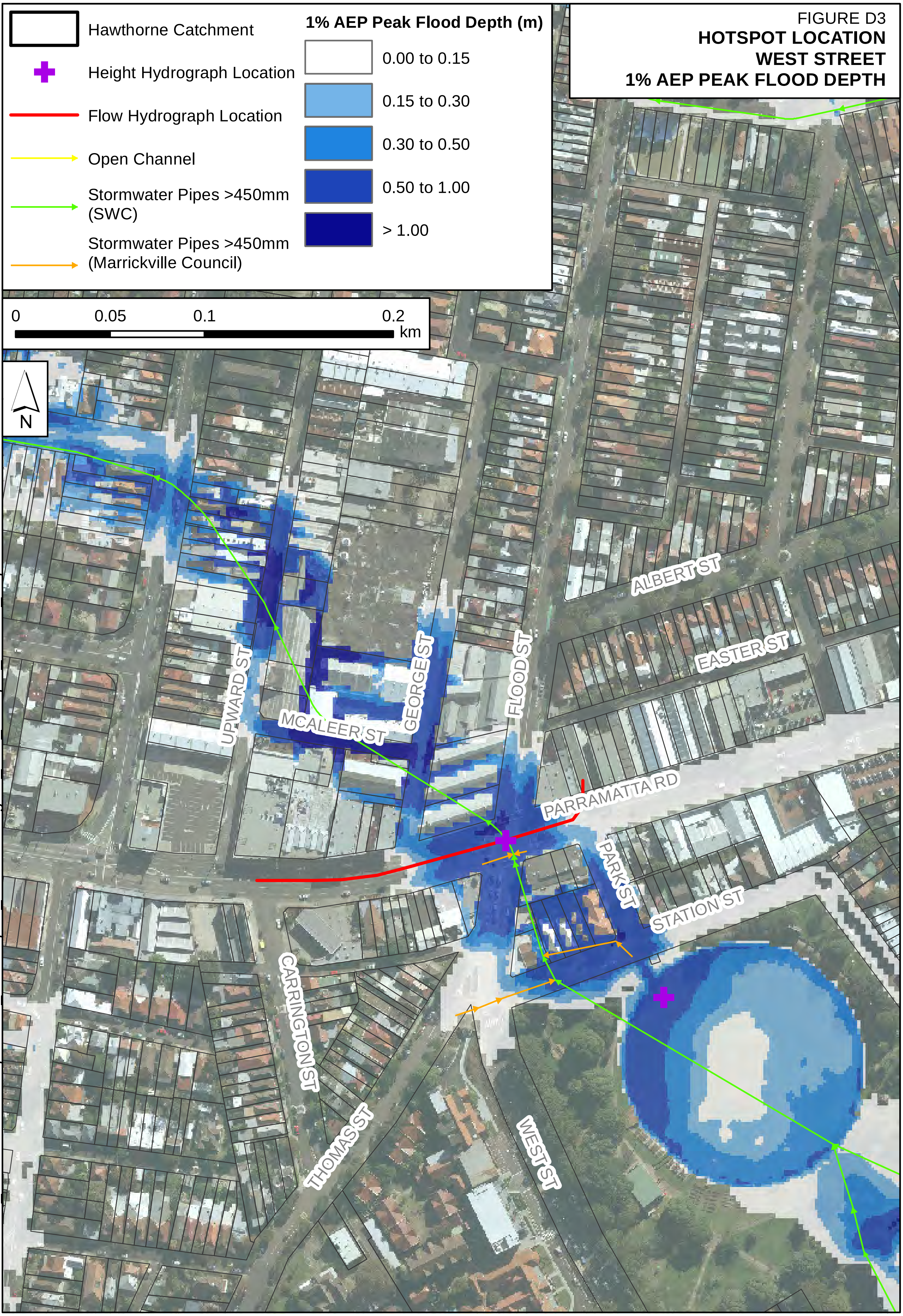
Low : 20

N

0 0.05 0.1 0.2 km



J:\Jobs\111054 02\ArcGIS\ArcMaps\2013 DraftReport AllClients\Figure D02 HotSpot GrosvenorCr Depth.mxd



