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Ashfield Council

DOBROYD CANAL

FLOOD STUDY

FINAL DRAFT REPORT

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OCTOBER 2013

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Project Dobroyd Canal Flood Study	**Project Number** 111053
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DOBROYD CANAL FLOOD STUDY

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LIST OF ABBREVIATIONS

1D	One (1) Dimensional
2D	Two (2) Dimensional
ACC	Ashfield City Council
ALS	Airborne Laser Scanning
BCC	Burwood City Council
DEM	Digital Elevation Model
IFD	Intensity-Frequency-Duration
LiDAR	Airborne Light Detection and Ranging Survey
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 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 City Council (ACC) and Burwood City Council (BCC) 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 (SWC).

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 polices at various levels within the planning framework to ensure new development is constructed in a manner compatible with the flood hazard.

The Dobroyd Canal Flood Study constitutes the first stage of the management process for the Dobroyd Canal Catchment.

EXECUTIVE SUMMARY

BACKGROUND

The Dobroyd Canal catchment is located in Sydney's Inner West region, approximately 10 km from the CBD. The catchment includes the suburbs of Ashbury, Ashfield, Burwood, Burwood Heights, Croydon, Croydon Park, Haberfield and Summer Hill. Approximately 62% of the catchment is within Ashfield Council, 28% is within Burwood Council and the remaining 10% is within the City of Canterbury and Canada Bay Councils.

The Dobroyd Canal catchment drains to Iron Cove on the Parramatta River via an open channel and a series of inlet pits and pipes. Sydney Water Corporation (SWC) owns the larger "trunk" drainage assets including the open channel and the smaller pit and pipe networks are owned by the various councils. Open channel sections extend from Iron Cove up to the intersection of Carshalton and Norton Street.

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 Burwood Council when undertaking flood-related planning decisions for existing and future developments.

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.

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 November 1961, March 1975 and March 1983 having the greatest number of records.

HYDROLOGIC AND HYDRAULIC MODELLING PROCESS

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

These models were verified by comparison to specific yield rates for similar areas in the Sydney Metropolitan region, similarity to the adjacent Hawthorne Canal Flood Study and comparison to previous studies undertaken in the Dobroyd Canal catchment.

The design rainfall events that were modelled were the 50%, 20%, 10%, 5% and 1% AEP 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 including in the vicinity of Heighway Avenue (Ashfield), in the vicinity of Paisley Road (Burwood), on Queen Street (Burwood) and at the junction of Brown Street and Bland Street (Ashfield). A detailed examination of existing flood behaviour at these "hotspots" has been undertaken. The study shows that the railway line restricts flows and exacerbates the flooding problem. The former two "hotspots" are a result of this behaviour and extends floodwaters to surrounding streets. Major road routes such as the Dobroyd Parade (that leads onto the City West Link), the Hume Highway and Frederick Street (adjacent to the junction with Parramatta Road) are shown to experience significant flooding during many AEP design events. Inundation of these roads is likely to result in severe traffic disruption that would extend outside the Dobroyd Canal catchment.

A preliminary investigation into properties subject to flood related development controls shows that approximately 2,200 lots (of the approximately 9,900 lots within the catchment and accounting for around 22%) are liable to be tagged under the criteria adopted for the study.

1. INTRODUCTION

1.1. Background

The study was initially commissioned by Sydney Water Corporation (SWC) with the intent of modelling trunk drainage assets owned by SWC only. Subsequently, Ashfield City Council (ACC) and Burwood City Council (BCC) were invited to participate in the flood study. Both Councils accepted the opportunity and the scope of work was expanded to include modelling of Council's drainage infrastructure and local overland flow.

1.2. General

The Dobroyd Canal catchment drains to Iron Cove on the Parramatta River. Dobroyd Canal is also known as "Iron Cove Creek". The catchment includes the suburbs of Ashbury, Ashfield, Burwood, Burwood Heights, Croydon, Croydon Park, Haberfield and Summer Hill (shown in Figure 1). Approximately 62% of the catchment is within Ashfield Council, 28% is within Burwood Council and the remaining 10% is within the City of Canterbury and Canada Bay Councils.

Drainage elements in the catchment include kerbs and gutters, pits and pipes, and a network of trunk drainage elements including culverts and open 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 channel sections extend from Iron Cove up to the intersection of Carshalton and Norton Street.

1.3. Description of Study Area

The study area's catchment is fully urbanised, with approximately 79% of the catchment zoned for residential developments, 9% for special purpose, 6% for open space areas (parks and recreation areas), and the remaining 7% for business/commercial and industrial areas.

Elevations in the upper part of the catchment reach approximately 55 m AHD near Arthur Street and some reaches are relative steep with 2% to 4% grades. Overall catchment slope averages 0.8% along the main flow-path from headwaters to outlet. The main channel is tidal to upstream of Parramatta Road and channel width varies from ~ 2 m in upper areas to ~ 22 m at its confluence with Iron Cove.

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 50%, 20%, 10%, 5% and 1% AEP design storms and the Probable Maximum Flood (PMF) in the Dobroyd 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 City Council (ACC) and Burwood City Council (BCC). 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 Dobroyd Canal there are no stream gauges or official historical records available. A picture of flooding must therefore be obtained from an examination of Council records (if any), 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.

Table 1: Data Sources

Type of Data	Format Provided (Source)	Format Stored
Location, description and invert depths of pits, pipes and trunk drainage network	GIS (SWC)	DRAINS and TUFLOW models
Ground levels from ALS data	GIS (SWC)	GIS and TUFLOW model
Detailed survey data	GIS (SWC)	GIS and TUFLOW model
GIS information (cadastre, drainage pipe layout)	GIS (SWC)	GIS and TUFLOW model
Design rainfall	AR&R (1987)	DRAINS
Recorded flood data	Observation by Sydney Water	Report
Hydrology	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

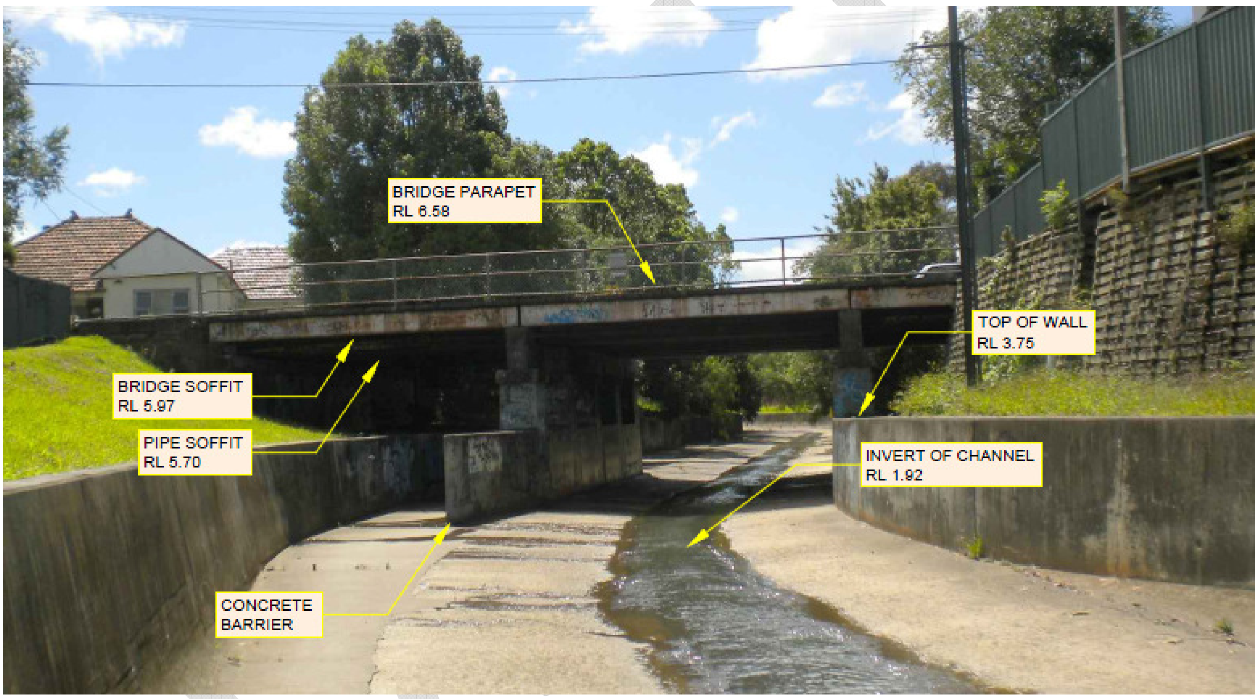
Figure 2).

2.4. Cross-section Data

Within the Dobroyd 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).

In locations where bridges traverse the open channel, additional survey was performed by Chase Burke & Harvey (CBH) Surveyors. From this, definition of the cross-sectional area was obtained, particularly where the bridge soffit was not the same height as the top of the concrete-lined channel, as shown in Photo 1.

Photo 1: Church 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 Dobroyd 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 1% AEP.

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 Database

An historic flood database, provided by SWC, provided information of flooding within the catchment from 1951 to 1988 (SWC, 2011). A summary of available historical flood levels is provided in Table 2 and Figure 5.

Table 2: Summary of Historical Flood Levels

Flood Events	Total Records	Number of Observed Flood Levels
September 1951	1	1
February 1959	3	3
November 1961	52	51
November 1969	2	1
October 1972	2	0
February 1973	5	1
April 1973	2	1
March 1975	14	10
March 1977	5	1
February 1980	1	0
March 1983	10	8
August 1986	5	4
November 1988	1	0

2.6.2. Community Consultation

A community consultation process was undertaken in collaboration with Ashfield City Council and Burwood City Council. This included distribution of an information sheet and a questionnaire to gather information pertaining to the community's experience of flooding within the catchment. BCC undertook this distribution to properties affected by preliminary 1% AEP extents. As ACC undertook the Dobroyd Canal Flood Study in conjunction with the Hawthorne Canal Flood Study, this information was distributed to the entire LGA.

The response rate was on average 6% across the two catchments. The responses received from the Ashfield Council area dominated the response rate with a ratio of 44:1. Given that the Ashfield LGA accounts for a larger portion of the overall catchment as well as the downstream and more flood affected regions, it is reasonable that the Ashfield residents would be more aware of flooding.

It was found that a quarter of the respondents had lived in the area for less than 5 years. This relatively high proportion can be accounted for by the proportion of rental dwellings within the respective LGA's (the Australian Bureau of Statistics recorded 40% of the Ashfield population and 37% of the Burwood population as residing in rental dwellings). As such, many would not have been present during less recent flood events and so were unable to provide information on these.

Table 3: Summary of Reported Incidences of flooding

Flood Event	Total Responses	House Flooded (above floor)	Other Buildings Flooded (above floor)	Other Descriptions of Flooding
1982	1	0	1	Depth of 0.3m reported
1984	1	0	1	Depth of 0.3m reported
1985	1	0	1	Depth of 0.3m reported
1980s	1	0	0	
1994-1995	1	0	0	
1998	1	0	1	Depth of 0.25m reported
2008	1	0	1	
2010	2	0	0	
2011	4	1	0	Depth of 0.5m reported
March 2012	6	0	1	
April 2012	2	0	1	
May 2012	1	2	1	
No Date Given	38	3	11	

2.7. Historical Rainfall Data

2.7.1. Overview

Rainfall data is recorded either daily (24hr rainfall totals to 9:00 am) or continuously (pluviometers measuring rainfall in small increments – less than 1 mm). Daily rainfall data have been recorded for over 100 years at many locations within the Sydney basin. In general, pluviometers have only been installed since the 1970's. Together these records provide a picture of when and how often large rainfall events have occurred in the past.

However, 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 9:00 am in the morning. Thus if a single storm is experienced both before and after 9:00 am, 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 Dobroyd Canal catchment are usually localised and as such are only accurately represented 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 Dobroyd Canal catchment.

Station Number	Station Name	Operating Authority	Distance from centre of the catchment (km)	Elevation (m AHD)	Date Opened	Date Closed	Type
66000	Ashfield Bowling Club	BOM	1.16	25	30/03/1896		Daily
566112	Ashfield (Ashfield Park Bowling Club)	SWC	1.20	20	2/12/1993	1/02/2001	Continuous
66017	Barnwell Park Golf Course	BOM	1.52	4	29/11/1929	28/11/2003	Daily
66150	Canterbury Heights	BOM	1.83	61	30/08/1906	29/12/1916	Daily
66165	Ashfield Prospect Rd	BOM	2.00	43	01/01/1894	1/01/1904	Daily
66194	Canterbury Racecourse AWS	BOM	2.48	3	2/10/1995		Synop
66091	Burwood 2 Public School	BOM	2.81		29/09/1911	29/12/1923	Daily
66113	Burwood 1	BOM	2.87		01/01/1884	1/01/1922	Daily
66026	Homebush	BOM	2.87		30/10/1924	29/12/1952	Daily
66034	Abbotsford (Blackwall Point Rd)	BOM	3.28	15	1/01/2004		Daily
66111	Croydon	BOM	3.34		30/01/1879	29/12/1921	Daily
66013	Concord Golf Club	BOM	3.91	15	1/01/1930		Daily
566020	Enfield (Composite Site)	SWC	3.93	10	14/04/1959		Continuous
566020	Enfield (Composite Site)	SWC	3.93	10	14/04/1959		Daily
566065	Lilyfield Bowling Club	SWC	3.94	20	21/12/1988		Continuous
66036	Marrickville Golf Club	BOM	4.24	6	29/04/1904	29/12/1970	Daily
66036	Marrickville Golf Club	BOM	4.24	6	6/04/2001		Operational
66071	Gladesville Champion Rd	BOM	4.52	10	27/02/1997	29/09/2000	Daily
566026	Marrickville Sps	SWC	4.92	5	1/05/1904		Continuous
566026	Marrickville Sps	SWC	4.92	5	1/05/1904		Daily
66108	Hunters Hill St Josephs College	BOM	5.06		1/01/1916	1/01/1923	Daily
66018	Earlwood Bowling Club	BOM	5.09	31.1	30/07/1914	29/12/1975	Daily
66064	Concord Walker Hospital	BOM	5.46	7.6	30/10/1894	29/12/1972	Daily
66175	Schnapper Island	BOM	5.46	5	28/02/1932	29/12/1939	Daily
66101	Fernbank	BOM	5.53		01/01/1889	1/01/1913	Daily
566078	South Cronulla	SWC	5.64	20	9/02/1990		Continuous
66070	Strathfield Golf Club	BOM	5.99	21	11/06/1997		Operational
66070	Strathfield Golf Club	BOM	5.99	21	1/01/1952		Daily

2.7.3. Analysis of Daily Read Data

An analysis of the records for the nearest daily rainfall stations, namely Ashfield Bowling Club (66000) and Barnwell Park Golf Course (66017), was undertaken. The Ashfield gauge is located within the Dobroyd Canal Catchment (adjacent to the eastern catchment border) and the Barnwell Park gauge is located to the north of the catchment, both of which are shown on Figure 6. Additional daily rainfall stations surrounding the catchment are shown within Figure 9 however these were of insufficient record length and had been decommissioned prior to 1952. The Ashfield Bowling Club station was established in March 1896 and is still active. The Barnwell Park Golf Course station was established in November 1929 and decommissioned in November 2003.

The results indicate that the 1986 and 1990 events were the largest daily rainfall events in recent times. The 1986 event is known to have caused flooding in the Dobroyd Canal Catchment based upon SWC records (see Section 2.6). Although there is no evidence to suggest that the 1990 storm event 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.

Table 5: Daily rainfalls greater than 150mm at Ashfield Bowling Club and Barnwell Park Golf Course

Ashfield Bowling Club (66000)		
Mar 1896 – to date		
Rank	**Date**	**Rainfall (mm)**
1	6/08/1986	245
2	9/03/1913	210
3	28/03/1942	206
4	3/02/1990	206
5	10/02/1956	194
6	17/06/1950	182
7	13/02/1911	175
8	27/11/1955	167
9	22/02/1954	160
10	26/03/1984	158
11	24/01/1955	157
12	11/03/1958	154
13	19/02/1959	152
14	10/01/1949	151

Barnwell Park Golf Course (66017)		
Nov 1929 – Nov 2003		
Rank	**Date**	**Rainfall (mm)**
1	30/03/1942	315
2	11/06/1991	253
3	6/08/1986	250
4	5/02/1990	245
5	11/02/1992	238
6	30/04/1988	228
7	10/02/1956	201
8	9/04/1973	197
9	16/02/1988	164
10	19/11/1961	163
11	10/01/1949	156
12	1/05/1955	156
13	27/11/1955	155
14	8/08/1998	152
15	15/06/1952	151

2.7.4. Analysis of Pluviometer Data

Continuous pluviometer records provide a more detailed description of temporal variations in rainfall. As such, the Ashfield Park Bowling Club, Enfield, Lilyfield Bowling Club and Marrickville Bowling Club pluviometer stations were analysed.

These pluviometer stations are all operated by SWC, with Marrickville and Enfield having the longest records. The Marrickville gauge was established in 1904 with sub-daily records available from December 1979. The Enfield gauge was established in 1959 with sub-daily records beginning in June 1983. The Ashfield gauge was established in December 1993 and the Lilyfield gauge was established in December 1988. However, the Ashfield gauge has since been decommissioned, as of February 2001.

Rainfall intensities at the gauges were assessed for the 1 hour and 2 hour storm burst durations and compared to frequencies derived from AR&R 1987 in Table 6 These durations were selected for analysis based upon the critical duration analysis (discussed in Section 7.2.), which found these storm durations to produce the highest flood levels within the Dobroyd Canal Catchment. From Table 6 it can be seen that a large magnitude rainfall event has not occurred within the operational period of any of these gauges.

Table 6: Approximate ARI Recorded at Pluviometer Stations

Station Name	Years of Record	Highest Approximate ARI (AR&R 1987)	
		1 hour storm burst	2 hour storm burst
Ashfield Park Bowling Club (566112)	7	1 – 2 year ARI	2 – 5 year ARI
Enfield (566020)	30	10 – 20 year ARI	2 – 5 year ARI
Lilyfield Bowling Club (566065)	24	10 – 20 year ARI	10 – 20 year ARI
Marrickville Bowling Club (566026)	34	10 – 20 year ARI	10 – 20 year ARI

The 10th April 1998 event produced the highest intensity 2 hour storm burst at the pluviometer stations analysed. A comparison of significant rainfall events and their respective ranking is shown in Table 7 (1 being the highest ranked storm burst at the pluviometer gauge).

The Ashfield pluviometer is the only gauge located within the catchment however it also has the shortest operational period. As a result, the 1998 storm event was the only significant event recorded at the gauge with corresponding reports of flooding. Despite the 1998 event recording the highest intensity 2 hour storm burst, there were insufficient records of resulting flooding to calibrate to this event with only a single indicative depth reported.

Table 7: Rainfall Intensities for the 10th April 1998

	Duration (minutes)		
	30	**60**	**120**
Ashfield Park Bowling Club (566112)			
Max Rainfall (mm)	26	33	57
Intensity (mm/hr)	52	33	28
Approximate ARI	1 – 2 year ARI	1 – 2 year ARI	2 – 5 year ARI
Rank comparative to gauge records for relevant duration	3	1	1
Enfield (566020)			
Max Rainfall (mm)	24	42	64
Intensity (mm/hr)	48	42	32
Approximate ARI	1 – 2 year ARI	2 – 5 year ARI	2 – 5 year ARI
Rank comparative to gauge records for relevant duration	20	4	1 (equal rank as 5/8/1986)
Lilyfield Bowling Club (566065)			
Max Rainfall (mm)	41	47	59
Intensity (mm/hr)	82	47	30
Approximate ARI	5 – 10 year ARI	2 – 5 year ARI	2 – 5 year ARI
Rank comparative to gauge records for relevant duration	3	2	2
Marrickville Bowling Club (566026)			
Max Rainfall (mm)	39	51	76
Intensity (mm/hr)	78	51	38
Approximate ARI	5 – 10 year ARI	5 – 10 year ARI	10 – 20 year ARI
Rank comparative to gauge records for relevant duration	3	4	2

2.8. Design Rainfall Data

The design rainfall intensity-frequency-duration (IFD) data was obtained from the Bureau of Meteorology's online design rainfall tool. The input parameters for these calculations are sourced from AR&R (1987).