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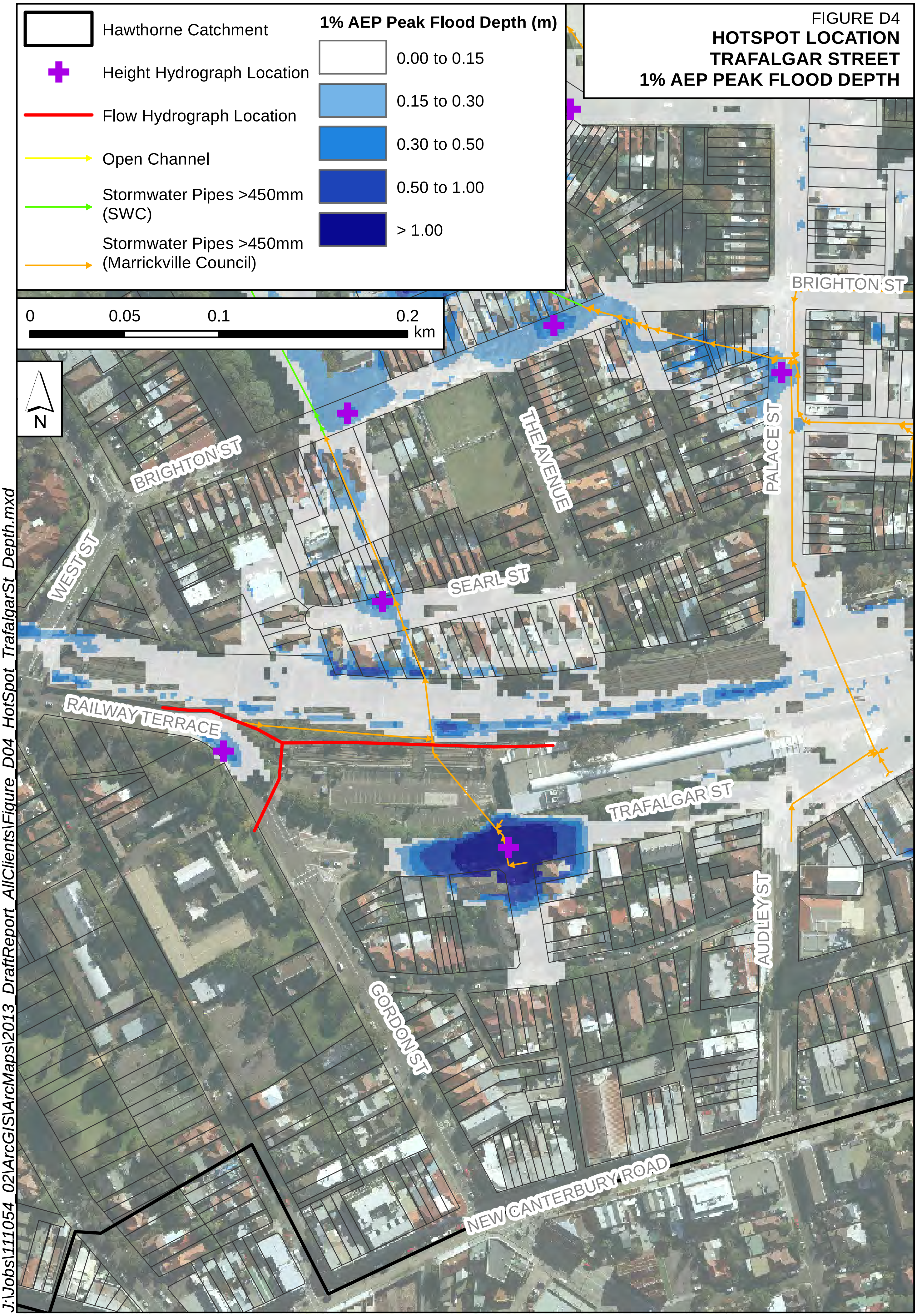
FIGURE D4
HOTSPOT LOCATION
TRAFALGAR STREET
1% AEP PEAK FLOOD DEPTH
Hawthorne Catchment
Height Hydrograph Location
Flow Hydrograph Location
Open Channel
Stormwater Pipes >450mm (SWC)
Stormwater Pipes >450mm (Marrickville Council)
1% AEP Peak Flood Depth (m)
0.00 to 0.15
0.15 to 0.30
0.30 to 0.50
0.50 to 1.00
> 1.00
0
0.05
0.1
0.2
km
N
BRIGHTON ST
BRIGHTON ST
WEST ST
THE AVENUE
PALACE ST
SEARL ST
RAILWAY TERRACE
TRAFALGAR ST
AUDLEY ST
GORDON ST
NEW CANTERBURY ROAD
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FIGURE D5
HOTSPOT LOCATION
LIGHT RAIL
1% AEP PEAK FLOOD DEPTH

Hawthorne Catchment

Height Hydrograph Location

Flow Hydrograph Location

Open Channel

Stormwater Pipes >450mm (SWC)

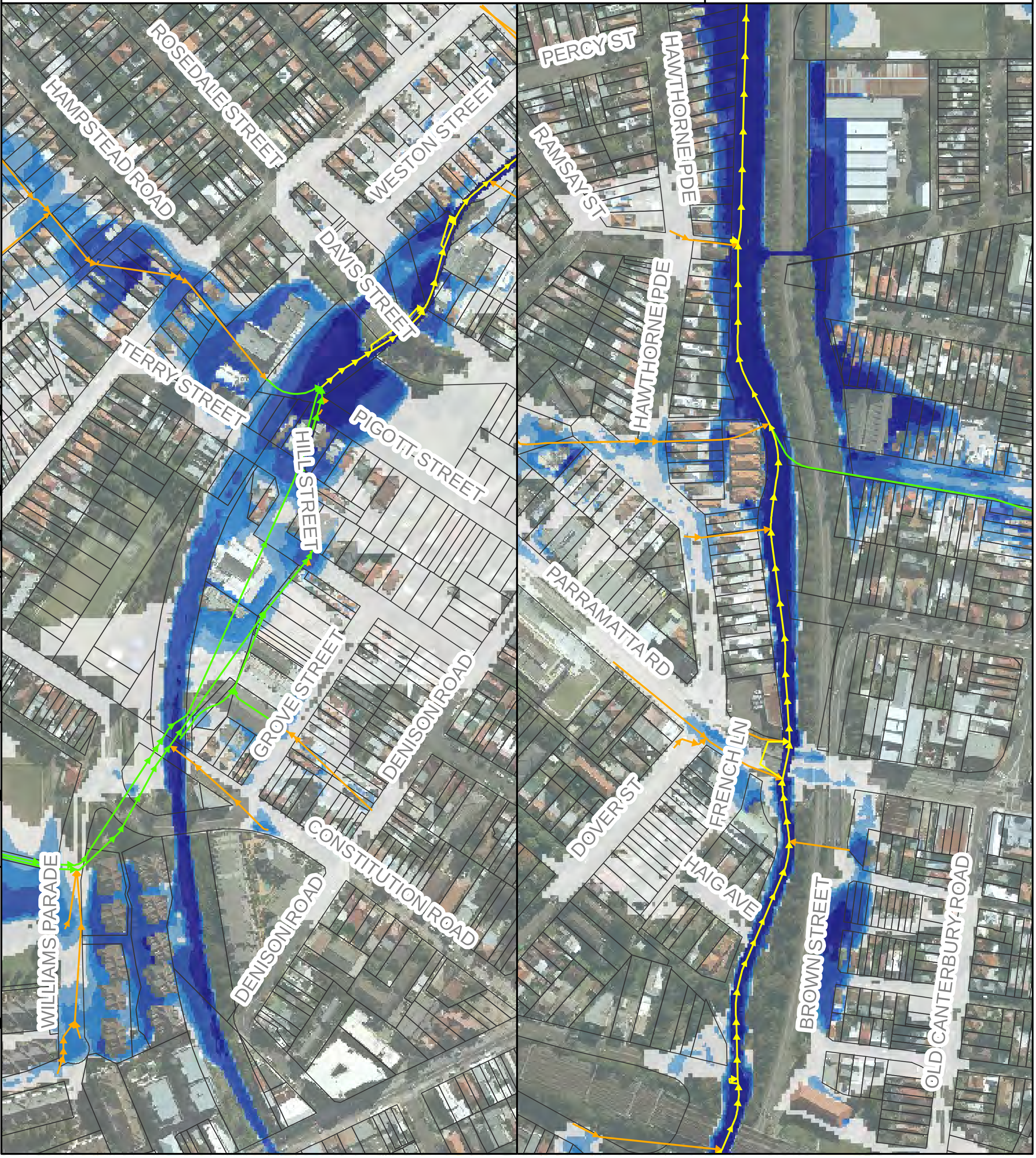
Stormwater Pipes >450mm (Council)