Table 8: Rainfall IFD data at the centre of the Dobroyd 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	94.5	121	154	173	198	230	255
6 minutes	88.4	113	144	162	186	216	239
10 minutes	72.4	93	119	134	154	180	199
20 minutes	53	68.3	88.4	100	115	135	151
30 minutes	43.1	55.7	72.5	82.3	95.2	112	125
1 hour	29.2	37.9	49.6	56.5	65.6	77.5	86.6
2 hours	19.1	24.8	32.5	37.1	43.1	51	57.1
3 hours	14.7	19.1	25.1	28.7	33.3	39.4	44.1
6 hours	9.44	12.2	16.1	18.3	21.2	25.1	28.1
12 hours	6.09	7.89	10.3	11.8	13.7	16.2	18.1
24 hours	3.97	5.15	6.74	7.69	8.92	10.5	11.8
48 hours	2.55	3.31	4.33	4.94	5.74	6.79	7.58
72 hours	1.91	2.47	3.24	3.69	4.28	5.06	5.65

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 9.

Table 9: 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	102.6

2.9. Previous Studies

2.9.1. Dobroyd SWC 53 Capacity Assessment (SWC, 1998)

This report was prepared by Sydney Water and investigated the current performance of Sydney Water Corporation's Dobroyd SWC 53 and gives an estimate of the impact of simulated urban consolidation on that performance.

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

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

Local catchment pit and pipe details were unavailable and therefore not modelled. The report notes that this results in an overestimation of flows and ponding depths in the smaller design events modelled.

The hydraulic capacity in the main stormwater channel discharging into Iron Cove was found to be 183 m^3/s with a 5 year ARI peak flow of 105 m^3/s. The capacity of the main channel was found to be in the range of 25 – 50 year ARI with 51% of the current trunk drainage system able to contain flows from a 5 year ARI storm event. Note that given suitably conservative tail water levels it is likely that all of these estimates would be revised downwards.

2.9.2. Hydraulic Study and On-Site Detention Modelling for Burwood Council Catchments (Robinson GRC Consulting, 2002)

Robinson GRC Consulting prepared this report on behalf of Burwood City Council from 2000 to 2002. The catchments within the bounds of Burwood City Council's jurisdiction, and hence included in the study, included the Dobroyd Canal catchment, Cooks River catchment, Powells Creek catchment, Exile Bay catchment, St Lukes catchment and William Street catchment. The primary objective of this study was to develop a computer model to assess the 1% AEP event and from this determine insufficiencies in the drainage system, as well as identify overland flow paths that occurred to an unfavourable frequency. Once these "hotspots" were identified, possible mitigation measures were proposed with further modelling undertaken to assess these. Additional to this, the report modelled the 50%, 5% and 1% AEP event with the purpose to propose Permissible Site Discharge (PSD) and storage volumes for potential On-Site Detention (OSD) systems.

The data collected for the purpose of this study included:

- survey of pit levels;
- survey of levels of the kerb, gutter, road centrelines and driveways in locations that were deemed important;
- survey of property levels that may be subject to flooding;
- three laser-doppler flow gauges recorded over the period of the 8th May 2000 to the 31st August 2000. One was located in the Cooks River catchment and two were located in the Dobroyd Canal catchment; and
- two tipping-bucket rain gauges recorded over the period of the 3rd May 2000 to the 15th September 2000. These were located at the Woodstock Park Community Centre (on Church Street, Burwood) and in Council's Depot (near Tangarra Road, Croydon Park).

However, during the period in which the flow gauges and rain gauges were in operation, the rainfall experienced was not of a significant magnitude. The largest rainfall recorded over the period of record was 13 mm over a 24 hour period.

The hydraulic model established for this report was DRAINS. This model was calibrated to the flow gauge and rain gauge records that were collected for the purpose of this study. However, as these events were not of a significant magnitude, the calibration was determined to be inconclusive.

The hotspots identified in this report were:

(Croydon Branch)

- Appian Way;
- Wyatt Avenue and Weldon Street;
- Tahlee Street;
- Devonshire Street;
- Murray Street;
- Brady Street;
- Fitzroy Street;
- Rosa Street;
- Paisley Road;
- Church Street;
- Elizabeth Street;
- Shaftesbury Road and Paisley Road
- Albert Crescent (West);
- Lucas Road;
- Albert Crescent (East);
- Webb Street;
- Irrara Street;
- Young Street (South);
- Young Street (North);
- Wright Street;
- Robinson Street;
- Queen Street;

(Main Dobroyd Branch (South))

- Culdees Road;
- Ardgryffe Street;
- Waratah Street;
- Boyle Street;
- Beaufort Street;
- Seymour Street;
- Beresford Avenue;
- Brighton Street (South);
- Croydon Avenue South;
- Greenhills Street;

(Badminton Street Branch)

- Claremont Road;
- Badminton Road (North);
- Badminton Road (South);
- Austin Avenue;
- Gala Avenue;
- Brighton Street (North);
- Croydon Avenue (North);
- Greenhills Street (North).

The general assessment concerning hotspots in the Dobroyd Canal catchment was that the drainage network followed previously existing creek lines that have since been built over. With the urbanisation of the catchment a road network was established that appears to disregard the topography such as creeks.

The report found that the potential for remedial work was limited and “the provision of overland flow paths through properties … appears to be the most effective type of remedy” (Robinson GRC Consulting, 2002).

2.9.3. Stormwater Drainage Infrastructure Review for Burwood Council (Brown Consulting (NSW), 2004)

Brown Consulting carried out this study on behalf of Burwood Council in 2004. The study investigated overland flow that resulted from the drainage system’s inability to convey runoff under current conditions, the impact of increased development, the effectiveness of OSD, and the re-assessment of the proposed remedial works identified by Robinson GRC Consulting. From this, recommendations were made as to what provisions Burwood Council may have to establish developer contributions under Section 94 due to increased development within the Town Centre area. The Town Centre area was identified as being the area surrounding Burwood Train Station, which includes the Dobroyd Canal, St. Lukes and Powells Creek catchments.

This study used the DRAINS model that had been established for the catchments by Robinson GRC Consulting (although it was noted that in 2003, Robinson GRC Consulting merged with WP Brown and Partners, now Brown Consulting (NSW)). However the version of DRAINS utilised was updated to the latest version available at the time the study was being undertaken.

Increased development was assessed as an increase in modelled impervious percentage. In the Town Centre area the impervious percentage was increased from 70% impervious in the current conditions to 90% impervious. Elsewhere in the catchments the impervious area was increased by 6%. This scenario was modelled for all the catchments identified in the Robinson GRC Consulting report.

OSD was modelled for three different scenarios. The first scenario applied OSD to 30% of the Town Centre area without applying OSD outside this area. The second scenario applied OSD to 30% of the Town Centre area and 10% of the area outside this area. The third scenario applied OSD to 50% of the Town Centre area and 10% of the area outside this area.

2.9.4. Flood Study for Proposed New Residence: No. 7 Alexandra Street, Ashfield NSW (ACOR Consultants, 2007)

This report was undertaken by ACOR Consultants on behalf of the property owner. The flood study was prompted by a request from Ashfield Council upon receipt of a Development Application (DA) proposal for the site.

The hydrologic model used for the study was DRAINS. The flow rates produced by DRAINS were applied to the HEC-RAS hydraulic model for the 100 year ARI and the 20 year ARI. The hydraulic model extended from the Ramsey Street Bridge up to the John Street Bridge and Croydon Road.

The peak flood rates produced by DRAINS are summarised in Table 10 and compared to the current study in Section 6.4.4.

Table 10: ACOR Consultants – DRAINS peak flow rates

Sub Catchment	20 year ARI		100 year ARI	
	Q_{20} (m^3/s) from DRAINS for catchment	Q_{20} (m^3/s) from Cumulative in Channel	Q_{100} (m^3/s) from DRAINS for catchment	Q_{100} (m^3/s) from Cumulative in Channel
Arthur St – 1	33.7	33.7	55.4	55.4
Thomas St – 1	27.9	61.5	45.9	101
Elizabeth St – 3	12.3	73.7	20.2	121
John St – 4	9.5	82.9	15.6	136
Burwood – 5	22.2	104	36.6	172
Alexandra St – 6	4.63	107	7.09	176
Parramatta Rd – 7	27.5	133	45.1	220
Henley Marine Dr – 8	15.5	142	23.6	235
Iron Cove – 9	28.7	170	47.4	279

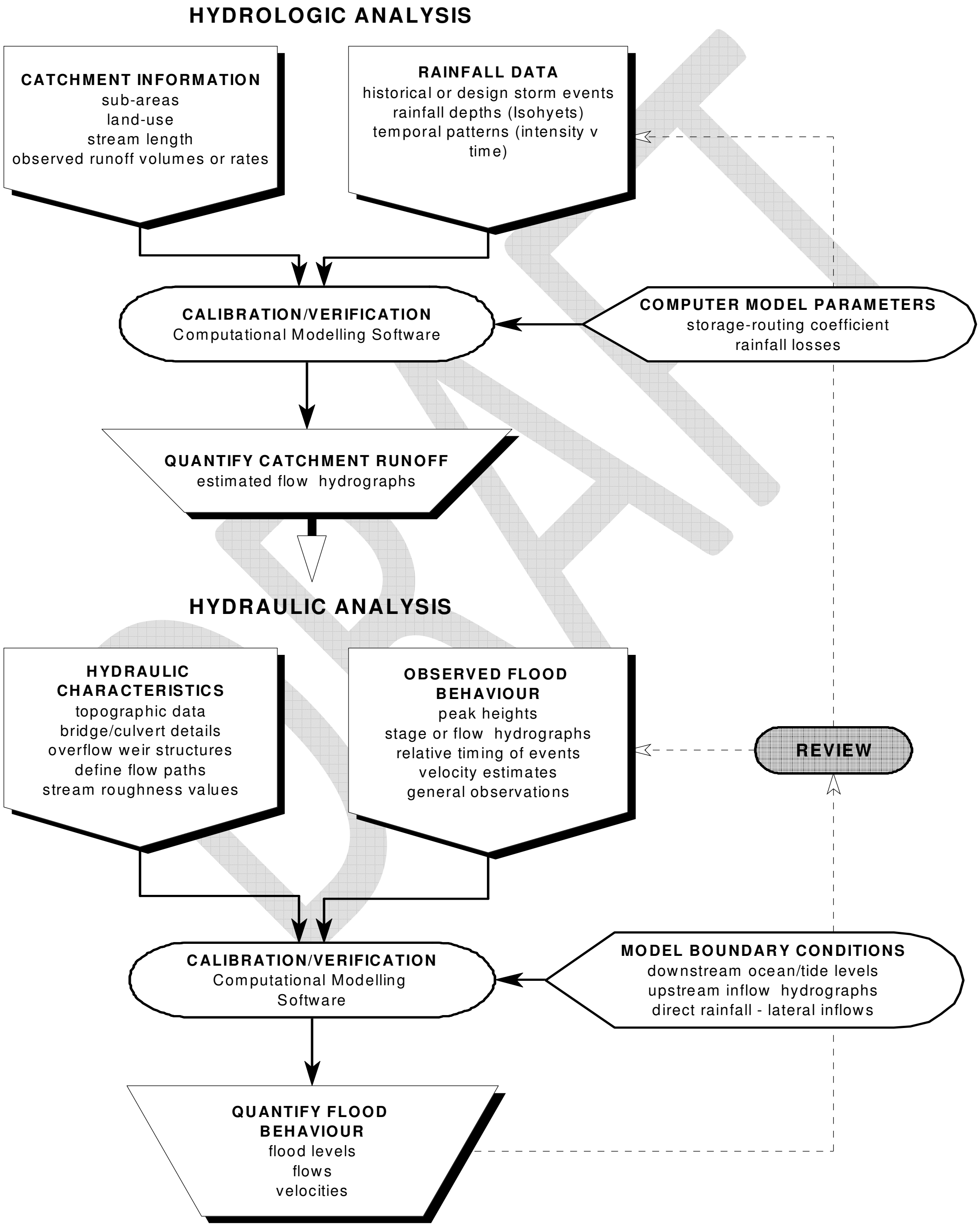
The hydraulic model determined the peak flood level in the open channel adjacent to No. 7 Alexandra Street to be 6.36 m AHD in the 100 year ARI event.

The report concluded that the minimum floor level at No. 7 Alexandra Street be 6.87 m AHD (or above), thereby complying with the council's specification that new floor levels be 0.5m above the 100 year ARI peak flood level.

3. STUDY METHODOLOGY

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

Diagram 1: Flood Study Process



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.

As such, the hydrologic model, DRAINS, was built and used to create flow boundary conditions for input into a two-dimensional unsteady flow hydraulic model, i.e. TUFLOW.

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

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

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

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

The sub-catchments in the hydrologic model were kept small (on average approximately 1.5 ha) such that the overland flow behaviour for the study was generally defined by the hydraulic model. This joint modelling approach was verified against previous studies and alternative methods.

3.1. Hydrologic Model

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

The DRAINS model is broadly characterised by the following features:

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

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

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

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

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

3.2. Hydraulic Model

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

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

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

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

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

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

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

3.3. 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 after the community consultation;
- design outflows for localised sub-catchments were obtained from the DRAINS hydrologic model and applied as inflows to the TUFLOW model;
- sensitivity analysis was undertaken to assess the relative effect of changing various TUFLOW modelling parameters.

4. HYDROLOGIC MODEL

4.1. Sub-catchment Definition

The total catchment represented by the current DRAINS model is 8.3 km^2. This area has been represented by a total of 551 sub-catchments giving an average sub-catchment size of approximately 0.015 km^2. The sub-catchment delineation ensures that where hydraulic controls exist that these are accounted for and able to be appropriately incorporated into hydraulic routing. The sub-catchment layout is shown in Figure 7.

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 Dobroyd 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.

Table 11: Impervious Percentage per Land-use

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

The proportion of each land-use category within a sub-catchment was determined based upon the hydraulic model roughness schematisation, shown in Figure 9. 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 2 and Photo 3.

Photo 2: Impervious area (shaded in red) within a representative residential area (outlined in blue)

Photo 3: Impervious area (shaded in red) within a representative commercial area (outlined in blue)

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 previous studies within the Dobroyd Canal catchment undertaken by Robinson GRC Consulting (2002) and ACOR Consultants (2007) and the adjacent catchment of Hawthorne 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 2.

The TUFLOW hydraulic model includes the Dobroyd Canal catchment drainage down to Iron Cove. The 2D model extends from WH Wagener Oval in Ashbury to the south, down to Iron Cove. The total area included in the 2D model is 8.3 km^2. The extents of the TUFLOW model are shown in Figure 1.

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 8. At this location, the 1D and the 2D domain are operating and the boundary was applied to both domains within the hydraulic model.

5.2.3. Outflows into Adjacent Catchments

In events of a relatively small magnitude, runoff produced within the Dobroyd Canal Catchment discharge into Iron Cove. However, in larger events some flood waters are restricted in their capacity to flow downstream and instead drain out of the catchment they originated in.

The hydraulic model was schematised so as not to restrict flow from crossing the watershed boundary. As such, the hydraulic model extent was expanded to include small portions of the adjoining catchments. Where the watershed boundary was crossed, the flow was removed from the hydraulic model with localised hydraulic boundaries.

The two locations where the watershed boundary was crossed were:

- within the Burwood Town Centre; and
- within the vicinity of Beaufort Street, Burwood.

Flow from the Burwood Town Centre that was impeded from crossing Shaftsbury Road and the railway embankment accumulated in these areas. When the height of this accumulated flood water exceeded the watershed boundary height, flow crossed into the St. Lukes Catchment. Within this adjacent catchment, the topography conveyed flow west along Railway Parade and then along Burwood Road underneath the railway embankment, where a localised hydraulic boundary was schematised.

Flow through properties on Boyle Street, Beaufort Street and Seymour Street occurred parallel to the watershed boundary. The height of the boundary above the ground level being traversed by the flow was not significant. As such, a portion of the flow has the potential to cross the watershed boundary into the Cooks River Catchment, located south of the Dobroyd Canal Catchment. Within this adjoining catchment, the flow is conveyed perpendicular to the flow within the study area catchment from which it originated. This flow travelled along the aforementioned roadways (and the properties adjacent to) before crossing Georges River Road, where a localised hydraulic boundary was schematised.

The discharge into adjoining catchments was quantified, the summary of which is provided in Table 13. Comparative to the flow discharged into Iron Cove, the amount crossing the watershed boundary into adjacent catchments was relatively insignificant.

Table 13: Discharge into adjacent catchments

Location	5% AEP	2% AEP	1% AEP	PMF
Burwood Town Centre				
Volume (m^3)	0	3.8	259.1	5,581
Peak Flow (m^3/s)	0	0.01	0.20	2.49
Beaufort Street, Burwood				
Volume (m^3)	11.0	83.6	290.0	17,974
Peak Flow (m^3/s)	0.06	0.15	0.48	9.56

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 9. The Manning's "n" values adopted for these areas, including flowpaths (overland, pipe and in-channel), are shown in Table 14. 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 14: Manning's "n" values adopted in TUFLOW

Surface	Manning's "n" Adopted
Pipes	0.015
Roads and Footpaths	0.02
Light Vegetation	0.03
General Overland Areas	0.04
Properties	0.05

5.4. Hydraulic Structures

5.4.1. Buildings

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

5.4.2. Fencing and Obstructions

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

5.4.3. Bridges

Key hydraulic structures were included in the hydraulic model, as shown in Figure 3. 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.

5.4.4. Sub-surface Drainage Network

Figure 8 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:

- 1043 pipes;
- 214 open channel segments; and

- 1243 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);
- AR&R Revision Project 11 Blockage of Hydraulic Structures (Engineers Australia, 2013); and
- the policies of various local authorities and infrastructure agencies.

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

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

Type of structure		Blockage conditions	
		Design blockage	**Severe blockage**
Sag Kerb Inlet	Kerb slot inlet only Grated inlet only Combined inlets	0/20% 0/50% [1]	100% (all cases)
On-grade kerb inlets	Kerb slot inlet only Grated inlet only (longitudinal bars) Grated inlet only (transverse bars) Combined inlets	0/20% 0/40% 0/50% [2]	100% (all cases)
Field (drop) inlets	Flush mounted Elevated (pill box) horizontal grate Dome screen	0/80% 0/50% 0/50%	100% (all cases)
Pipe inlets and waterway culverts	*Inlet height < 3m and width < 5m* Inlet Chamber	0/20% [3]	100% [4]
Pipe inlets and waterway culverts	*Inlet height > 3m and width > 5m* Inlet Chamber	0/10% [3]	25% [3]
Pipe inlets and waterway culverts	Culverts and pipe inlets with effective debris control features	As above	As above
Pipe inlets and waterway culverts	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 8.3.2. These scenarios included blockage of all pipes, blockage of all bridges and culverts over the open channel, and blockage of the drainage infrastructure (such as pipes and culverts, but excluding roadways that convey flow) underneath the railway embankment. 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. This is perhaps more relevant to vegetated open channels that are subject to sedimentation rather than the concrete lined open channels present in the Dobroyd Canal Catchment.

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.

Typically in urban areas such information is lacking. Issues which may prevent a thorough calibration of hydrologic and hydraulic models are:

- 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; and
- rainfall records for past floods are limited and there is a lack of temporal information describing historical rainfall patterns within the catchment.

In the event that a calibration and validation of the models is not possible or limited in scope, it is best practice to undertake a verification of the models and a detailed sensitivity analysis.

6.2. Correlating Data

The correlation between the historic flood level data (discussed in Section 2.6) and available pluviometer data (discussed in Section 2.7.4) is summarised in Table 16.