1% AEP Peak Flood Depth (m)

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

N

0 0.05 0.1 0.2 km



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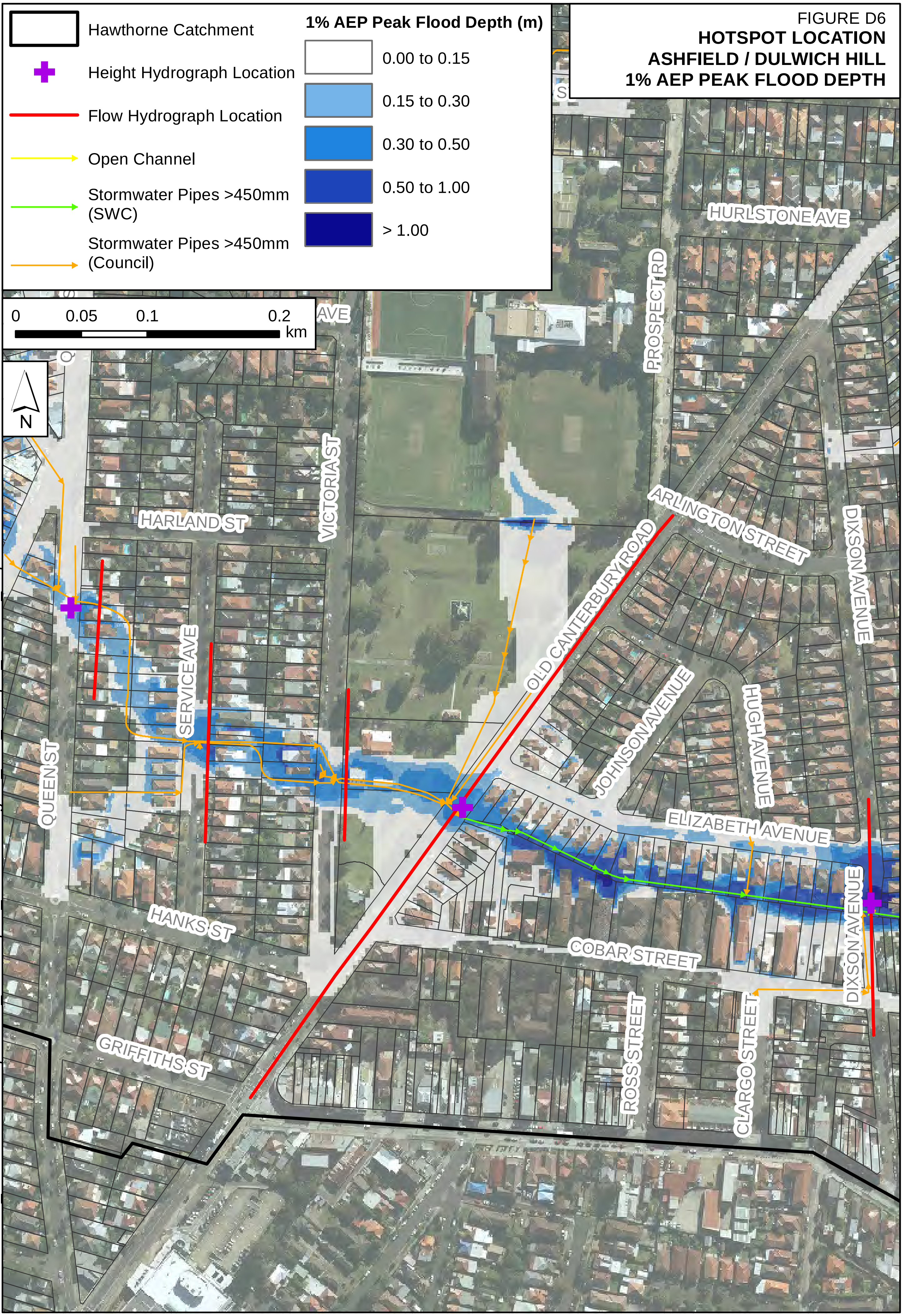

FIGURE D6
HOTSPOT LOCATION
ASHFIELD / DULWICH HILL
1% AEP PEAK FLOOD DEPTH

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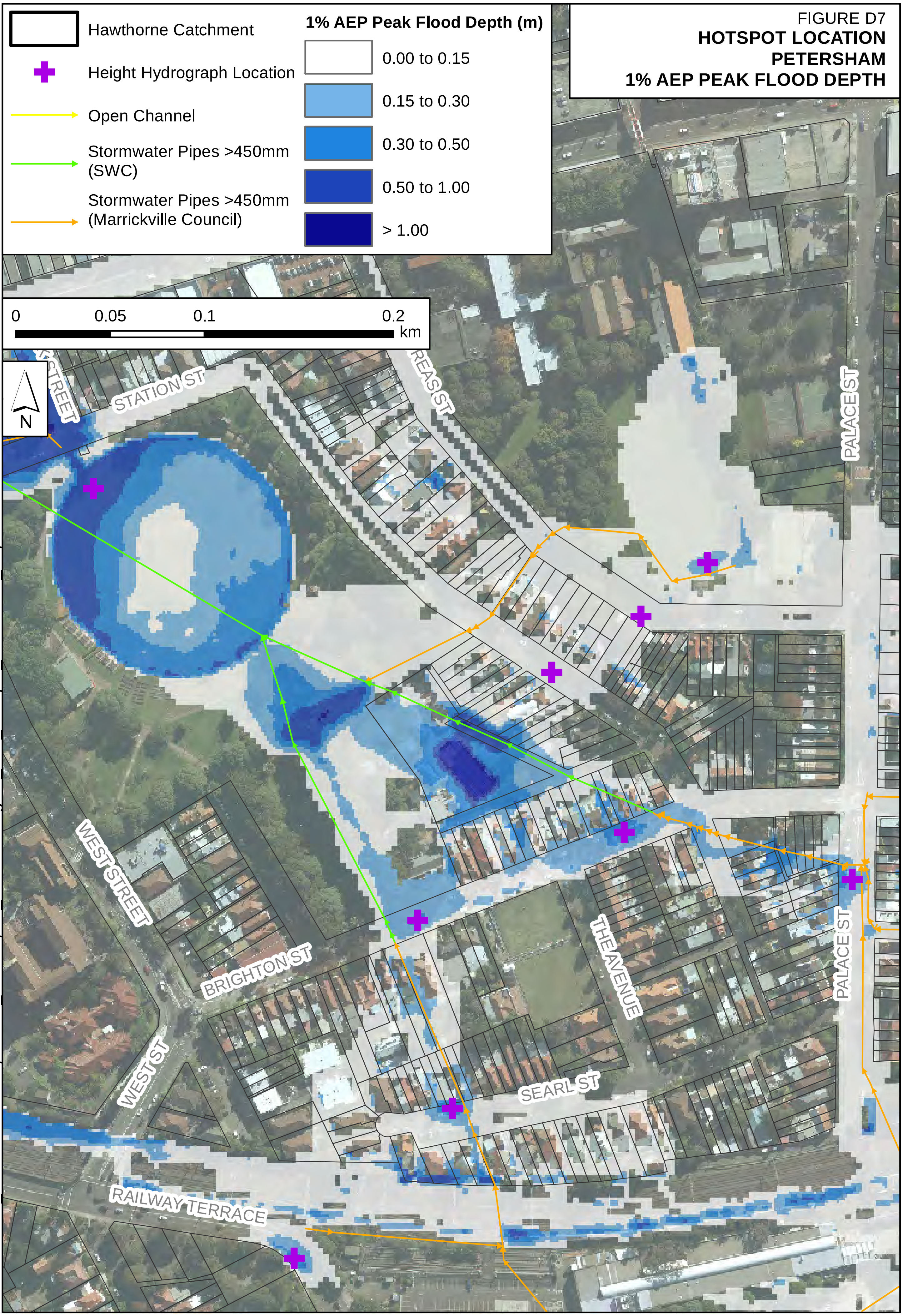