The approximate ARI for these storm events have been estimated based on the daily read rainfall station located at Ashfield Bowling Club (discussed in Section 2.7.4) and the IFD data for the centre of the Dobroyd Canal catchment (discussed in Section 2.8). However, this estimation considers the daily rainfall to have occurred at a constant intensity over the 24 hour period of record. As such it is possible that the rainfall intensity was greater over a shorter duration, and hence the approximate ARI's are likely to be an under estimation. Sufficiently located pluviometer stations provide a closer approximation of the storm intensity and ARI event. However, as can be seen in Table 16, many of the storm events occurred prior to the establishment of pluviometer stations.

For the storm events in which a pluviometer station was present, the number of corresponding recorded flood levels were found to be of an insufficient quantity or spatial distribution. The pluviometer stations were located outside the catchment and the ARI estimated for the rainfall recorded was typical of a small magnitude (shown in Table 19). Engineers Australia (2012) advises that calibration events "span the magnitude range of the intended design events with a preference for the more important design floods (eg. 1% AEP event)"

For this reason, a verification of the models was undertaken instead of calibrating or validating the models.

Table 16: Data available for various storm events

Storm Events	Total Records	Indicative Depths Available	Approximate ARI	Pluviometer Stations in Operation
September 1951	1	1	< 1 year ARI	N/A
February 1959	3	1	2 – 5 year ARI	N/A
November 1961	52	44	2 – 5 year ARI	N/A
November 1969	2	1	1 – 2 year ARI	N/A
October 1972	2	0	< 1 year ARI	N/A
February 1973	5	1	< 1 year ARI	N/A
April 1973	2	1	< 1 year ARI	N/A
March 1975	14	12	1 – 2 year ARI	N/A
March 1977	5	0	1 – 2 year ARI	N/A
February 1980	1	0	< 1 year ARI	566026 – Marrickville Bowling Club
March 1983	10	7	1 – 2 year ARI	566026 – Marrickville Bowling Club
August 1986	5	3	20 – 50 year ARI	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club
November 1988	1	0	< 1 year ARI	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club
1998	1	1	N/A *	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club 566065 - Lilyfield Bowling Club 566112 – Ashfield Bowling Club
2008	1	0	N/A *	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club 566065 - Lilyfield Bowling Club
2010	2	0	N/A *	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club 566065 - Lilyfield Bowling Club
2011	4	1	N/A *	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club 566065 - Lilyfield Bowling Club
8 March 2012	6	3	1 – 2 year ARI	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club 566065 - Lilyfield Bowling Club
April 2012	2	0	N/A *	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club 566065 - Lilyfield Bowling Club
May 2012	1	0	N/A *	566020 - Enfield (Composite Site) 566026 – Marrickville Bowling Club 566065 - Lilyfield Bowling Club

* Incomplete daily rainfall records during these periods

6.3. 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 Dobroyd Canal catchment. DRAINS was the hydrologic model used in Rose Bay and the catchment is located approximately 12 km from the Dobroyd Canal Catchment.

Comparison of specific yield was used for the model verification and is calculated by dividing the peak discharge by the area of the upstream catchment. This calculation removes the effects that variations in sub-catchment size have on peak discharge. Also, to remove the effects that differences in catchment delineation can have on peak discharge, the specific yield was calculated for multiple, randomly-selected, sub-catchments. The results are shown in Table 17.

Table 17: Comparable Sub-catchment Hydrologic Model Check

Sub-catchment	Dobroyd 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	1.7	1.0	0.6	1	0.6	0.7
2	10.1	5.3	0.5	0.4	0.2	0.6
3	20.7	10.3	0.5	0.6	0.4	0.6

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

6.4. Hydrologic and Hydraulic Model Verification

Verification of the hydraulic model was undertaken by:

- comparing the flood levels collated from all the observed historic storm events to modelled design flood levels;
- comparing the modelled design results against the results in the 1998 report by SWC;
- comparing the modelled design results against the results in the 2004 report by Brown Consulting (NSW); and
- comparing the modelled design results against the results in the 2007 report by ACOR Consultants.

6.4.1. Comparison with observed historic flood levels

The number of properties for which flooding is reported to have occurred, including those with no date or no depth specified, affected by various magnitudes of design storm events are shown in Table 18. It is noted that there are some properties that are not affected in the 1% AEP event for which flooding has been reported. However, the flooding reported for these properties include those with ponding of water on the property, sewage backing up within the property, localised or private drainage issues, or no information given. Furthermore, no flood depths, against which verification could be undertaken, were specified for the properties not affected by the 1% AEP event.

Table 18: Comparison of properties with reported flooding and results from design storm events

	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Number of properties with any reported flooding – Not affected by flooding in hydraulic model	25	20	17	17	16	15
Number of properties with any reported flooding – Affected by flooding in hydraulic model	122	127	130	130	131	132

Indicative depths provided by the community and SWC for events occurring subsequent to 1980 have been compared against the current model results for design event rainfall, shown in Table 19. Even across this time span (1980 to date); it is possible that the catchment conditions have changed, such as increased impervious area or altered land use zoning etc. However, no information is available that would allow these changes to be quantified or incorporated into the model.

Table 19: Peak Flood Depths (m) – Indicative results (events with pluviometer stations) compared to the design events in the current study results

Storm Events	Approximate ARI (Storm Duration 60 min)	Pluviometer Stations in Operation	Indicative Depth	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI
February 1980	1 – 2 year ARI	566026	N/A						
March 1983	1 – 2 year ARI	566026	0.08	0.14	0.23	0.25	0.28	0.31	0.34
			0.10	0.00	0.02	0.29	0.62	0.94	1.22
			0.10	0.03	0.13	0.19	0.25	0.30	0.35
			0.10	0.05	0.11	0.32	0.64	0.96	1.25
			0.10	0.05	0.14	0.28	0.45	0.64	0.85
			0.30	0.02	0.24	0.38	0.48	0.54	0.59
			0.50	0.24	0.37	0.44	0.50	0.56	0.62
August 1986	2 – 5 year ARI < 1 year ARI	566020 566026	0.10	0.21	0.33	0.39	0.46	0.53	0.59
			0.50	0.05	0.14	0.28	0.45	0.64	0.85
			0.90	0.00	0.06	0.26	0.58	0.91	1.19
November 1988	1 – 2 year ARI < 1 year ARI	566020 566026	N/A						
1998	2 – 5 year ARI 5 – 10 year ARI 2 – 5 year ARI 1 – 2 year ARI	566020 566026 566065 566112	0.25	0.07	0.15	0.20	0.27	0.34	0.64
2008	1 – 2 year ARI < 1 year ARI < 1 year ARI	566020 566026 566065	N/A						
2010	< 1 year ARI < 1 year ARI 2 – 5 year ARI	566020 566026 566065	N/A						
2011	< 1 year ARI 2 – 5 year ARI < 1 year ARI	566020 566026 566065	0.40	0.14	0.17	0.19	0.21	0.22	0.24
8 March 2012	< 1 year ARI < 1 year ARI < 1 year ARI	566020 566026 566065	0.17	0.17	0.25	0.28	0.31	0.34	0.38
			0.30	0.22	0.26	0.28	0.30	0.32	0.35
			0.42	0.14	0.17	0.19	0.21	0.22	0.24
April 2012	< 1 year ARI < 1 year ARI < 1 year ARI	566020 566026 566065	N/A						
May 2012	< 1 year ARI < 1 year ARI < 1 year ARI	566020 566026 566065	N/A						

6.4.2. Comparison with the SWC (1998) report

Comparison was undertaken on the 20% AEP peak flows produced in the TUFLOW hydraulic model and those in the SWC report, summarised in Table 20.

Table 20: SWC (1998) results compared to the current study results – for the 20% AEP event

Pipe/Channel ID	Branch	Land Feature	SWC Report (1998) (m^3/s)	Current Study (m^3/s)
A-B	Main Branch	Open Channel	105.0	77.2
B-C	Main Branch	Open Channel	105.1	73.3
D-E	Main Branch	Culvert under Ramsay St	98.3	62.2
HA-HB	Main Branch	Open Channel	84.0	51.8
J-K	Main Branch	Culvert under Church St	82.8	49.1
L-M	Main Branch	Open Channel	58.6	43.9
N-O	Main Branch	Culvert under John St	58.9	39.2
RC-RD	Main Branch	Culvert under Banks St	52.9	34.8
T-U	Main Branch	Culvert under Elizabeth St	46.5	30.3
W-X	Main Branch	Culvert under Railway	31.1	20.2
VA1-VA2	Main Branch	Culvert under Railway	14.3	11.6
ZBB-ZC	Main Branch	Open Channel	24.4	20.8
ZC1-ZD	Main Branch	Culvert under Hume Hwy	15.0	11.4
ZE-ZF	Main Branch	Culvert under Norton St	14.1	11.1
C4-C5	Chidgeys Branch	Pipe under corner of Alt St and Martin St	6.5	2.8
H13-H14	Alt St Branch	Pipe under Alt St (adjacent to Parramatta Road)	9.9	5.0
L34-L35	Croydon Branch	Pipe under Railway (near Reed St)	10.8	5.9
L46-L46A	Croydon Branch	Pipe under Railway (near Burwood Town Centre)	4.2	1.5

Peak flows in the current study were consistently less than the previous study. These differences were greater in the downstream sections of the main channel.

6.4.3. Comparison with the Brown Consulting (2004) report

Peak flood depths and peak flows detailed in the Brown Consulting report were compared to those produced by the current study in the TUFLOW hydraulic model, summarised in Table 21.

Table 21: Brown Consulting (2004) results compared to current study results

Location	10% AEP		1% AEP	
	Brown Consulting Q (m^3/s)	Current Study Q (m^3/s)	Brown Consulting Q (m^3/s)	Current Study Q (m^3/s)
Dobroyd Centre-North Catchment				
Overflow from Appian Way through properties into Wyatt St	2.0	2.2	3.0	3.6
Overflow from Weldon St to Tahlee St	6.9	4.7	11.1	8.5
Ponding depth at Paisley Rd	1.1 (m)	1.3 (m)	2.1 (m)	2.0 (m)
Ponding depth within the Christadelphian Bible Studies Centre grounds at 72/74 Paisley St	1.4 (m)	1.1 (m)	1.5 (m)	1.2 (m)
Overflow from 3 Albert Cres into Brand St	1.8	1.7	2.9	2.7
Overflow from vicinity of Brand St into Webb St	3.8	2.5	5.7	5.7
Overflow from Irrara St into Young St through houses	4.4	2.2	11.3	6.6
Overflow from Young St, north through properties into Wright St	4.9	2.9	11.9	7.5
Overflow from Wright St to Robinson St	5.5	3.3	12.2	8.1
Overflow from Robinson St to Ivanhoe Rd	5.6	3.4	12.1	8.6
Ponding depth within Queen St at the low point near No. 2	1.2 (m)	1.5 (m)	1.6 (m)	1.7 (m)
Peak catchment overland outflow at No. 2 Queen Street (the boundary with Ashfield LGA)	5.2	3.4	11.7	9.6
Peak catchment pipe outflow at No. 2 Queen Street (the boundary with Ashfield LGA)	8.7	8.5	8.8	7.9
Dobroyd South Catchment (Badminton Branch)				
Overflow from Badminton Road (North) to Austin Avenue (through properties)	0.3	0.6	0.5	0.9
Overflow from Badminton Road (South) to Austin Avenue (through properties)	3.5	2.8	5.6	4.5
Overflow from Austin Avenue to Brighton Street (through properties)	4.9	4.2	8.1	7.4
Overflow from Brighton Street to Croydon Avenue (through properties)	5.4	5.6	9.4	10.2
Overflow out of Gala Avenue cul-de-sac	0.2	0.2	0.4	0.3
Overflow from Croydon Avenue to Greenhills Street (through properties)	6.3	6.3	11.9	11.6

Overflow across Greenhills Street	6.3	6.8	12.0	12.6
Dobroyd South Catchment (Main Branch)				
Overflow from Ardgryffe Street to Waratah Street (through property)	1.2	2.3	1.9	3.6
Overflow from Boyle Street to Beaufort Street (through property)	3.1	2.9	5.9	5.3
Overflow from Seymour Street to Beresford Avenue (through School)	4.5	3.3	8.3	6.2
Overflow from Beresford Avenue to Brighton Street (through properties)	5.3	2.7	9.4	6.6
Overflow from Croydon Avenue to Greenhills Street (through property)	6.9	4.1	11.8	9.1

The current results compared variably to the Brown Consulting (2004) results, however given the differences in methodology this is not unreasonable.

Additionally, comparison was made between the 1% AEP flood extent obtained in the current study with the hotspots identified in the preceding Robinson GRC Consulting (2002) report, shown on Figure 5B. It was found that the hotspots identified in the previous report coincided with the flow paths identified in the current study.

6.4.4. Comparison with the ACOR Consultants (2007) report

Comparison was undertaken on the peak flows produced in the TUFLOW hydraulic model and those in the ACOR Consultants report, summarised in Table 22.

Table 22: Peak flow comparison between hydraulic model and ACOR Consultants report

Sub Catchment	5% AEP		1% AEP	
	ACOR Consultants Q (m^3/s)	Current Study Q (m^3/s)	ACOR Consultants Q (m^3/s)	Current Study Q (m^3/s)
Elizabeth St – 3	74	45	121	56
John St – 4	83	51	136	66
Burwood – 5 (Croydon Road)	22	12	37	15
Alexandra St – 6	107	63	176	84
Parramatta Rd – 7	133	82	220	110
Iron Cove – 9	170	101	279	139

Peak flows in the current study were significantly less than those in the previous study. The peak flows produced in the previous study were obtained using the DRAINS hydrologic model and did not explicitly account for storage within the catchment. Within the Dobroyd Canal catchment, this has a significant influence due to parks that act as detention basins and obstructions such as the railway embankment impeding flow.

The Heighway Avenue and Paisley Road "hotspots" (discussed in Section 9) are the most significant examples of impeded flow and are caused by the limited conveyance capacity through the railway embankment. These hotspots are located upstream of the ACOR hydraulic study area, within the Burwood sub-catchment (in the case of the Paisley Road Hotspot) and upstream of the Elizabeth Street sub-catchment (in the case of the Heighway Avenue Hotspot).

The ACOR study assumed that all overland flow occurred along roadways and discharges into the open channel. This contrasts to results within the current study, which shows significant flow through private property, perpendicular to the roadway. As such, the attenuation of flow that occurs due to the combination of these factors is significant and their exclusion from the ACOR hydrologic model accounts for differences in peak flow results.

The highest 1% AEP flood level across the cadastral lot was also compared to the previous study. Modelled results in the current study produce a peak flood level of 5.5 m AHD compared to 6.4 m AHD in the previous study. Given that peak flows vary significantly between the two studies, it is unsurprising that the peak flood levels within the property are significantly lower in the current study.

6.5. Discussion

Although the available data within the Dobroyd Canal catchment was insufficient to undertake a comprehensive calibration of the models, a comprehensive verification of the models has been carried out. Furthermore, the Dobroyd Canal catchment has strong similarities to the adjacent Hawthorne Canal catchment, which was calibrated. These similarities include catchment conditions, parameter adoption and methodology.

In totality, the comparison to specific yield rates for similar areas in the Sydney Metropolitan region, similarity to the Hawthorne Canal Flood Study (which was calibrated), the comparison with previous studies, and sensitivity analysis provide a strong confidence in the model and the model results (within reasonable tolerance).

7. DESIGN EVENT MODELLING

7.1. Overview

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

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

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

7.2. Critical Duration

To determine the critical storm duration for various parts of the catchment, modelling of the 1% AEP event was undertaken for a range of design storm durations from 15 minutes to 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 a combination of the 25 minute, 1 hour and 2 hour design storm durations were critical across the whole catchment for the 1% AEP event. The 25 minute design storm duration was mostly critical in areas of shallow overland flow, with 92% of the area considered critical in this storm duration having a peak flood depth no greater than 0.3 m. As such, the 25 minute storm burst was disregarded as a critical storm burst. The 1 hour storm duration was critical over a greater area than the 2 hour storm duration, both of which occur along the main drainage lines. However, the height difference between the two durations was within ± 0.025 m across 90% of the area affected by these two durations. Furthermore, the 1 hour design storm duration was mostly critical in the area upstream of the railway embankment on Heighway Avenue, which has been classified a "hotspot" (discussed in Section 10.1.1)

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 generally critical in the open channel sections and the trunk drainage system that extends from Croydon Road up to and including the Paisley

Road "hotspot" (discussed in Section 10.1.2). Between the two durations, the locations with the largest height difference were Timbrell Park, the junction of two open channel branches between Church Street and John Street, and the Heighway Avenue "hotspot". In these locations the 1 hour storm duration was greater than the 30 minute storm duration by 0.3 m to 1 m and accounted for 15% of the total area affected by these two durations.

Based on this outcome, it was considered appropriate to adopt the 1 hour storm burst for all events.

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 each design rainfall event is summarised in Table 23.

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).

Table 23: Design Rainfall Event and Downstream Boundary Conditions

Design Event (AEP)	Rainfall Event	Ocean Level
50% AEP	50% AEP Rainfall	50% AEP Ocean Level 1.28 m AHD
20% AEP	20% AEP Rainfall	20% AEP Ocean Level 1.32 m AHD
10% AEP	10% AEP Rainfall	10% AEP Ocean Level 1.35 m AHD
5% AEP	5% AEP Rainfall	5% AEP Ocean Level 1.38 m AHD
2% AEP	2% AEP Rainfall	5% AEP Ocean Level 1.38 m AHD
1% AEP (Enveloped)	1% AEP Rainfall	5% AEP Ocean Level 1.38 m AHD
	5% AEP Rainfall	1% AEP Ocean Level 1.44 m AHD
PMF	Probable Maximum Precipitation	1% AEP Ocean Level 1.44 m AHD

7.4. Design Results

The results from this study are presented as:

- Peak flood level profiles in Figure 11;
- Flow and level hydrographs in Figure 12;
- Peak flood depths and level contours in Figure 13 to Figure 19;
- Peak flood velocities in Figure 20;
- Provisional hydraulic hazard in Figure 21 to Figure 24;
- Provisional hydraulic categorisation in Figure 25 to Figure 28;
- Preliminary flood emergency response classification of communities in Figure 30; and
- Preliminary flood planning areas in Figure 31.

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

The results have been provided to Ashfield City Council and Burwood City 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 10.

A tabulated summary of peak flood depth and level results at key locations are detailed in Table 24.

Table 24: Peak Flood Levels (m AHD) and Depths (m) at Key Locations

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
H01	Open Channel – Upstream of Timbrell Dr	Level	1.33	1.39	1.45	1.55	1.63	1.77	2.89
		Depth	2.37	2.43	2.48	2.57	2.65	2.78	3.83
H02	Timbrell Drive	Level	N/A	N/A	N/A	N/A	N/A	N/A	2.72
		Depth	N/A	N/A	N/A	N/A	N/A	N/A	1.23
H03	Dobroyd Parade	Level	2.10	2.16	2.18	2.21	2.22	2.23	2.99
		Depth	0.82	0.88	0.90	0.93	0.94	0.95	1.71
H04	Open Channel – Downstream of Parramatta Rd	Level	2.13	2.34	2.49	2.72	2.96	3.19	5.50
		Depth	1.51	1.70	1.84	2.06	2.28	2.49	4.71
H05	Open Channel – Upstream of Church St	Level	4.41	4.53	4.70	4.93	5.17	5.38	8.43
		Depth	2.38	2.50	2.66	2.90	3.14	3.35	6.40
H06	Open Channel – Upstream of Banks St	Level	9.17	9.74	10.02	10.14	10.23	10.30	11.29
		Depth	2.38	2.88	3.17	3.29	3.38	3.44	4.44
H07	Heighway Avenue	Level	13.26	13.30	13.57	13.89	14.21	14.50	17.48
		Depth	0.37	0.41	0.68	1.00	1.32	1.61	4.59
H08	Norton Street	Level	16.50	16.67	16.77	16.88	16.98	17.07	17.93
		Depth	0.45	0.62	0.72	0.84	0.94	1.03	1.89
H09	Hume Highway	Level	17.05	17.30	17.41	17.53	17.64	17.73	18.64
		Depth	0.26	0.51	0.62	0.74	0.84	0.94	1.85
H10	Brown Street	Level	21.89	22.12	22.22	22.33	22.42	22.52	23.23
		Depth	2.09	2.31	2.41	2.52	2.62	2.71	3.43
H11	Frederick Street	Level	9.02	9.12	9.21	9.29	9.35	9.41	9.92
		Depth	0.13	0.23	0.32	0.40	0.46	0.52	1.03
H12	Queen Street	Level	7.58	8.49	8.78	8.88	8.97	9.03	10.02
		Depth	0.27	1.18	1.46	1.56	1.65	1.72	2.70
H13	Webb Street	Level	15.28	15.40	15.45	15.51	15.57	15.63	16.58
		Depth	0.61	0.72	0.77	0.83	0.89	0.95	1.91
H14	Paisley Road	Level	18.51	18.92	19.13	19.38	19.64	19.87	21.85
		Depth	0.65	1.06	1.28	1.53	1.79	2.02	4.00

The tabulated summary of peak flows at key locations is presented in Table 25.