FIGURE D7
HOTSPOT LOCATION
PETERSHAM
1% AEP PEAK FLOOD DEPTH

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FIGURE D8
HOTSPOT LOCATION
SLOANE STREET
1% AEP PEAK FLOOD DEPTH

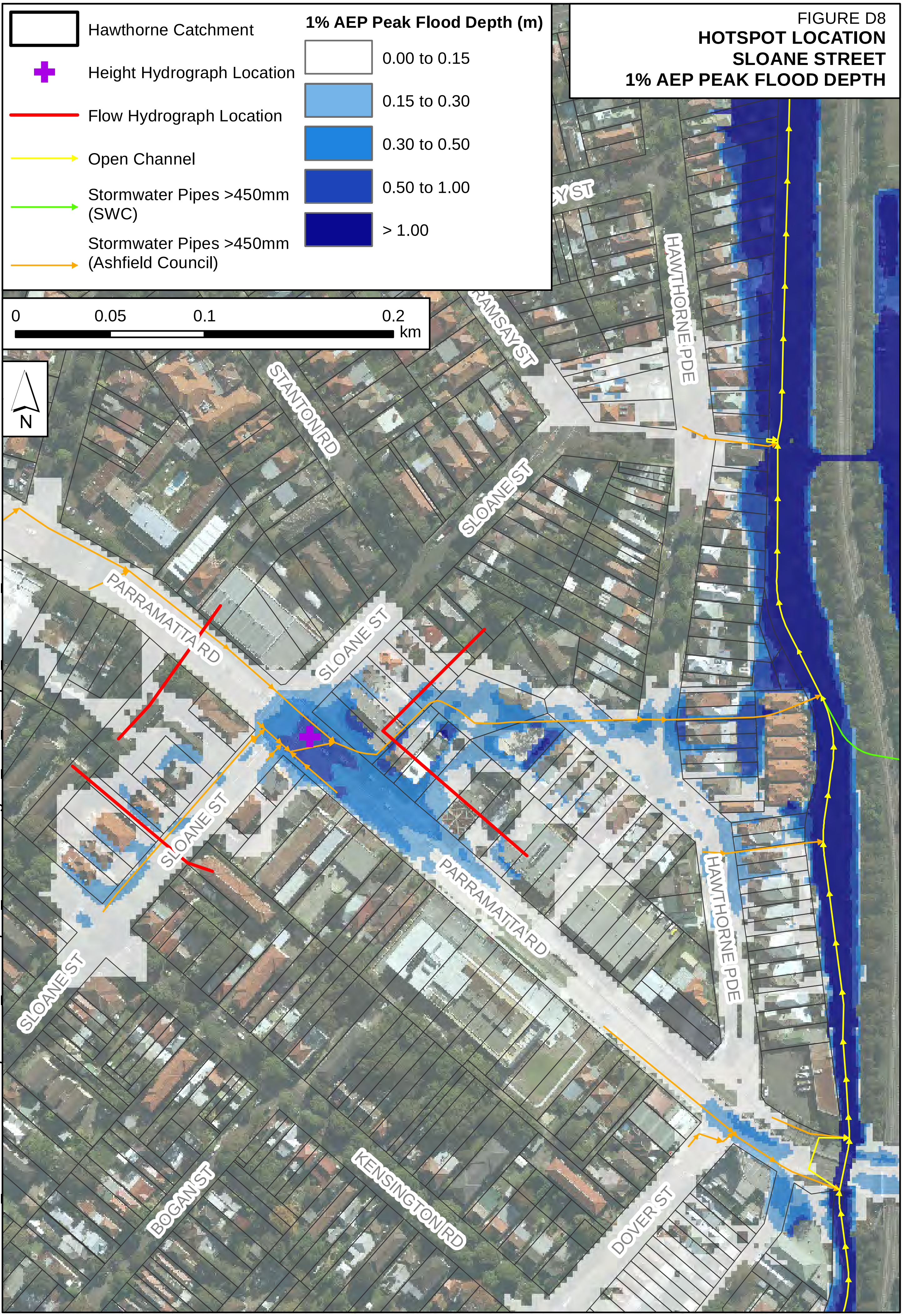

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FIGURE D9