Table 25: Peak Flows (m^3/s) at Key Locations

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Q01	Open Channel – Upstream of Timbrell Dr	Overland	0.0	0.1	0.1	0.1	0.1	0.4	369.2
		Pipe/Channel	62.3	77.2	87.1	100.5	110.4	139.1	252.2
Q02	Open Channel – Downstream of Parramatta Rd	Overland	0.7	1.6	3.9	7.2	9.6	12.3	234.3
		Pipe/Channel	51.0	59.2	65.5	74.7	86.9	98.0	251.0
Q03	Open Channel – Upstream of Banks St	Overland	0.0	0.0	0.7	2.0	3.8	5.4	88.1
		Pipe/Channel	26.4	34.8	41.8	46.6	51.9	56.6	147.6
Q04	Under Railway Embankment – Heighway Ave	Overland	0.0	0.0	0.0	0.0	0.1	0.6	93.0
		Pipe/Channel	13.0	20.3	27.1	31.6	36.6	40.3	78.8
Q05	Open Channel – Downstream of Hume Hwy	Overland	1.4	2.1	2.4	2.8	5.3	8.4	79.8
		Pipe/Channel	13.8	22.7	23.7	27.7	32.0	35.8	79.4
Q06	Hume Highway	Overland	3.1	8.9	12.9	17.8	22.8	28.0	112.0
		Pipe/Channel	5.3	5.2	5.2	5.2	5.2	5.2	4.7
Q07	Bland Street	Overland	0.3	1.4	2.2	3.2	4.2	5.3	23.0
		Pipe/Channel	0.6	0.7	0.7	0.7	0.7	0.7	0.7
Q08	Frederick Street	Overland	1.5	3.2	6.1	9.6	13.2	16.6	78.3
		Pipe/Channel	3.7	3.8	3.7	3.9	3.8	3.9	4.2
Q09	Queen Street	Overland	0.0	0.0	1.6	3.5	5.9	8.0	108.9
		Pipe/Channel	8.0	8.6	8.5	8.5	7.9	8.5	8.1
Q10	Webb Street	Overland	1.2	3.0	4.2	5.6	7.1	8.7	62.9
		Pipe/Channel	6.4	6.4	6.5	6.5	6.5	6.6	6.7
Q11	Under Railway Embankment – Paisley Rd	Overland	0.0	0.0	0.0	0.0	0.0	0.0	14.9
		Pipe/Channel	5.2	5.9	6.3	6.6	7.0	7.2	8.7

The tabulated summary of peak velocities within the open channel and overtopping structures traversing the open channel is presented in Table 26.

Table 26: Peak Velocities (m/s) in Open Channel

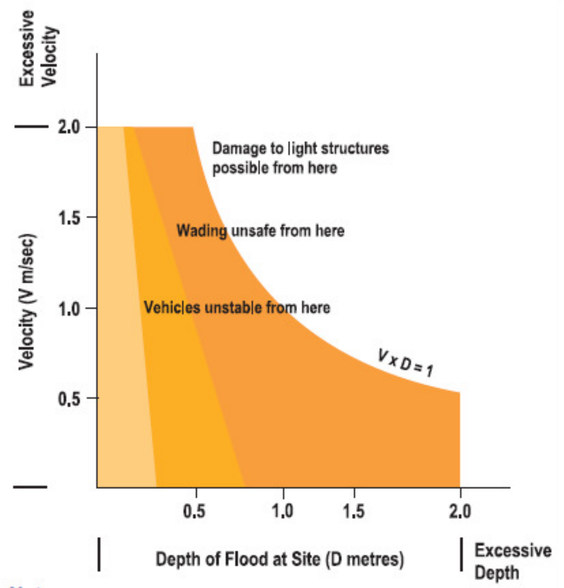
Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Timbrell Dr	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	0.6
Upstream of Timbrell Dr	Open Channel	1.4	1.6	1.8	2.0	2.1	2.7	3.9
Ramsey Rd	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	1.7
Upstream of Ramsey Rd	Open Channel	2.1	2.2	2.3	2.4	2.5	2.6	2.6
Parramatta Rd	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	1.8
Upstream of Parramatta Rd	Open Channel	2.3	2.3	2.4	2.4	2.6	2.7	5.9
Church St	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	2.3
Upstream of Church St	Open Channel	2.5	2.6	2.7	2.7	2.7	2.7	6.2
John St	Overtopping Structure	0.0	0.0	0.0	0.7	0.9	1.1	2.5
Upstream of John St	Open Channel	4.3	4.5	4.5	4.6	4.6	4.9	14.3
Banks St	Overtopping Structure	0.0	0.0	0.0	0.2	0.5	0.7	1.7
Upstream of Banks St	Open Channel	2.7	2.9	3.2	3.5	3.8	4.1	8.5
Elizabeth St	Overtopping Structure	0.0	0.3	0.6	0.8	0.9	1.0	1.8
Upstream of Elizabeth St	Open Channel	2.3	2.7	3.1	3.5	3.7	3.9	6.0
Heighway Ave	Overtopping Structure	0.0	0.0	0.5	0.8	0.8	0.8	2.0
Upstream of Heighway Ave	Open Channel	2.4	2.7	2.9	3.2	3.3	3.3	3.4
Liverpool Rd	Overtopping Structure	0.0	0.0	0.0	0.5	0.8	0.9	1.9
Upstream of Liverpool Rd	Open Channel	3.0	3.1	3.2	3.2	3.6	3.9	5.9

7.4.2. Provisional Flood Hazard Categorisation

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

Maps of provisional hydraulic hazard in the Dobroyd Canal catchment are presented in Figure 21 to Figure 24.

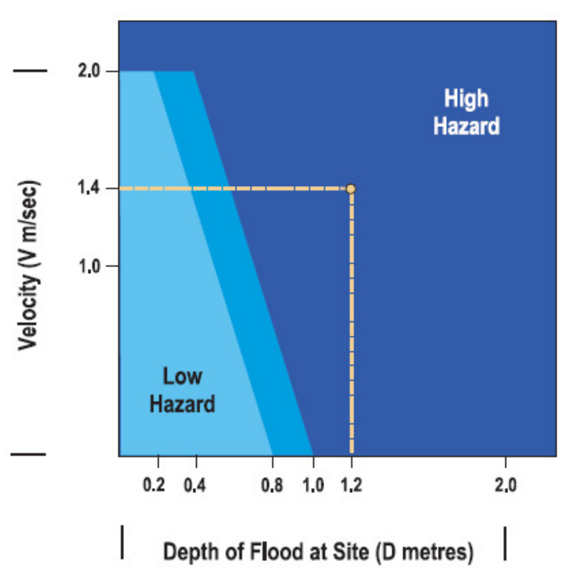
Diagram 2: (L1) Velocity and Depth Relationship; (L2) Provisional Hydraulic Hazard Categories (NSW State Government, 2005)



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):

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

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

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

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 25, Figure 26, Figure 27 and Figure 28 show the provisional hydraulic categorisations for the Dobroyd Canal catchment for the 20% AEP, 5% AEP, 1% AEP and PMF events respectively.

7.4.4. Preliminary Flood Emergency Response Classification of Communities

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

Criteria for determining flood ERP classifications and an indication of the emergency response required for these classifications are provided in the Floodplain Risk Management Guideline, 2007 (Flood Emergency Response Planning: Classification of Communities). Table 27 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 27: 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 Dobroyd 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 Dobroyd Canal catchment is shown in Figure 30. Areas that are likely to be isolated due to floodwater and contain properties that are likely to be inundated were classified as either Low Flood Island (LFI) or Low Trapped Perimeter (LTP) Areas. These high priority areas include properties along Dobroyd Parade, Queen Street, Heighway Avenue and Paisley Road. The areas classified as Rising Road Access are likely to be inundated but have roads rising uphill and away from the rising floodwaters. Therefore, residents should not be trapped unless they delay 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 sub-catchments;
- 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 all culverts and bridges over open channel was assessed for 20% and 50% blockage;
- Blockage (railway embankment): Sensitivity to blockage of key drainage infrastructure underneath the railway embankment was assessed for 20% and 50% blockage;
- Climate Change (Rainfall Increase): Sensitivity to rainfall/runoff estimates were assessed by increasing the rainfall intensities by 10%, 20% and 30% as recommended under current guidelines;
- Climate Change (Sea Level Rise): Sea level rise scenarios of 0.4 m and 0.9 m were assessed.

These sensitivity scenarios were undertaken for the 1% AEP rainfall event with the 5% AEP ocean level.

8.2. Climate Change Background

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

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

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

8.2.1. Rainfall Increase

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

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

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

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

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

8.2.2. Sea Level Rise

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

The Policy Statement says:

> *"Over the period 1870-2001, global sea levels rose by 20 cm, with a current global average rate of increase approximately twice the historical average. Sea levels are*

> *expected to continue rising throughout the twenty-first century and there is no scientific evidence to suggest that sea levels will stop rising beyond 2100 or that current trends will be reversed… However, the 4th Intergovernmental Panel on Climate Change in 2007 also acknowledged that higher rates of sea level rise are possible"* (NSW State Government, 2009)

In light of this uncertainty, the NSW State Government's advice is subject to periodical review. As of 2012 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.

8.3. Results

The sensitivity scenario results were compared to the 1% AEP rainfall event with the 5% AEP ocean level. A summary of peak flood level and peak flow differences at various locations are provided in:

- Table 28 and Table 29 for variations in routing and roughness;
- Table 30 and Table 31 for variations in blockage;
- Table 32 and Table 33 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.

8.3.1. 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 1% AEP results to determine the Flood Planning Levels.

However, decreasing the roughness parameter resulted in increased peak flood levels at two key locations. These locations (Timbrell Drive and the open channel section upstream of Timbrell Drive) 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 this flow constriction.

Table 28: Results of Sensitivity Analysis – 1% AEP Depths (m)

ID	Location	Peak Flood Depth 1% AEP	Difference with 1% AEP (m)			
			Routing Decreased by 20%	Routing Increased by 20%	Roughness Decreased by 20%	Roughness Increased by 20%
H01	Open Channel – Upstream of Timbrell Dr	2.78	0.08	0.09	0.13	0.04
H02	Timbrell Drive	0.00	0.09	0.10	0.15	0.08
H03	Dobroyd Parade	0.95	0.00	0.00	0.00	0.00
H04	Open Channel – Downstream of Parramatta Rd	2.49	0.00	0.00	0.07	-0.02
H05	Open Channel – Upstream of Church St	3.35	0.00	0.00	0.07	-0.08
H06	Open Channel – Upstream of Banks St	3.44	0.00	0.00	0.00	-0.01
H07	Heighway Avenue	1.61	0.00	-0.01	0.05	-0.05
H08	Norton Street	1.03	0.00	0.00	-0.02	0.02
H09	Hume Highway	0.94	0.00	0.00	-0.01	0.03
H10	Brown Street	2.71	0.01	-0.01	-0.01	0.02
H11	Frederick Street	0.52	0.00	0.00	-0.01	0.01
H12	Queen Street	1.72	0.00	0.00	0.00	0.00
H13	Webb Street	0.95	0.00	0.00	-0.02	0.02
H14	Paisley Road	2.02	0.00	0.00	0.01	0.00

Table 29: Results of Sensitivity Analysis – 1% AEP Flows (m^3/s)

ID	Location	1% AEP	Routing Decreased by 20%	Routing Increased by 20%	Roughness Decreased by 20%	Roughness Increased by 20%
Q01	Open Channel – Upstream of Timbrell Dr					
	Overland	0.4	0.9	1.1	2.4	0.4
	Pipe/Channel	139.1	127.4	129.1	124.5	133.5
Q02	Open Channel – Downstream of Parramatta Rd					
	Overland	12.3	12.3	12.2	12.6	11.3
	Pipe/Channel	98.0	98.1	97.8	101.1	94.9
Q03	Open Channel – Upstream of Banks St					
	Overland	5.4	5.4	5.4	6.7	4.2
	Pipe/Channel	56.6	56.7	56.5	57.1	56.2
Q04	Under Railway Embankment – Heighway Ave					
	Overland	0.6	0.6	0.7	1.0	0.3
	Pipe/Channel	40.3	41.4	40.3	41.6	39.4
Q05	Open Channel – Downstream of Hume Hwy					
	Overland	8.4	8.4	8.1	9.7	7.7
	Pipe/Channel	35.8	35.9	35.8	36.4	34.6
Q06	Hume Highway					
	Overland	28.0	28.3	27.8	29.0	27.0
	Pipe/Channel	5.2	5.1	5.2	5.2	5.1
Q07	Bland Street					
	Overland	5.3	5.3	5.2	5.4	5.1
	Pipe/Channel	0.7	0.7	0.7	0.7	0.7
Q08	Frederick Street					
	Overland	16.6	16.8	16.5	17.5	15.6
	Pipe/Channel	3.9	3.9	3.9	3.9	3.9
Q09	Queen Street					
	Overland	8.0	8.0	8.0	8.9	7.2
	Pipe/Channel	7.9	7.9	7.9	7.9	7.9
Q10	Webb Street					
	Overland	8.7	8.8	8.7	9.1	8.3
	Pipe/Channel	6.6	6.6	6.6	6.6	6.6
Q11	Under Railway Embankment – Paisley Rd					
	Overland	0.0	0.0	0.0	0.0	0.0
	Pipe/Channel	7.2	7.2	7.2	7.2	7.2

8.3.2. 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. Where the flood level varied by greater than 0.1 m (at the Paisley Road hotspot) this sensitivity appears to be dominated by the pipes under the railway embankment, with little to no impact from blockage of surrounding pipes. As such, there is no difference in levels between the scenario with all pipes blocked and the scenario with only the railway embankment pipes blocked.

Generally, blockage of all 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.

Blockage of the drainage infrastructure under the railway embankment resulted in increased flood levels immediately upstream of the embankment and decreased flood levels along the flow paths downstream of the embankment.

Table 30: Results of Blockage Analysis – 1% AEP Depths (m)

ID	Location	Peak Flood Depth 1% AEP	Difference with 1% AEP (m)					
			Blockage (Pipes) by 20%	Blockage (Pipes) by 50%	Blockage (Bridges) by 20%	Blockage (Bridges) by 50%	Blockage (Railway) by 20%	Blockage (Railway) by 50%
H01	Open Channel – Upstream of Timbrell Dr	2.78	-0.01	0.05	0.21	0.32	0.04	-0.02
H02	Timbrell Drive	0.00	0.00	0.08	0.21	0.35	0.08	0.00
H03	Dobroyd Parade	0.95	0.00	0.00	0.00	0.00	0.00	0.00
H04	Open Channel – Downstream of Parramatta Rd	2.49	-0.01	-0.02	0.28	0.93	-0.07	-0.23
H05	Open Channel – Upstream of Church St	3.35	-0.01	-0.03	0.26	1.03	-0.13	-0.40
H06	Open Channel – Upstream of Banks St	3.44	0.00	0.00	0.02	0.04	-0.06	-0.19
H07	Heighway Avenue	1.61	0.00	0.01	0.24	1.13	0.20	1.29
H08	Norton Street	1.03	0.02	0.05	0.00	0.01	0.00	0.00
H09	Hume Highway	0.94	0.03	0.06	0.00	0.00	0.00	0.00
H10	Brown Street	2.71	0.00	0.00	0.00	0.00	0.00	0.00
H11	Frederick Street	0.52	0.02	0.05	0.00	0.00	0.00	0.00
H12	Queen Street	1.72	0.03	0.07	0.01	0.02	-0.01	-0.03
H13	Webb Street	0.95	0.01	0.03	0.00	0.00	-0.02	-0.06
H14	Paisley Road	2.02	0.06	0.20	0.00	0.00	0.06	0.20

Table 31: Results of Blockage Analysis – 1% AEP Flows (m^3/s)

ID	Location	1% AEP	Blockage (Pipes) by 20%	Blockage (Pipes) by 50%	Blockage (Bridges) by 20%	Blockage (Bridges) by 50%	Blockage (Railway) by 20%	Blockage (Railway) by 50%
Q01	Open Channel – Upstream of Timbrell Dr							
	Overland	0.4	0.4	0.7	4.5	14.6	0.7	0.2
	Pipe/Channel	139.1	134.4	118.3	104.5	68.2	127.4	114.3
Q02	Open Channel – Downstream of Parramatta Rd							
	Overland	12.3	13.0	13.8	12.6	13.3	12.3	12.1
	Pipe/Channel	98.0	97.2	95.3	90.2	73.0	93.2	81.8
Q03	Open Channel – Upstream of Banks St							
	Overland	5.4	5.4	5.4	5.4	5.1	3.9	1.6
	Pipe/Channel	56.6	56.8	56.8	50.6	39.9	52.3	45.1
Q04	Under Railway Embankment – Heighway Ave							
	Overland	0.6	0.6	0.7	2.7	14.1	2.4	12.6
	Pipe/Channel	40.3	41.2	40.4	35.2	22.3	34.7	22.1
Q05	Open Channel – Downstream of Hume Hwy							
	Overland	8.4	8.3	8.2	10.0	14.5	8.3	8.3
	Pipe/Channel	35.8	35.8	35.9	34.2	29.7	35.8	35.8
Q06	Hume Highway							
	Overland	28.0	29.3	30.8	28.1	28.0	28.1	28.0
	Pipe/Channel	5.2	4.0	2.4	4.7	4.7	5.0	4.7
Q07	Bland Street							
	Overland	5.3	5.3	5.3	5.3	5.2	5.3	5.2
	Pipe/Channel	0.7	0.7	0.7	0.7	0.7	0.7	0.7
Q08	Frederick Street							
	Overland	16.6	17.9	19.9	16.6	16.6	16.6	16.6
	Pipe/Channel	3.9	3.2	2.0	3.9	3.9	3.9	3.9
Q09	Queen Street							
	Overland	8.0	9.2	10.8	8.2	8.7	7.7	7.1
	Pipe/Channel	7.9	6.3	3.8	7.9	7.8	7.9	8.0
Q10	Webb Street							
	Overland	8.7	9.0	9.7	8.7	8.7	8.2	7.0
	Pipe/Channel	6.6	5.8	3.8	6.6	6.6	6.5	6.5
Q11	Under Railway Embankment – Paisley Rd							
	Overland	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Pipe/Channel	7.2	6.5	4.9	7.2	7.2	6.5	4.9

8.3.3. Climate Variations

The effect of increasing the design rainfalls by 10%, 20% and 30% has been evaluated for the 1% AEP rainfall event with impacts on peak flood levels observed throughout the study area. Generally speaking, each incremental 10% increase in rainfall results in an approximately 0.1 m increase in peak flood levels at most of the locations analysed. The 1% AEP event with a rainfall increase of 30% is approximately equivalent to a 0.2% AEP event in present day conditions and an impact on flood levels is not unexpected.