HOTSPOT LOCATION
TRESSIDER AVENUE
1% AEP PEAK FLOOD DEPTH

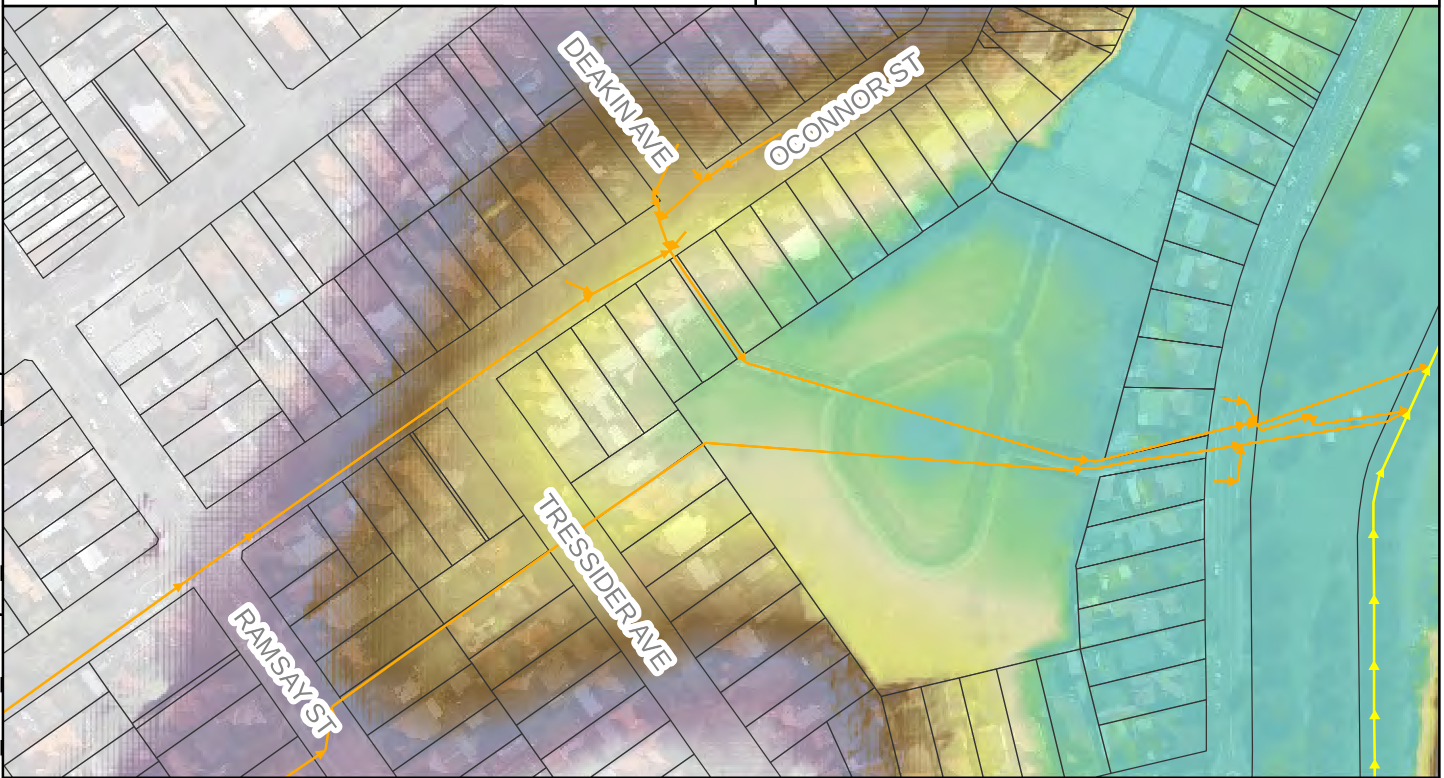
Hawthorne Catchment
Height Hydrograph Location
Flow Hydrograph Location
Open Channel
Stormwater Pipes >450mm (SWC)
Stormwater Pipes >450mm (Ashfield Council)