The sea level rise scenarios were found not to have a significant effect on peak flood levels, except in the most downstream reaches of the catchment. Timbrell Drive and Timbrell Park were particularly vulnerable to sea level rise, with the lowest point along Timbrell Drive being approximately 1.5 m AHD and below the raised sea levels. In contrast, the propagation of sea level rise impacts within the open channel was found to be restricted by structures traversing the channel, particularly the Timbrell Drive Bridge. This is shown in the profiles for the sea level rise scenarios found in Figure 11.

Table 32: Results of Climate Change Analysis – 1% AEP Depths (m)

ID	Location	Peak Flood Depth 1% AEP	Difference with 1% AEP (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 Timbrell Dr	2.78	0.28	0.30	0.34	0.15	0.54
H02	Timbrell Drive	0.00	0.22	0.34	0.38	0.30	0.79
H03	Dobroyd Parade	0.95	0.01	0.02	0.02	0.01	0.13
H04	Open Channel – Downstream of Parramatta Rd	2.49	0.22	0.48	0.76	0.00	0.01
H05	Open Channel – Upstream of Church St	3.35	0.17	0.39	0.64	0.00	0.00
H06	Open Channel – Upstream of Banks St	3.44	0.06	0.14	0.19	0.00	0.00
H07	Heighway Avenue	1.61	0.20	0.37	1.09	0.00	0.00
H08	Norton Street	1.03	0.07	0.13	0.19	0.00	0.00
H09	Hume Highway	0.94	0.08	0.15	0.21	0.00	0.00
H10	Brown Street	2.71	0.08	0.15	0.21	0.00	0.00
H11	Frederick Street	0.52	0.04	0.08	0.11	0.00	0.00
H12	Queen Street	1.72	0.05	0.10	0.15	0.00	0.00
H13	Webb Street	0.95	0.04	0.08	0.12	0.00	0.00
H14	Paisley Road	2.02	0.18	0.35	0.51	0.00	0.00

Table 33: Results of Climate Change Analysis – 1% AEP Flows (m^3/s)

ID	Location	1% AEP	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 – Upstream of Timbrell Dr						
	Overland	0.4	5.6	15.7	21.1	2.8	36.1
	Pipe/Channel	139.1	135.9	126.2	135.4	123.0	99.7
Q02	Open Channel – Downstream of Parramatta Rd						
	Overland	12.3	13.9	16.3	18.5	12.3	12.3
	Pipe/Channel	98.0	107.1	117.2	126.1	98.0	97.9
Q03	Open Channel – Upstream of Banks St						
	Overland	5.4	7.1	9.8	12.2	5.4	5.4
	Pipe/Channel	56.6	61.0	67.1	71.6	56.6	56.7
Q04	Under Railway Embankment – Heighway Ave						
	Overland	0.6	2.3	5.9	10.5	0.6	0.8
	Pipe/Channel	40.3	44.2	46.3	52.8	40.3	41.3
Q05	Open Channel – Downstream of Hume Hwy						
	Overland	8.4	11.1	13.9	16.7	8.4	8.4
	Pipe/Channel	35.8	39.2	42.6	46.1	35.8	35.8
Q06	Hume Highway						
	Overland	28.0	32.6	37.0	41.2	28.0	28.0
	Pipe/Channel	5.2	5.1	5.1	5.1	5.2	5.2
Q07	Bland Street						
	Overland	5.3	6.2	7.2	8.3	5.3	5.3
	Pipe/Channel	0.7	0.7	0.7	0.7	0.7	0.7
Q08	Frederick Street						
	Overland	16.6	19.6	22.7	25.5	16.6	16.6
	Pipe/Channel	3.9	3.9	4.0	3.9	3.9	3.9
Q09	Queen Street						
	Overland	8.0	10.1	12.2	14.6	8.0	8.0
	Pipe/Channel	7.9	7.9	7.9	7.9	7.9	7.9
Q10	Webb Street						
	Overland	8.7	10.1	11.5	12.9	8.7	8.7
	Pipe/Channel	6.6	6.6	6.6	6.6	6.6	6.6
Q11	Under Railway Embankment – Paisley Rd						
	Overland	0.0	0.0	0.0	0.0	0.0	0.0
	Pipe/Channel	7.2	7.4	7.6	7.7	7.2	7.2

9. PRELIMINARY FLOOD PLANNING AREAS – PROPERTY TAGGING

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 1% AEP 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 and Criteria

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 1% AEP event. This has been defined as the peak flood level within the open channel section of Dobroyd 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 1% AEP 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 34. Figure 31 identifies the extent of mainstream or overland flow property affectation.

Table 34: Number of Properties Tagged

	Mainstream	**Overland**	**Both Mainstream and Overland**	**Total**
Ashfield	145	852	431	1428
Burwood	0	400	0	400
Total	145	1252	431	1828

A total of 1428 properties were tagged for flood related development controls in Ashfield and 400 properties in Burwood. This gives total averages of 1.7 properties per hectare for Burwood and 2.9 properties per hectare for Ashfield. Considering only overland flow affectation, the average was 1.7 properties per hectare for Ashfield Council. As such, mainstream flood affectation accounted for the difference in total average properties per hectare between the two Councils, with the open channel situated solely within Ashfield Council.

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 Dobroyd Canal Catchment. 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 Dobroyd 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. Heighway Avenue

The main open channel in the Dobroyd Canal catchment is crossed by a railway embankment that is the property of City Rail. The embankment has an elevation greater than the surrounding streets by greater than 6 m. Heighway Avenue is aligned parallel to the embankment and is directly upstream of this flow constriction.

Figure C 2 shows the 1% AEP peak flood depths at this location and the location of flood height and flow hydrographs shown in Figure C 3 and Figure C 4.

Flooding Behaviour

The contributing catchment area is approximately 286 ha, the largest of the hotspots examined. Two culverts with a cross-sectional area of approximately 14.7 m^2 and 5.3 m^2 convey flow underneath the railway embankment. The alternative route for flow from this area of the catchment is through the Frederick Street roadway tunnel. Due to the difference in elevation between the embankment and the upstream ground level, the embankment at this location is not overtopped in events up to and including the PMF.

The elevation of Frederick Street is approximately 14.0 m AHD along the roadway from the junction with Heighway Avenue to the embankment. This increases on the downstream (north) side of the embankment, with the elevation of the Frederick Street roadway found to be approximately 14.5 m AHD. By comparison, the elevation of the roadway at Heighway Avenue adjacent to the open channel is approximately 13.2 m AHD. As such, flood depths on Heighway Avenue have to reach approximately 1.3 m before the alternative flow path through the Frederick Street roadway tunnel occurs.

The obvert of the smaller culvert is 12.8 m AHD and below the elevation of the Heighway Avenue roadway. The obvert of the larger culvert is 14.86 m AHD and above the elevation of the Frederick Street roadway. As such, flow occurs through the Frederick Street roadway tunnel prior to the submergence of the larger culvert.

The peak flood depths and flows at this location are shown in Table 35 and Table 36, corresponding with those presented in Section 7.4.1.

Table 35: Heighway Avenue – Peak Flood Levels (m AHD) and Depths (m)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
H07	Heighway Avenue	Level	13.26	13.30	13.57	13.89	14.21	14.50	17.48
		Depth	0.37	0.41	0.68	1.00	1.32	1.61	4.59

Table 36: Heighway Avenue – Peak Flows (m^3/s)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Q04	Under Railway Embankment – Heighway Ave	Overland (Frederick St)	0.0	0.0	0.0	0.0	0.1	0.6	93.0
		Pipe/Channel	13.0	20.3	27.1	31.6	36.6	40.3	78.8

The Heighway Avenue hotspot has the largest area of affectation within the Dobroyd Canal Catchment in a 1% AEP event, however the duration of inundation is comparatively short and the area typically drains within 30 minutes after rainfall has ceased.

This location is very sensitive to blockage. Blockage of all bridges over the open channel and blockage of the culverts underneath the railway embankment resulted in increases in peak flood levels greater than 1 m in the case of 50% blockage (discussed in Section 8.3.2).

10.1.2. Paisley Road

The railway embankment intersects one of the major natural overland flow paths between Brady Street and Reed Street. Paisley Road, which is parallel to the railway embankment on the upstream side of this intersection, follows the natural topography and is lower in elevation than the embankment. With the exception of pipes under the embankment, this flow path is effectively blocked and water ponds to the south of the embankment along Paisley Road and surrounding streets.

Figure C 5 shows the 1% AEP peak flood depths at this location and the location of flood height and flow hydrographs shown in Figure C 6 and Figure C 7.

Flooding Behaviour

The contributing catchment area is approximately 70 ha. Two pipes, each with a cross-sectional area of approximately 2.5 m^2, convey flow underneath the railway embankment. The capacity of this pipe and the surrounding pipes in this location was found to be less than a 2 year ARI event. In a PMF event, the embankment is overtopped at this location. The peak flows within the pipe and the overland flow path across the embankment are provided in Table 37.

Table 37: Paisley Road – Peak Flows (m^3/s)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Q11	Under Railway Embankment – Paisley Rd	Overland	0.0	0.0	0.0	0.0	0.0	0.0	14.9
		Pipe/Channel	5.2	5.9	6.3	6.6	7.0	7.2	8.7

The peak flood levels and depths at this location are shown in Table 38. The ground elevation of the railway embankment was approximately 21.5 m AHD, resulting in depths of approximately 0.35 m on the embankment during a PMF event.

Table 38: Paisley Road – Peak Flood Levels (m AHD) and Depths (m)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
H14	Paisley Road	Level	18.51	18.92	19.13	19.38	19.64	19.87	21.85
		Depth	0.65	1.06	1.28	1.53	1.79	2.02	4.00

This location was found to be relatively insensitive to blockage of the trunk drainage pipes underneath the railway embankment, with increases in peak flood levels up to 0.2 m in the case of 50% blockage (discussed in Section 8.3.2).

10.1.3. Queen Street

The Queen Street low point is located in the roadway adjacent to the south-east edge of Centenary Park. The park grounds are separated from the roadway with a retaining wall and have an elevation greater than the roadway by approximately 3-4 m. The front yards of the properties opposite the park are at approximately the same elevation as the roadway.

This hotspot is located downstream of the Paisley Road hotspot and is on the border between the Burwood City Council LGA and the Ashfield City Council LGA. Downstream of this hotspot the trunk drainage pipes discharge into the open channel east of Croydon Road.

Figure C 8 shows the 1% AEP peak flood depths at this location and the location of flood height and flow hydrographs shown in Figure C 9 and Figure C 10.

Flooding Behaviour

The pipe draining this area is roughly oval shaped, with dimensions of 2.275 m (width) by 1.525 m (height) and a cross-sectional area of approximately 2.6 m^2. The capacity of this pipe and the surrounding pipes in this location was found to be less than a 2 year ARI event. The peak flows within the pipe and the overland flow path from Queen Street are provided in Table 39.

Table 39: Queen Street – Peak Flows (m^3/s)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Q09	Queen Street	Overland	0.0	0.0	1.6	3.5	5.9	8.0	108.9
		Pipe/Channel	8.0	8.6	8.5	8.5	7.9	8.5	8.1

The peak flood levels and depths at this location are shown in Table 40.

Table 40: Queen Street – Peak Flood Levels (m AHD) and Depths (m)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
H12	Queen Street	Level	7.58	8.49	8.78	8.88	8.97	9.03	10.02
		Depth	0.27	1.18	1.46	1.56	1.65	1.72	2.70

The duration of inundation was greater at this location than the other hotspots discussed, with the area generally still draining 2 hours after rainfall has ceased (for the 1 hour storm duration).

This location was found to be relatively insensitive to the various blockage scenarios assessed. Given the location of this hotspot relative to the Paisley Road hotspot, it is relevant to note that the blockage of the pipes draining the Paisley Road hotspot had minimal impact on the peak flood levels at the Queen Street hotspot, with a decrease of 0.03 m in the case of 50% blocked (discussed in Section 8.3.2).

10.1.4. Brown Street / Bland Street

The vehicle and pedestrian road tunnel underneath the railway embankment has a lower elevation than either of the two streets that approach it, namely Bland Street and Brown Street. Bland Street, which approaches the tunnel from the north side, increases in elevation by approximately 5 m from the tunnel to the junction with Elizabeth Street. Brown Street to the south of the embankment has a similar elevation rise from the tunnel to the junction with Foxs Lane. As such, the road under the railway embankment acts as a trapped low point.

Parallel to the road tunnel and approximately 70 m to the east is a pedestrian tunnel underneath the railway embankment. It has an approximate elevation 4 m higher than that of the road tunnel.

Figure C 11 shows the 1% AEP peak flood depths at this location and the location of flood height hydrographs shown in Figure C 12.

Flooding Behaviour

The peak flood depths and flows discharging from this location are shown in Table 41 and Table 42, corresponding with those presented in Section 7.4.1.

Table 41: Brown Street / Bland Street – Peak Flood Levels (m AHD) and Depths (m)

ID	Location	Type	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
H10	Brown Street	Level	21.89	22.12	22.22	22.33	22.42	22.52	23.23
		Depth	2.09	2.31	2.41	2.52	2.62	2.71	3.43

Table 42: Downstream of Bland Street – Peak Flows (m^3/s)

ID	Location	Type	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Q07	Bland Street	Overland	0.3	1.4	2.2	3.2	4.2	5.3	23.0
		Pipe/Channel	0.6	0.7	0.7	0.7	0.7	0.7	0.7

Underneath the Bland Street road tunnel is a box culvert with a width of 1.2 m and a height of 0.9 m. Although there are inlet pits at the start and end of this roadway tunnel, the pipes connecting these inlets to the box culvert were smaller than 450 mm in diameter. As such, these pipes were assumed to be blocked as per the discussion in Section 5.5.

Consequent to these pipes being blocked, a flood depth of 1.9 m was found to remain in this area and not drain away (shown in the flood height hydrograph). However, in the hypothetical scenario that the inlets connected directly to the box culvert, the 1% AEP peak flood level at this location decreased by merely 0.14 m and drained within 1 hour after rainfall ceased.

An additional feature at this location that is pertinent to the flood behaviour is an underground car park. It is located on Brown Street to the south of the railway embankment and has an entrance approximately level with the low point of the roadway. This feature was unable to be modelled due to the lack of data, particularly relating to volume capacity and private pipe drainage infrastructure. By excluding the flood storage that would be provided by the car park, the model may produce a conservative over-estimation of flood levels at this location.

10.2. Additional 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. Alexandra Street and Church Street, Ashfield

Church Street traverses the main open channel. Alexandra Street does not cross the open channel and is aligned generally perpendicular to the channel alignment. It is located upstream of Church Street, adjacent to the junction between the main open channel and the trunk drainage system originating from the Burwood-Croydon branch.

Flooding Behaviour

Within this area, there are two flood mechanisms operating. These are mainstream flooding and local overland flow flooding. The Floodplain Development Manual (2005) definition for these categories can be found in the glossary provided in Appendix A.

In events up to and including the 1% AEP event, the Church Street bridge structure is not overtopped, as demonstrated in Table 43. Furthermore, the flood level in the open channel is lower than both the ground level and the peak flood level in the surrounding Church Street area, shown in Table 44. This indicates that flow experienced on Church Street during events of this magnitude is primarily from overland flow rather than mainstream flow. However, in the PMF event the flood level exceeded the banks of the open channel and mainstream flooding was found to occur.

Table 43: Church Street – Peak Velocities (m/s)

Location	Type	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Church St	Overtopping Structure	0.0	0.0	0.0	0.0	0.0	0.0	2.3
Upstream of Church St	Open Channel	2.5	2.6	2.7	2.7	2.7	2.7	6.2

Table 44: Church Street – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Open Channel Upstream of Church Street	Level	4.41	4.53	4.69	4.93	5.17	5.38	8.44
	Depth	2.33	2.44	2.59	2.82	3.05	3.24	6.26
Church Street (Ground Level 6.36 m AHD)	Level	6.43	6.46	6.47	6.47	6.48	6.49	10.19
	Depth	0.09	0.10	0.11	0.12	0.12	0.13	3.84

The section of Alexandra Street closest to the open channel is subject to mainstream flooding in events of a magnitude greater than and including the 5% AEP event. In events of a smaller magnitude than this, the peak flood level in the open channel is less than the ground level and the peak flood level in Alexandra Street, shown in Table 45. This indicates that flow experienced on Alexandra Street during events of a magnitude smaller than the 5% AEP event is primarily from overland flow rather than mainstream flow.

Table 45: Alexandra Street – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Open Channel Adjacent to Alexandra Street	Level	4.41	4.55	4.71	4.94	5.19	5.39	8.20
	Depth	2.09	2.22	2.37	2.59	2.81	3.00	5.79
Alexandra Street (Ground Level 4.76 m AHD)	Level	4.90	4.90	4.91	4.98	5.20	5.42	8.32
	Depth	0.14	0.14	0.15	0.22	0.44	0.66	3.56

10.2.2. Algie Park, Ashfield

The ground level inside Algie Park is generally lower than the surrounding property. A concrete wall is situated along the western boundary adjacent to residential property and a grassed ridge is located along the northern boundary. Collectively, these features form a detention basin within Algie Park.

Flooding Behaviour

Flows entered Algie Park via overland flow and pipes from the east, south and west. The pipes convey flow from Bland Street, Empire Street and Ramsay Street to converge into one 0.9 m diameter pipe entering the Algie Park grounds. The pipe network draining the Algie Park detention basin consisted of two pipes with a 0.9 m diameter. The peak flows entering and discharging from the park are shown in Table 46.

Table 46: Algie Park – Peak Flows (m^3/s)

Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Inflow	Pipe	1.5	1.7	1.8	1.9	1.9	1.9	2.0
	Overland	5.1	7.3	8.8	10.5	11.8	13.5	35.6
Outflow	Pipe (east)	1.5	1.8	1.9	1.9	1.9	1.9	1.9
	Pipe (west)	0.9	1.0	1.0	1.3	1.4	1.4	1.5
	Overland (spillway)	0.0	0.0	0.1	1.0	2.0	3.4	21.7
	Overland (bypass)	0.4	1.0	1.5	2.1	2.7	3.2	5.0

The grass ridge and concrete wall had an elevation of approximately 7.0 m AHD at the northern boundary. A spillway is located on the grass ridge and had a width of 20 m and an elevation of 6.5 m AHD. The lowest point within the detention basin was approximately 4.8 m AHD and required flood depths to reach 1.7 m for the spillway to be activated.

The lowest elevation on the Ramsay Street roadway upstream of the park was approximately 8.7 m AHD. The backyard of the properties to the east of the detention basin had a lower elevation than the roadway, with elevations of 6.0 m AHD in some locations. Although a small wall was located on the southern boundary of these properties, flow that was impeded from exiting the detention basin was found to accumulate and extend upstream through the park whereby the backyards of properties to the east of the concrete wall acted as an alternative flow-path. The peak flood levels and depths within Algie Park and the streets downstream of the park are shown in Table 47.

Table 47: Algie Park – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Algie Park	Level	6.24	6.46	6.57	6.67	6.75	6.82	6.58
	Depth	1.32	1.53	1.65	1.75	1.83	1.90	1.66
Laneway Downstream of Algie Park	Level	4.20	4.24	4.28	4.35	4.42	4.48	4.92
	Depth	0.35	0.39	0.43	0.50	0.57	0.63	1.07
Alt Street	Level	3.59	3.62	3.64	3.71	3.79	3.86	4.55
	Depth	0.21	0.25	0.26	0.33	0.41	0.48	1.17
Martin Street	Level	3.30	3.36	3.39	3.45	3.59	3.70	4.36
	Depth	0.18	0.23	0.26	0.32	0.46	0.57	1.23

10.2.3. Appian Way, Burwood

Appian Way is located in the upper reaches of the catchment. The flows discharging from this area contribute to the flows received at the Paisley Road hotspot, which is situated downstream of this location.

Flooding Behaviour

The contributing catchment area is approximately 8.4 ha. The pipe draining this area has a diameter of 450 mm. When the capacity of the pipe is exceeded, overland flow occurs along the topographical low point. The topography was defined by the ALS (discussed in Section 2.3), with the natural low point found to occur through property and generally perpendicular to the roadway. The capacity of the pipe draining this location was found to be less than a 2 year ARI event. The peak flows within the pipe and the overland flow path from Appian Way are provided in Table 48.

Table 48: Appian Way – Peak Flows (m^3/s)

Location	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Overland Flow	1.2	1.8	2.2	2.7	3.1	3.6	10.1
Pipe Flow (450mm diameter)	0.3	0.3	0.3	0.3	0.3	0.3	0.3

The peak flood levels and depths adjacent to the roadway are provided in Table 49. Peak flood levels at this location were insensitive to blockage of the pipes, with a difference in peak flood levels less than 0.001 m across the various blockage scenarios assessed.

Table 49: Appian Way – Peak Flood Levels (m AHD) and Depths (m)

Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
Appian Way	Level	36.15	36.18	36.19	36.20	36.21	36.22	36.30
	Depth	0.10	0.12	0.13	0.14	0.15	0.16	0.24

10.2.4. Webb Street, Burwood

Webb Street is located between the Paisley Road hotspot and the Queen Street hotspot. Both the land and the building floor level of the Hampton Court complex (along the eastern edge of the roadway) is elevated above the level of the road. This residential block was constructed after the Brown Consulting (2004) report wherein this area was referred to as Croydon Gardens.

Flooding Behaviour

Within the previous report, flow was considered to travel from Webb Street through Croydon Gardens before being conveyed onto Irrara Street. The current conditions are such that flow from upstream of Webb Street is conveyed into Irrara Street through the roadways. Flow generated within the majority of Hampton Court is retained in an open-space detention basin that was defined in the current study by the ALS ground topography.

Underneath Webb Street two upstream branches of the trunk drainage system converge. These branches originate from Paisley Road and the Burwood Town Centre. In the vicinity of Webb Street these pipes have a diameter of 1.65 m (from the Paisley Road branch) and 0.965 m (from the Burwood Town Centre branch). The pipe downstream of this convergence is irregularly shaped, with a cross-sectional area of approximately 2.9 m^2. Flow from the detention basin is conveyed into this irregular shaped pipe via a 0.6 m diameter pipe.

The inlet draining the Hampton Court detention basin is located towards the crest of the basin, thereby restricting flows from entering the pipe until flood levels within the basin have reached the necessary height. The ground level in the area surrounding the detention basin inlet is approximately 13.1 m AHD. By comparison, the lowest ground level within the basin is approximately 11.5 m AHD. Therefore, a flood depth of 1.6 m is attained within the detention basin prior to flood waters draining into the trunk drainage system.

Peak flows in the vicinity of this location are shown in Table 50 and an ID is provided where these locations correspond with those presented in Section 7.4.1. The pipes in this area were found to be functioning at capacity in the 2 year ARI event and greater. The pipe draining the detention basin was also at capacity in the 2 year ARI event. However this was due to backflow entering the pipe from the trunk drainage system rather than from the detention basin.

Table 50: Webb Street – Peak Flows (m^3/s)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
	Upstream of Webb St – From Paisley Rd	Pipe (1.65m diameter)	4.4	4.3	4.3	4.3	4.3	4.4	3.3
	Upstream of Webb St – From Burwood Town Centre	Pipe (0.965m diameter)	1.9	1.8	1.8	1.8	1.8	1.8	1.7
Q10	Webb Street	Overland	1.2	3.0	4.2	5.6	7.1	8.7	62.9
		Pipe	6.4	6.4	6.5	6.5	6.5	6.6	6.7
	Detention Basin	Pipe (0.6m diameter)	0.0	0.5	0.5	0.5	0.5	0.5	0.3
	Irrara Street	Pipe	6.3	6.3	6.4	6.4	6.4	6.5	6.4

The peak flood levels and depths at this location are shown in Table 51, corresponding with those presented in Section 7.4.1. Peak flood levels at this location were not particularly sensitive to blockage of the pipes in the trunk drainage system.

Table 51: Webb Street – Peak Flood Levels (m AHD) and Depths (m)

ID	Location	Type	2 yr ARI	5 yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	PMF
H13	Webb Street	Level	15.28	15.40	15.45	15.51	15.57	15.63	16.58
		Depth	0.61	0.72	0.77	0.83	0.89	0.95	1.91
	Hampton Court Detention Basin	Level	12.82	13.50	13.58	13.64	13.70	13.74	14.70
		Depth	1.32	1.99	2.08	2.13	2.19	2.23	3.19

11. ACKNOWLEDGEMENTS

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

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Figures

FIGURE 1
STUDY AREA

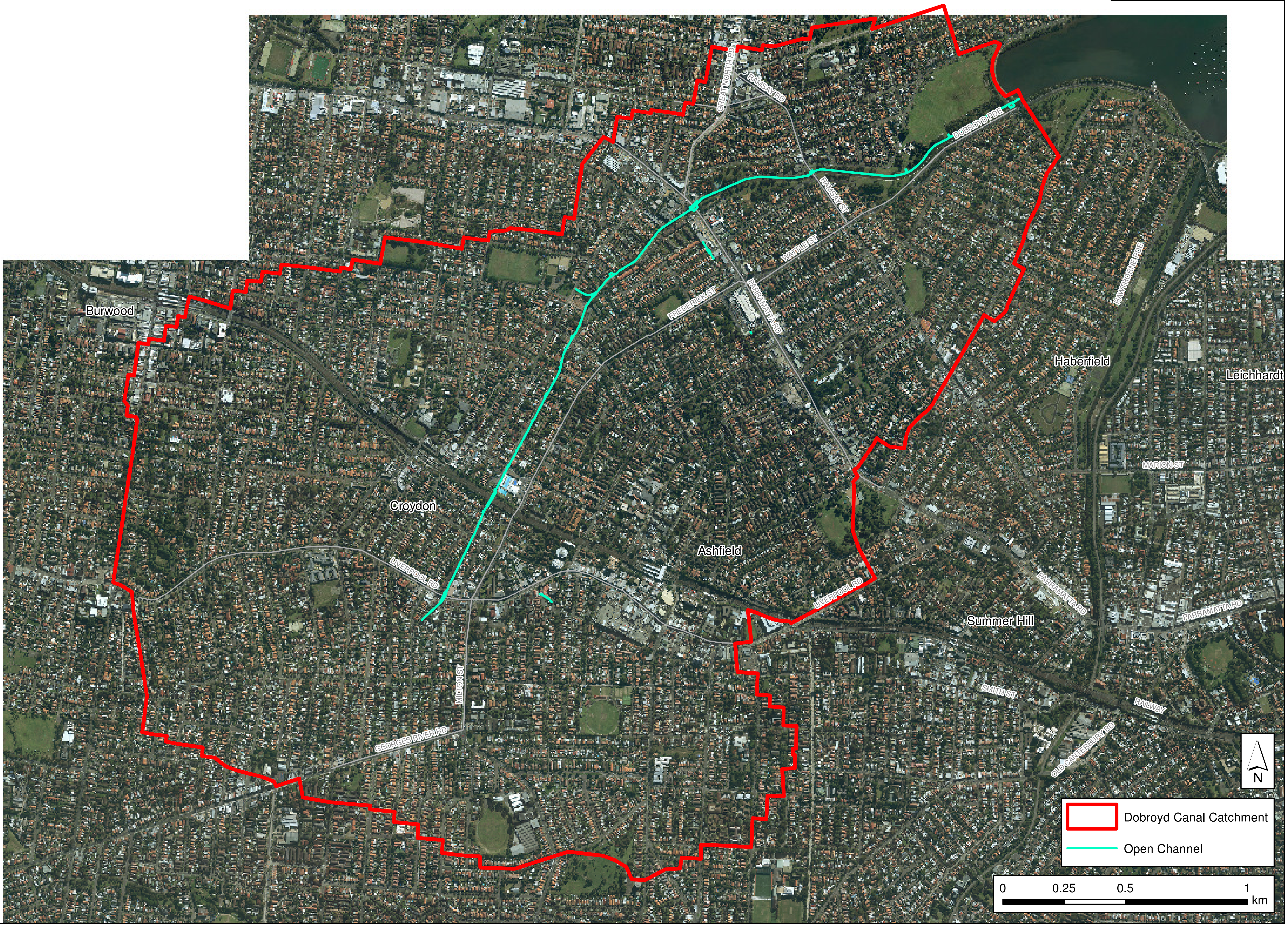

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FIGURE 2
LIDAR SURVEY DATA

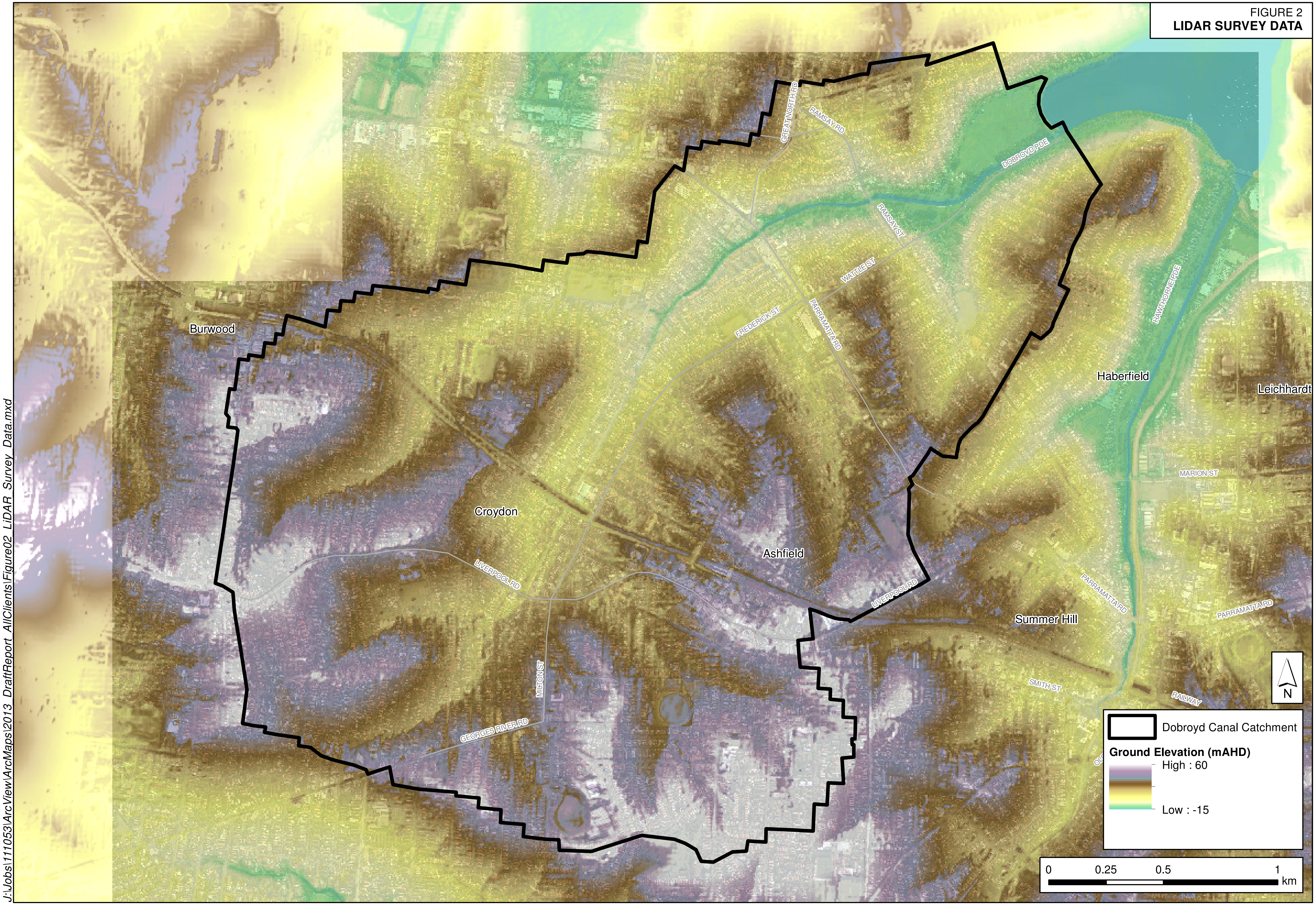

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FIGURE 3
CBH SURVEY DATA

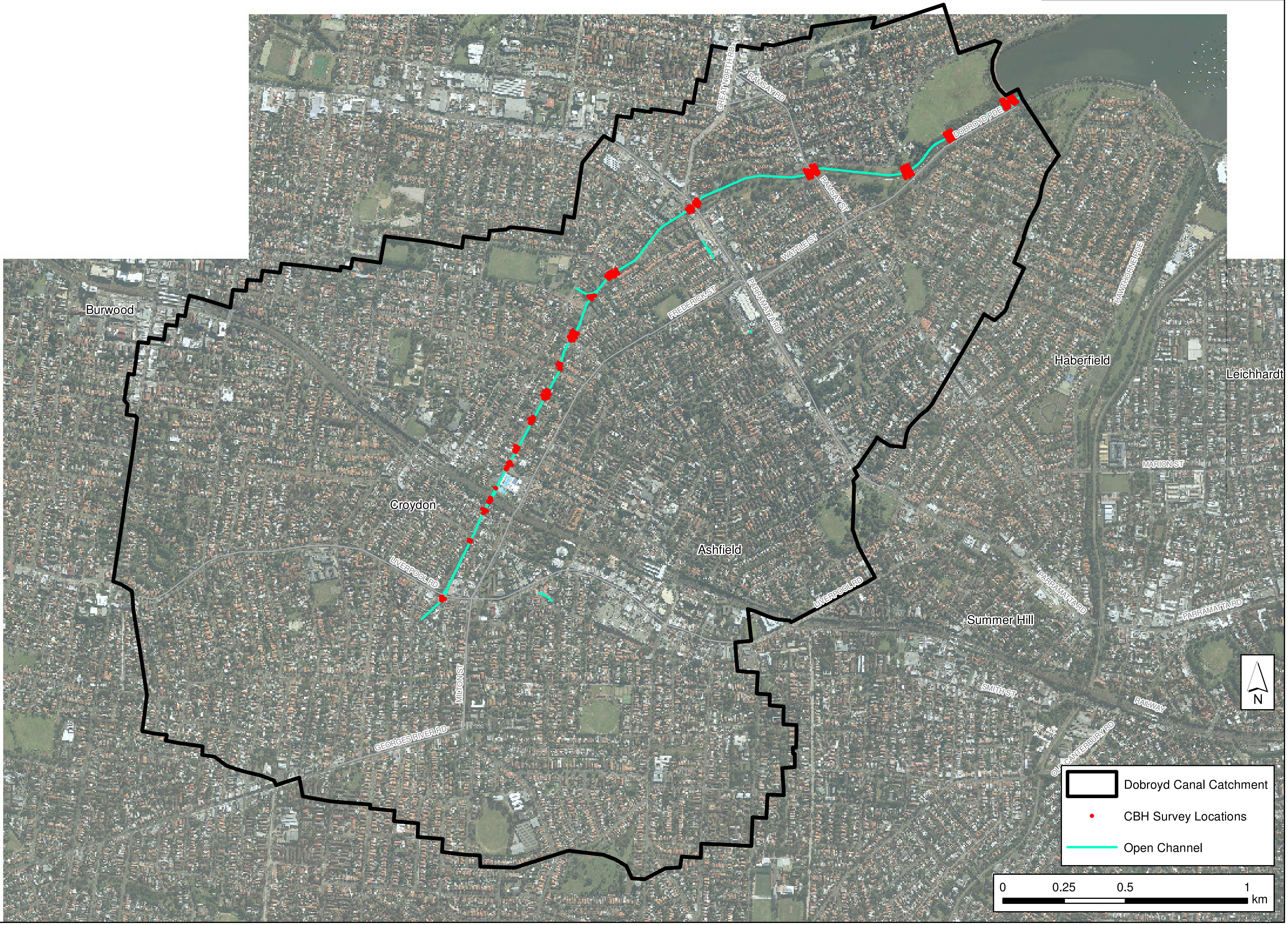

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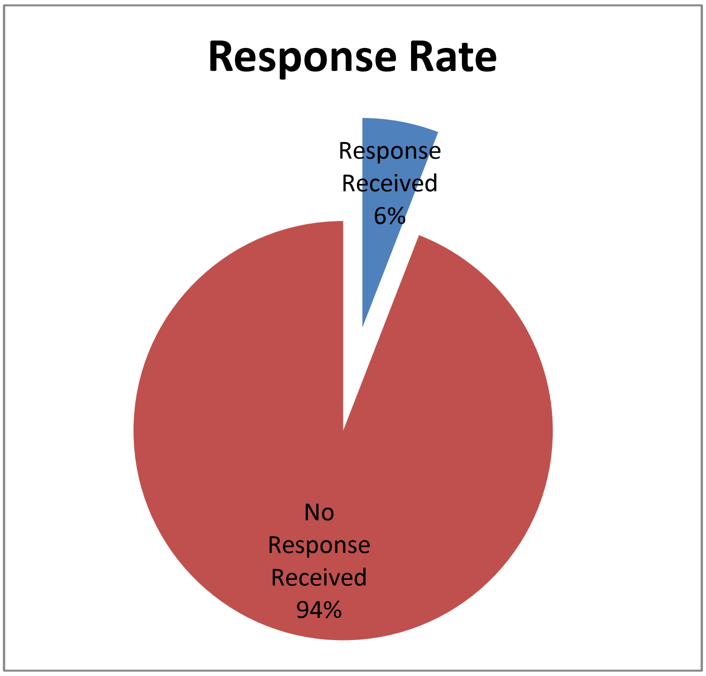
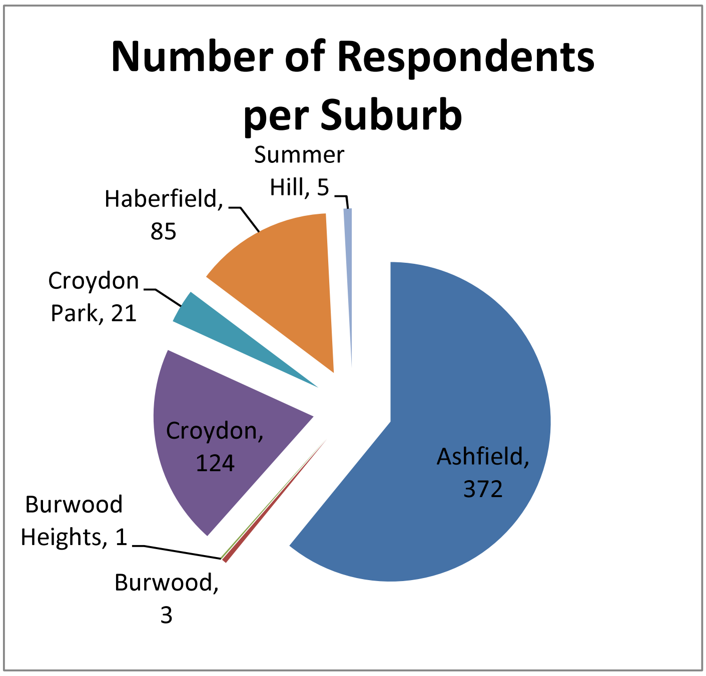
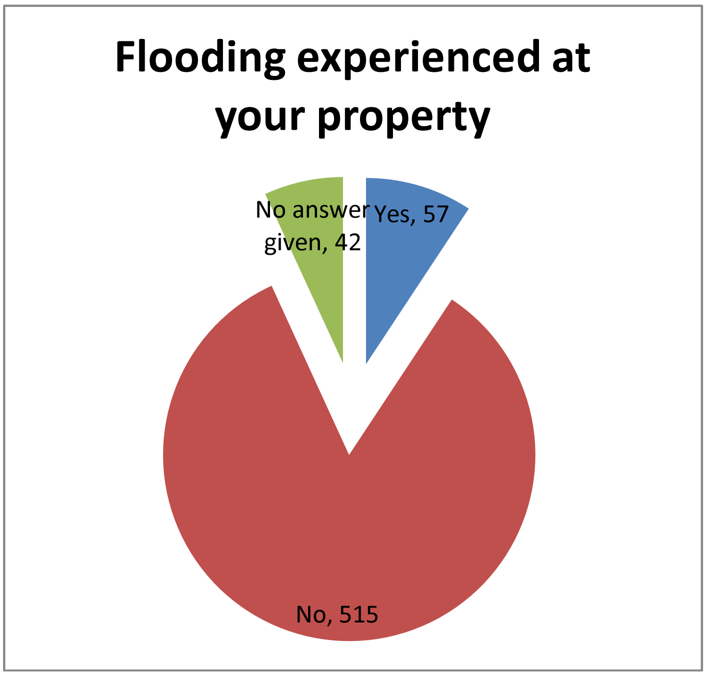
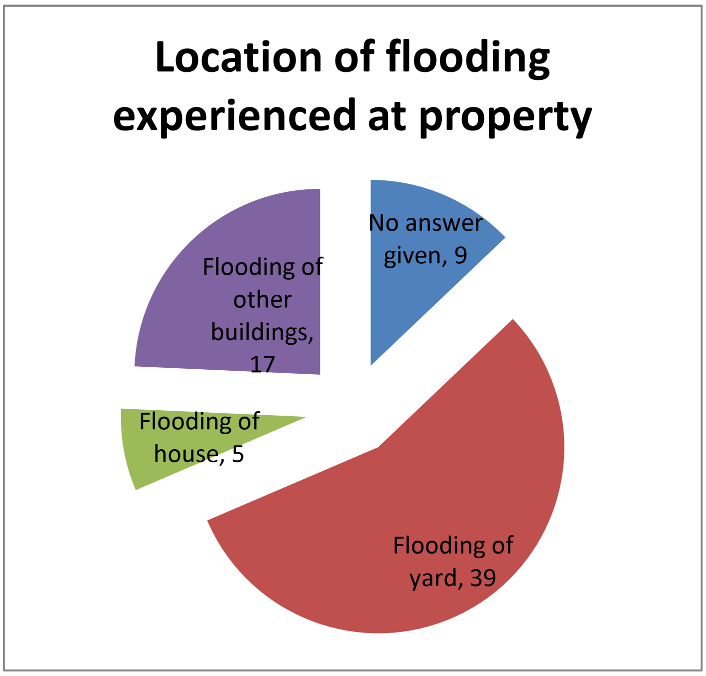
FIGURE 4A
COMMUNITY CONSULTATION STATISTICAL ANAYLSIS