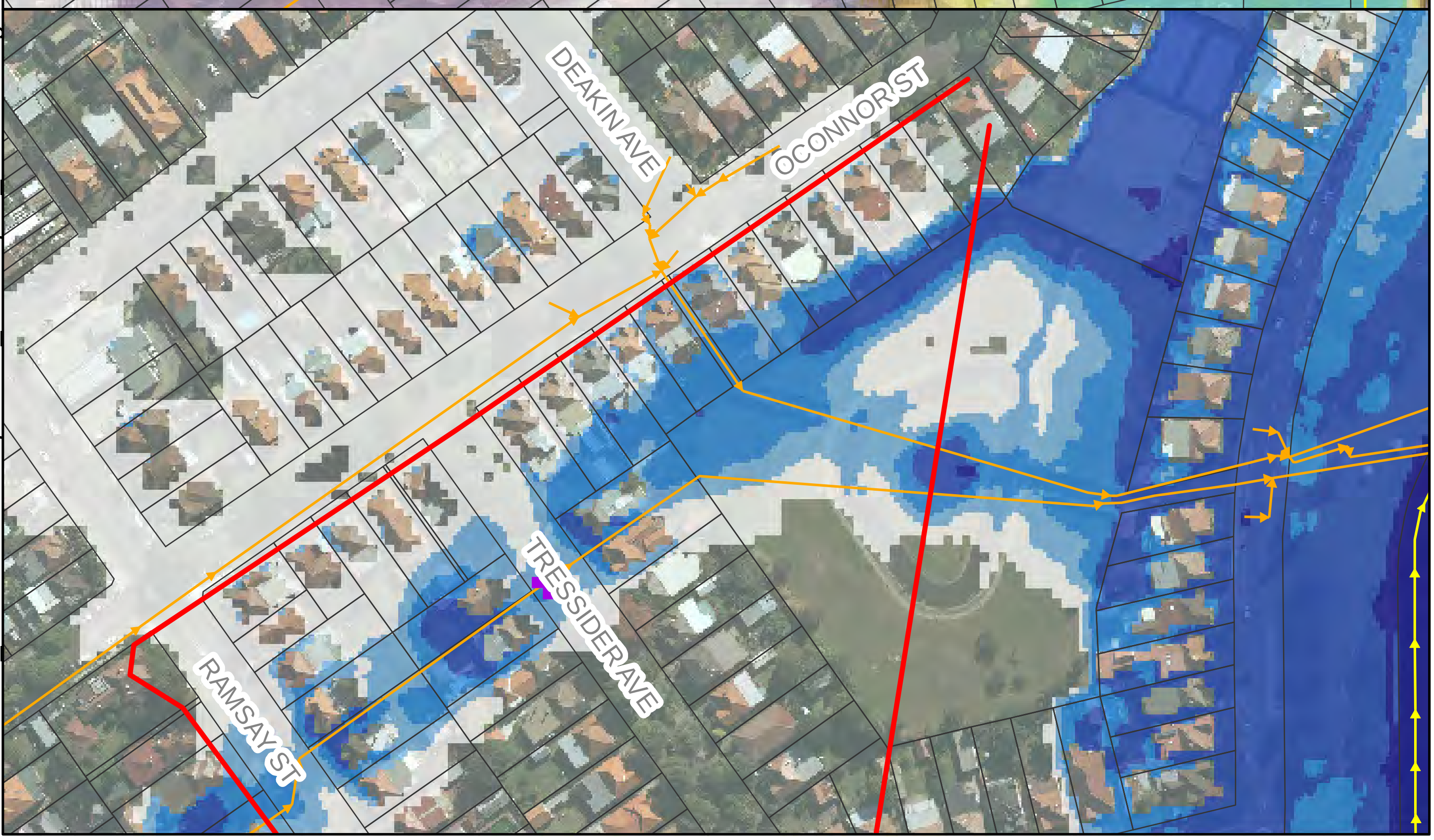
1% AEP Peak Flood Depth (m)
0.00 to 0.15
0.15 to 0.30
0.30 to 0.50
0.50 to 1.00
> 1.00

Ground Elevation (mAHD)
High : 12
Low : 2

0 0.05 0.1 0.2 km

N





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