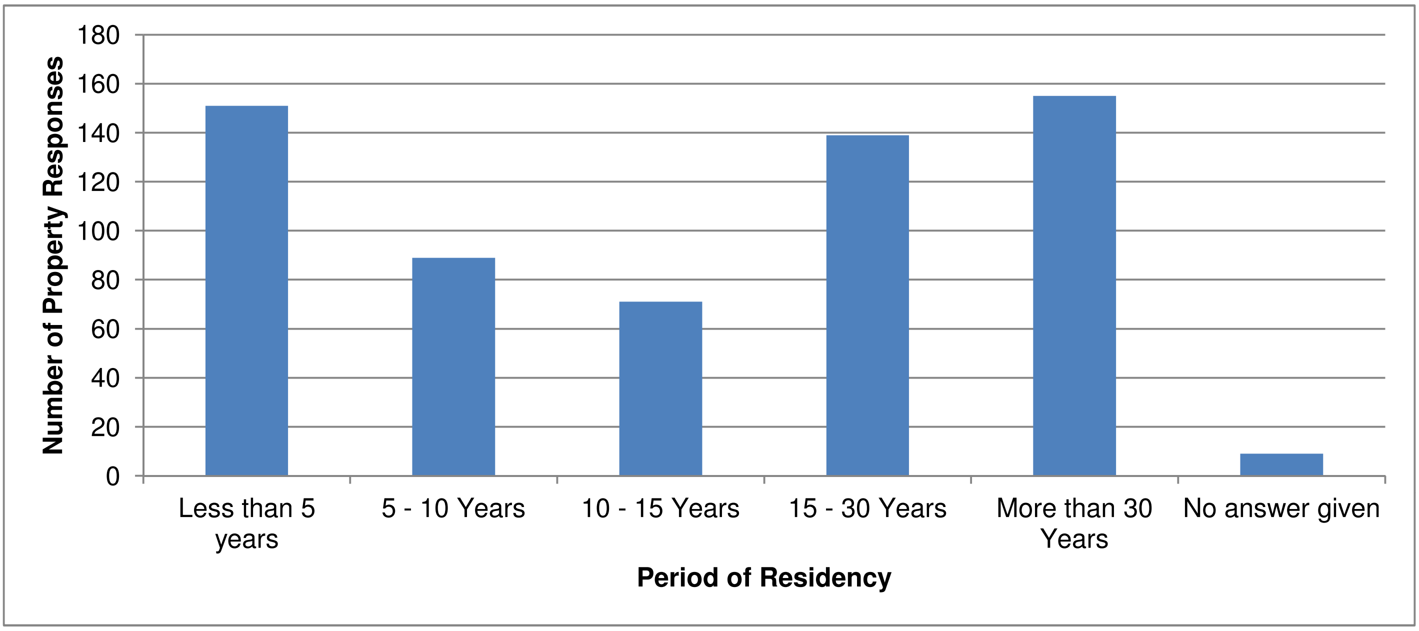


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FIGURE 4B
COMMUNITY CONSULTATION
PHOTOGRAPHS OF FLOODING PROVIDED

FIGURE 5A
HISTORIC FLOOD LEVEL LOCATIONS

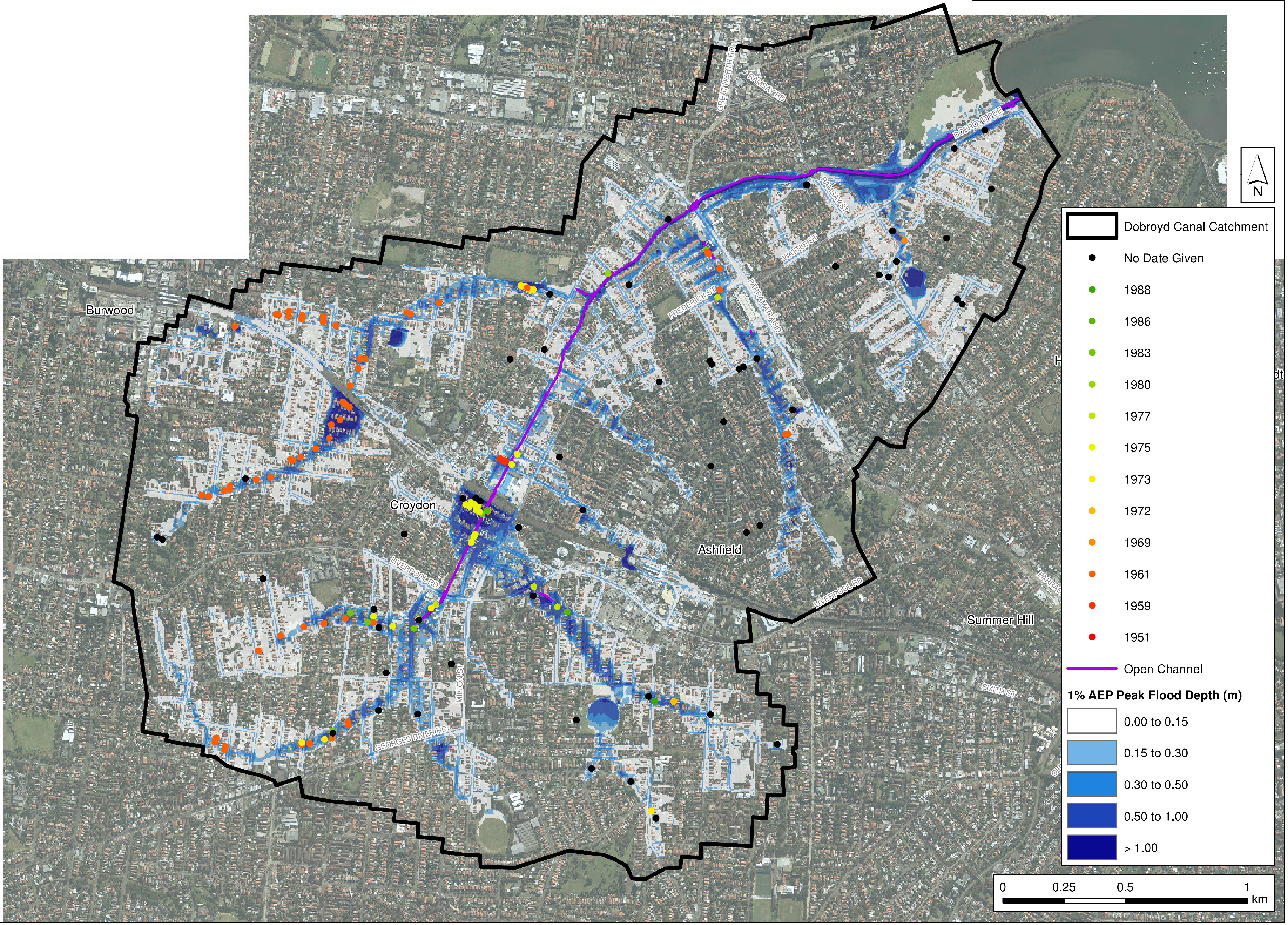

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FIGURE 5B
ROBINSON GRC CONSULTING (2002)
HOTSPOT LOCATIONS

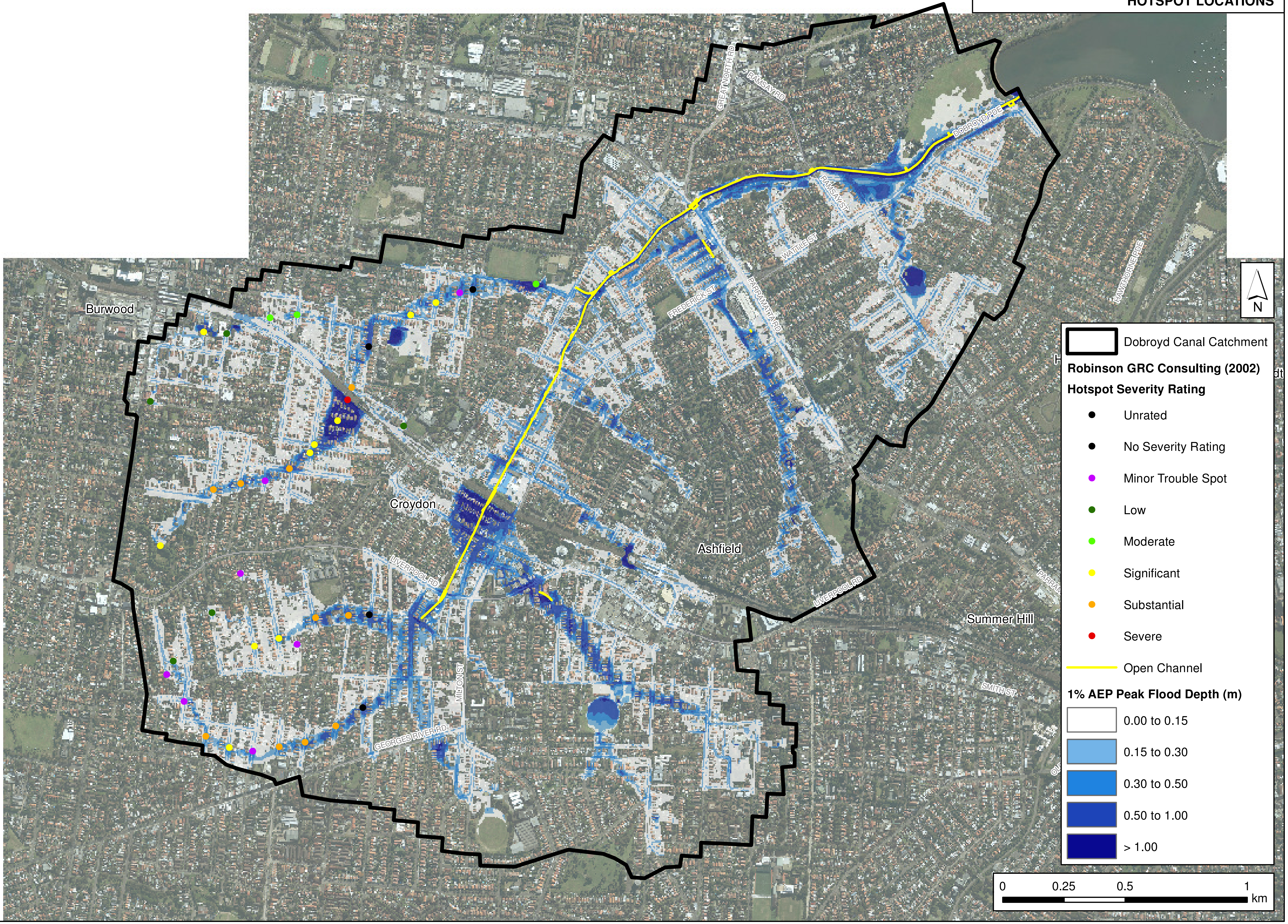

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FIGURE 6
GAUGE LOCATIONS

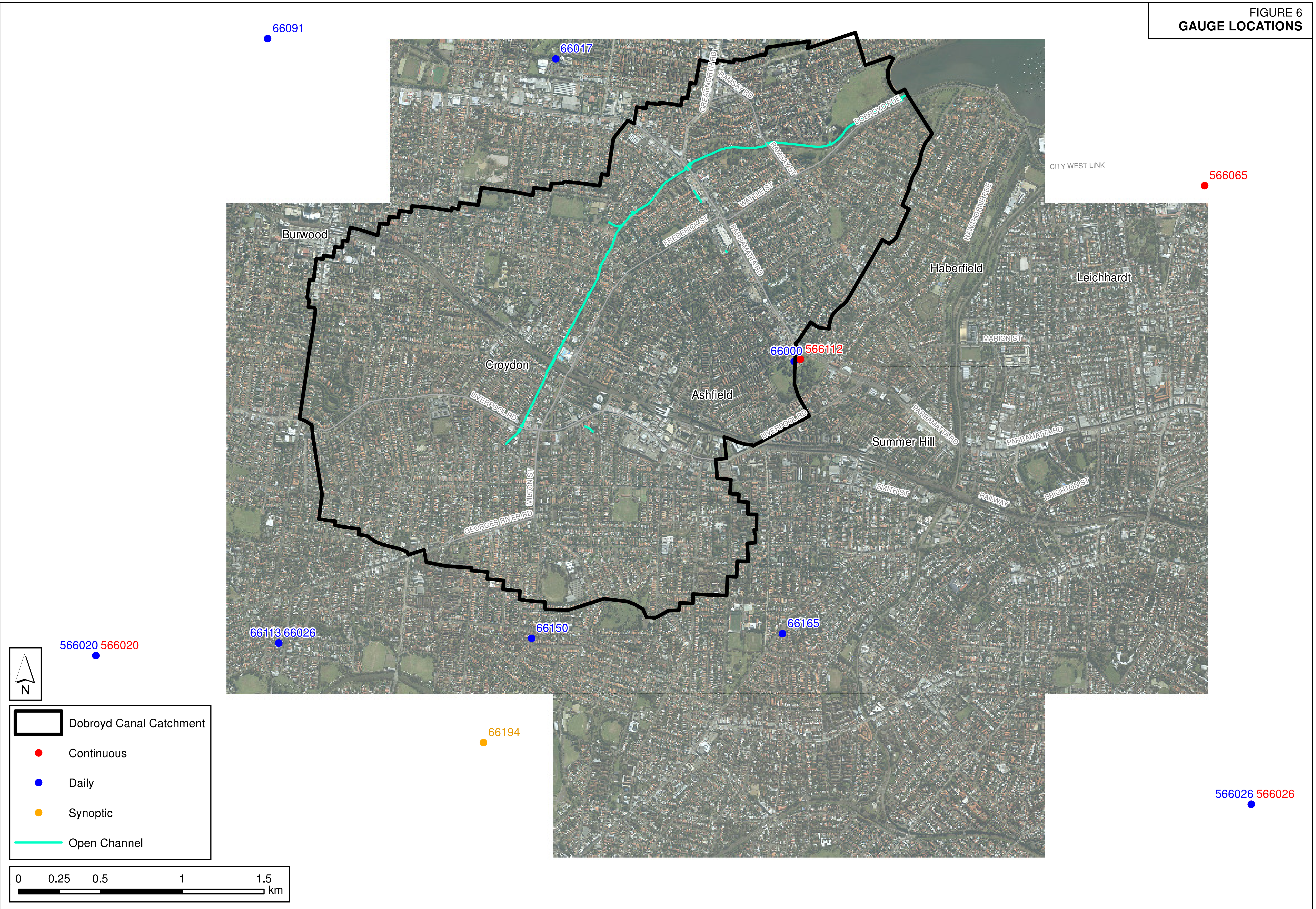

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FIGURE 7
HYDROLOGIC MODEL SCHEMATISATION

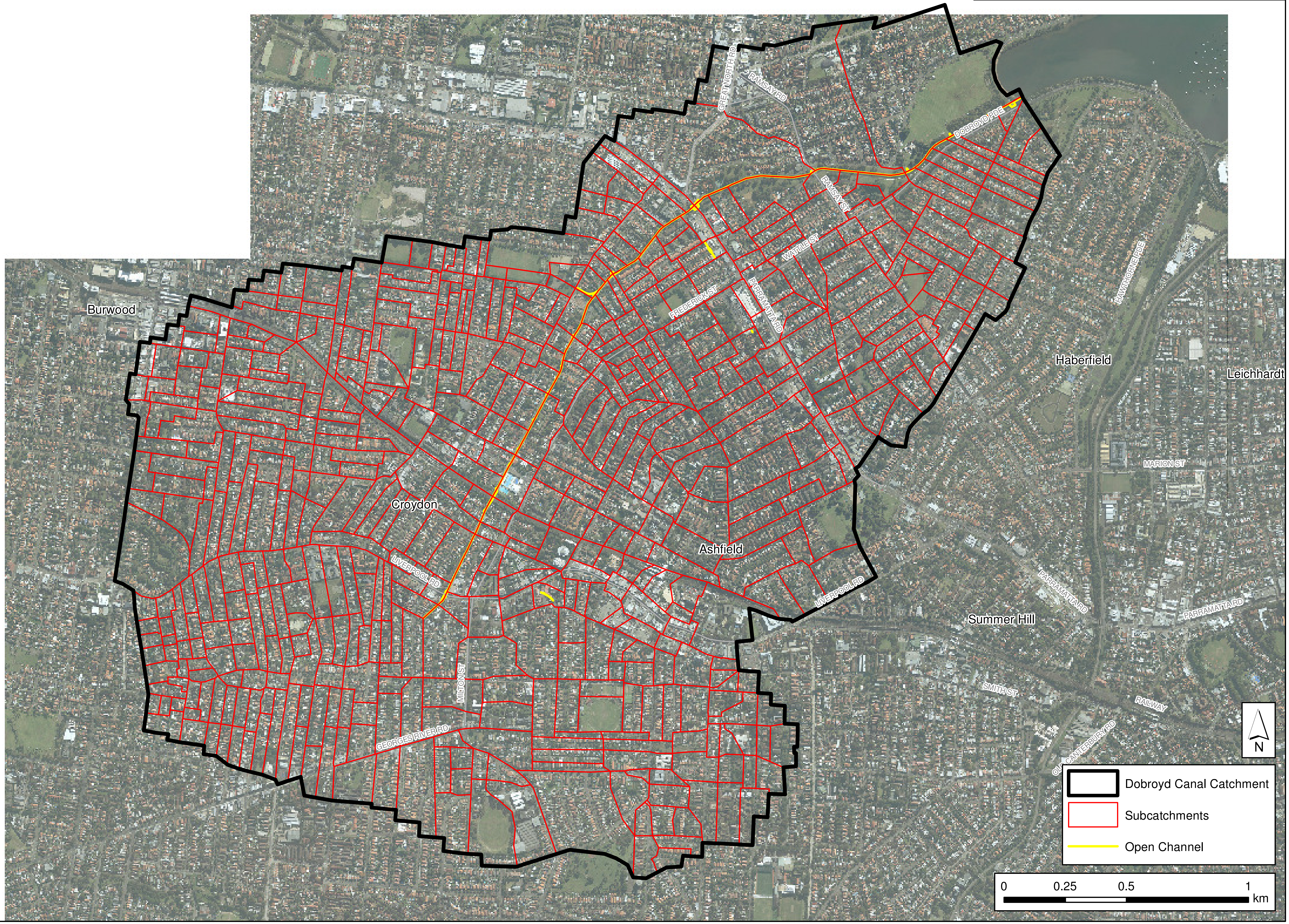

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FIGURE 8
HYDRAULIC MODEL SCHEMATISATION

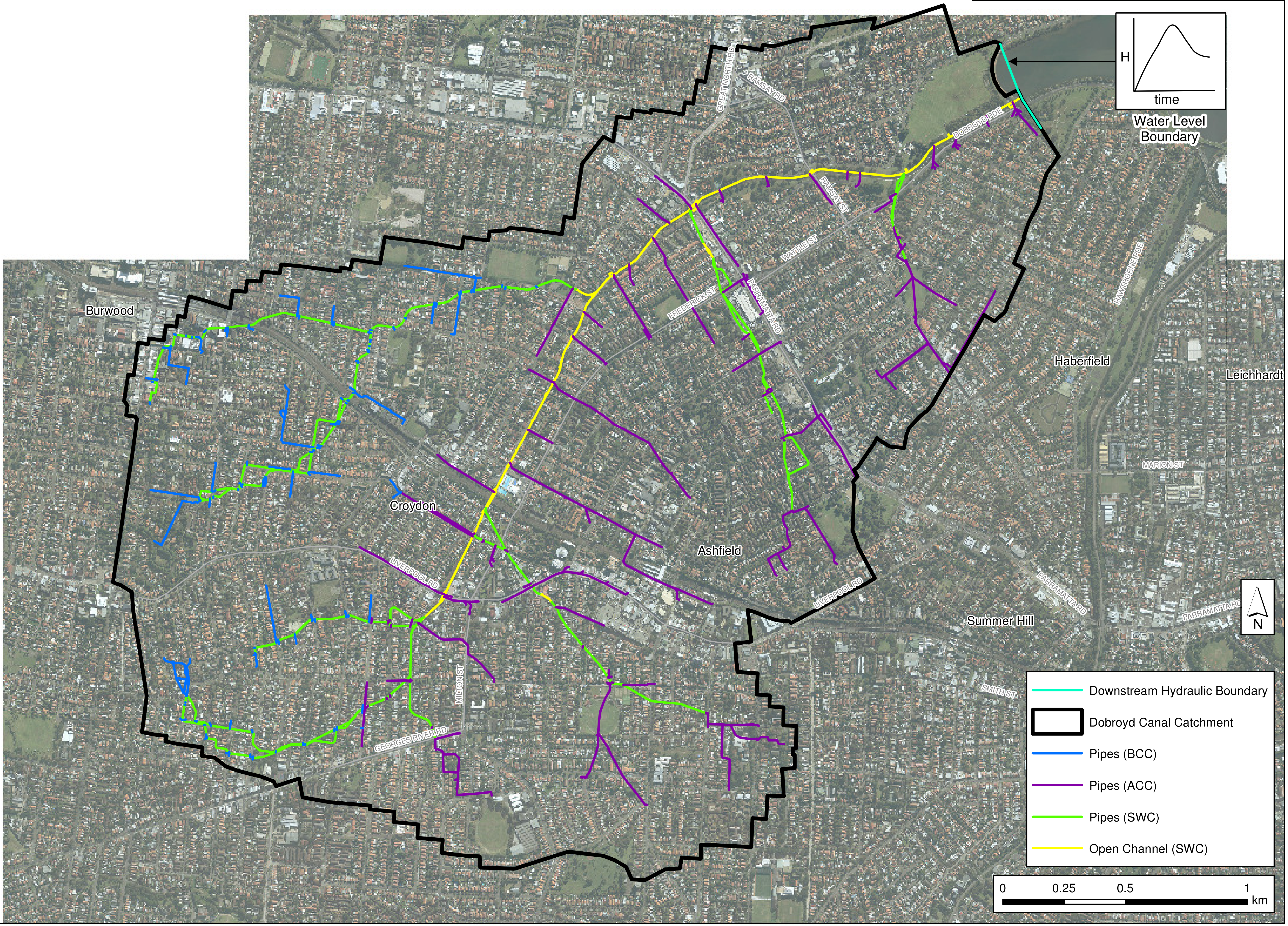

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FIGURE 9
HYDRAULIC MODEL ROUGHNESS VALUES

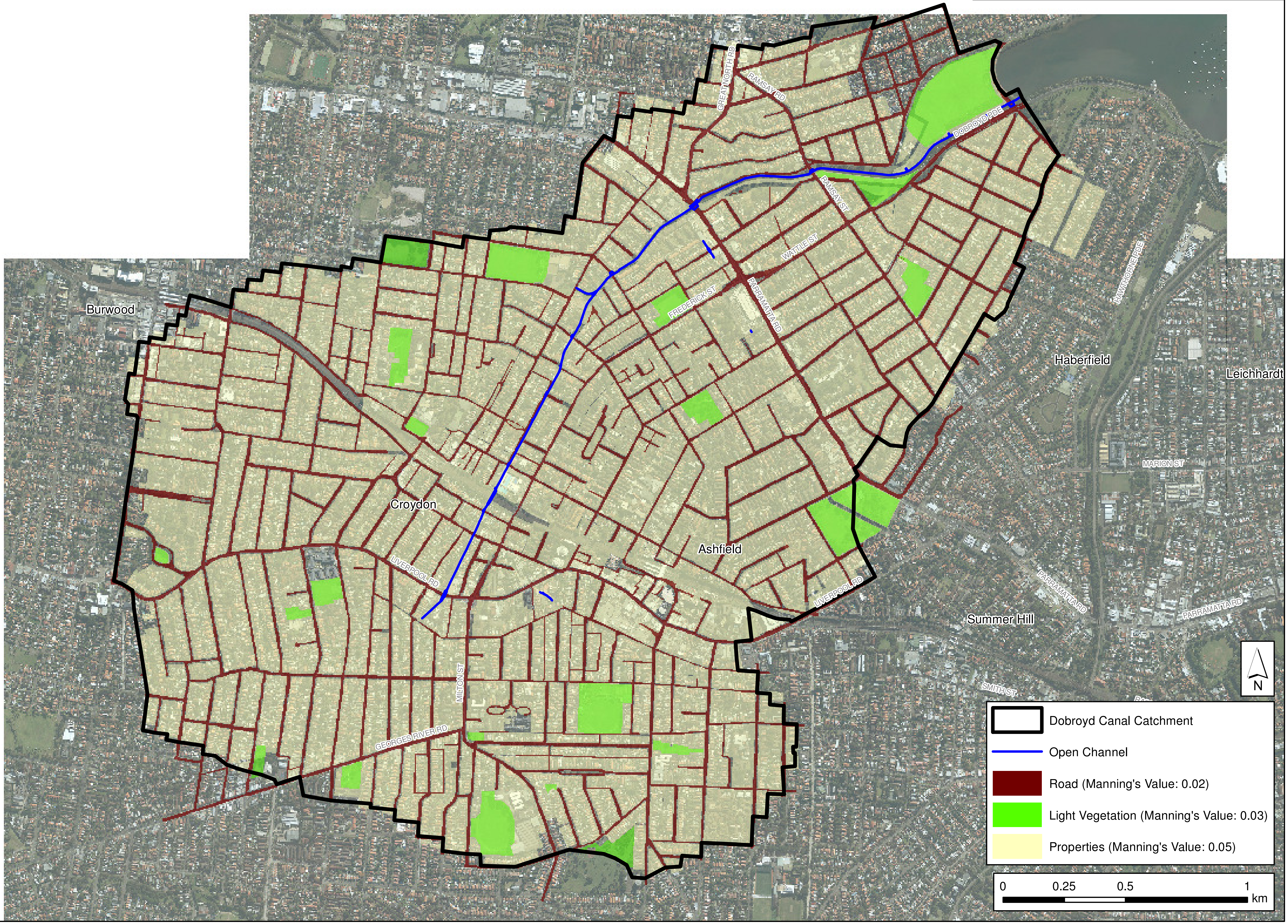

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FIGURE 10
RESULTS LAYOUT

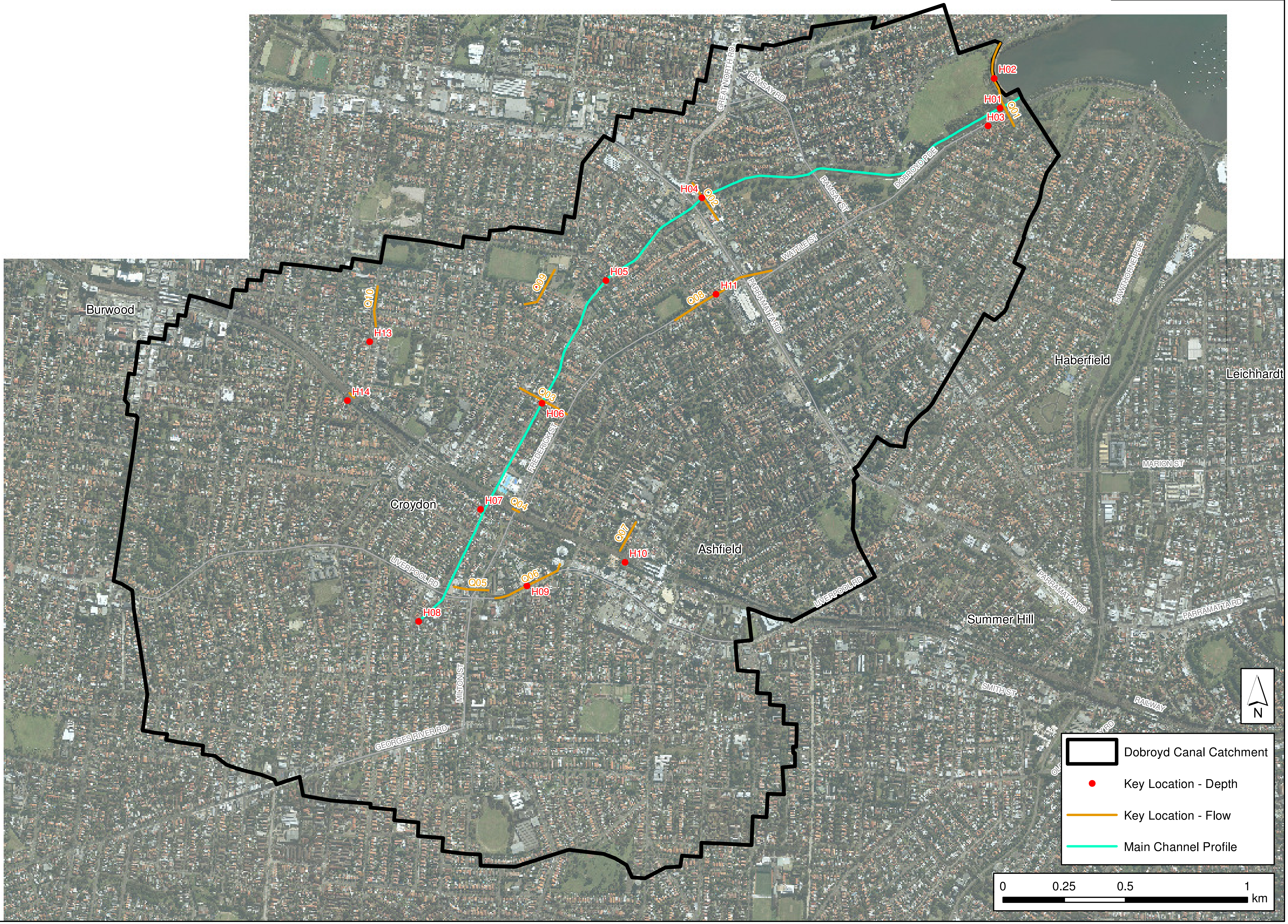

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FIGURE 11A
PEAK FLOOD LEVEL PROFILES
DESIGN EVENTS

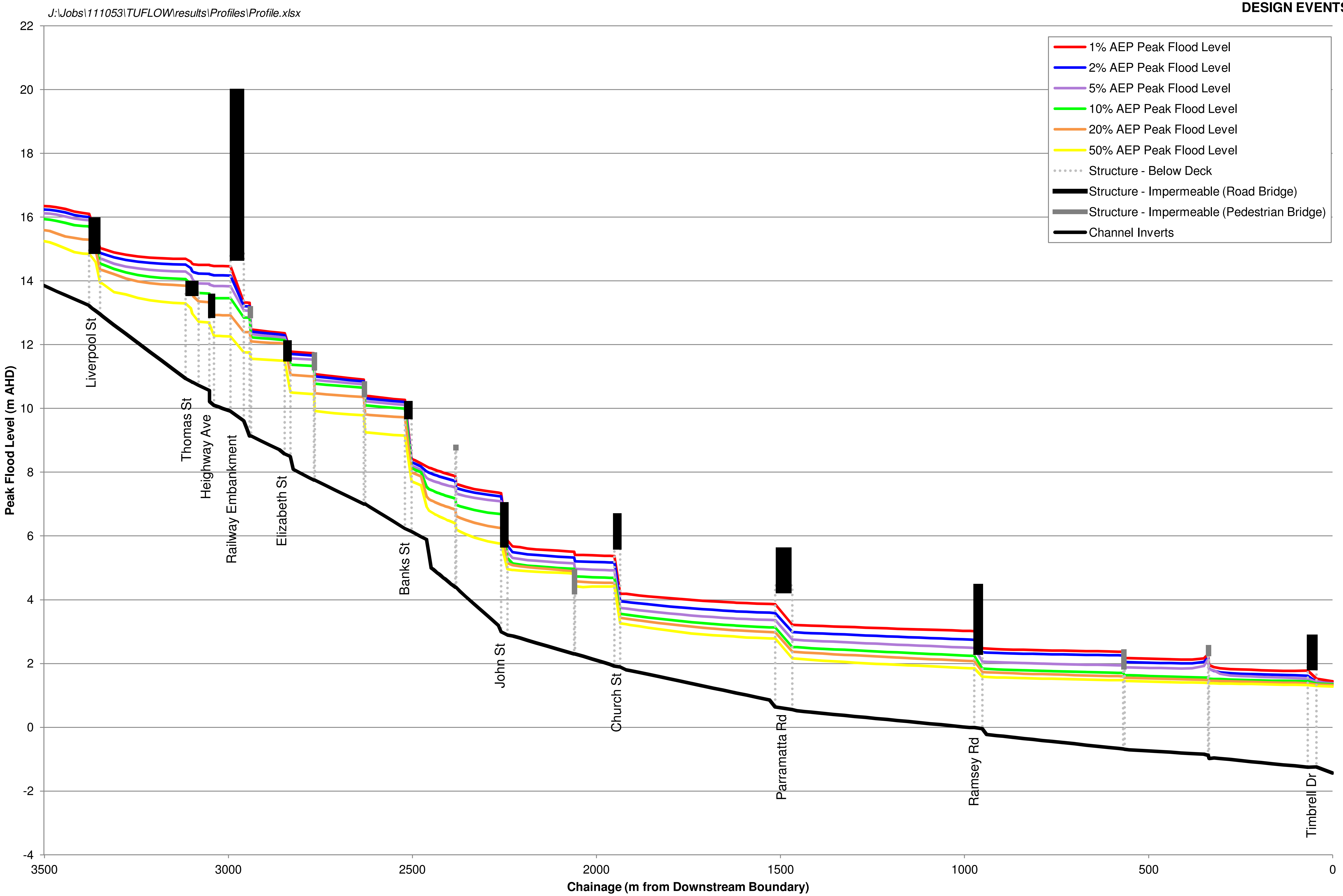
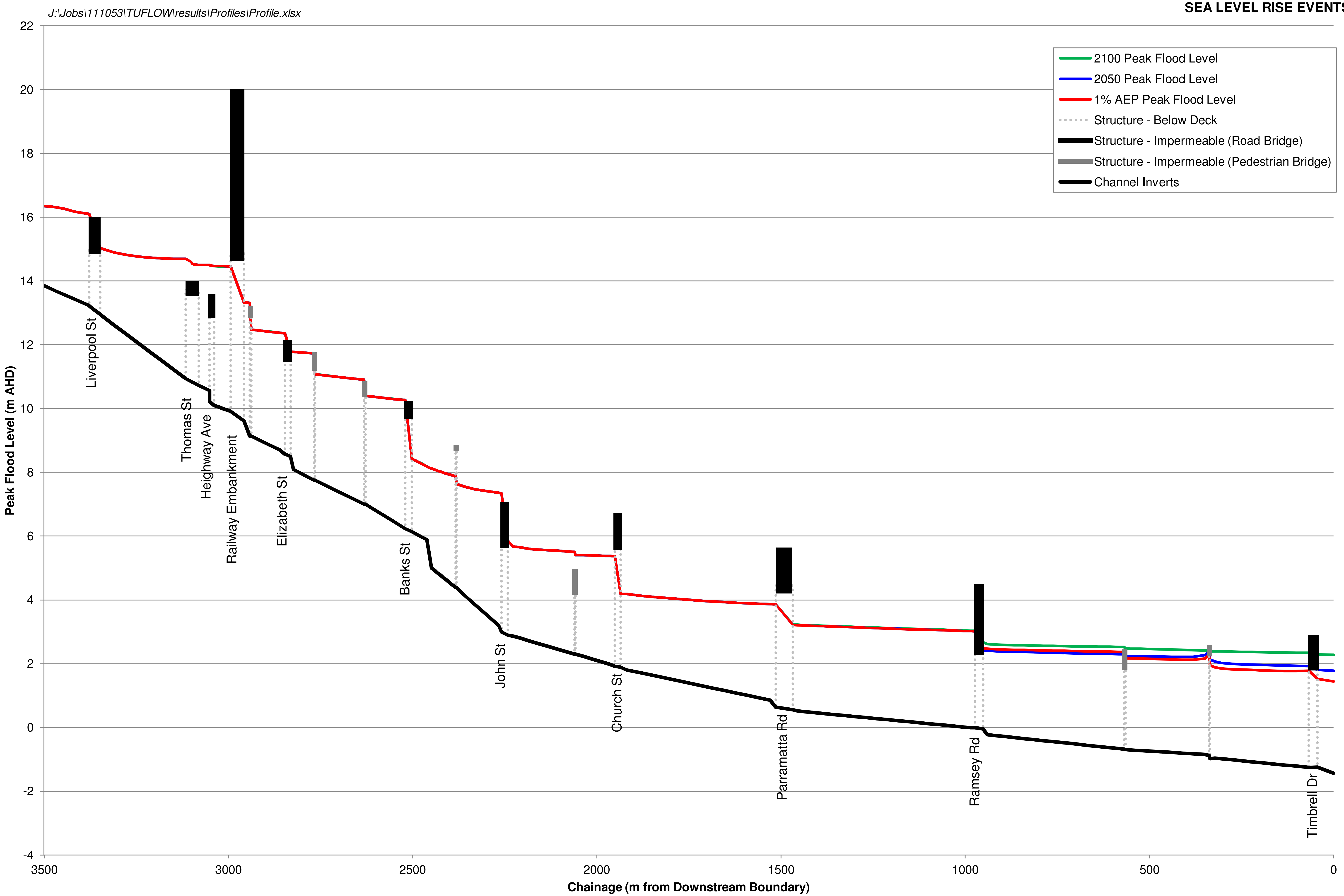


FIGURE 11B
PEAK FLOOD LEVEL PROFILES
SEA LEVEL RISE EVENTS



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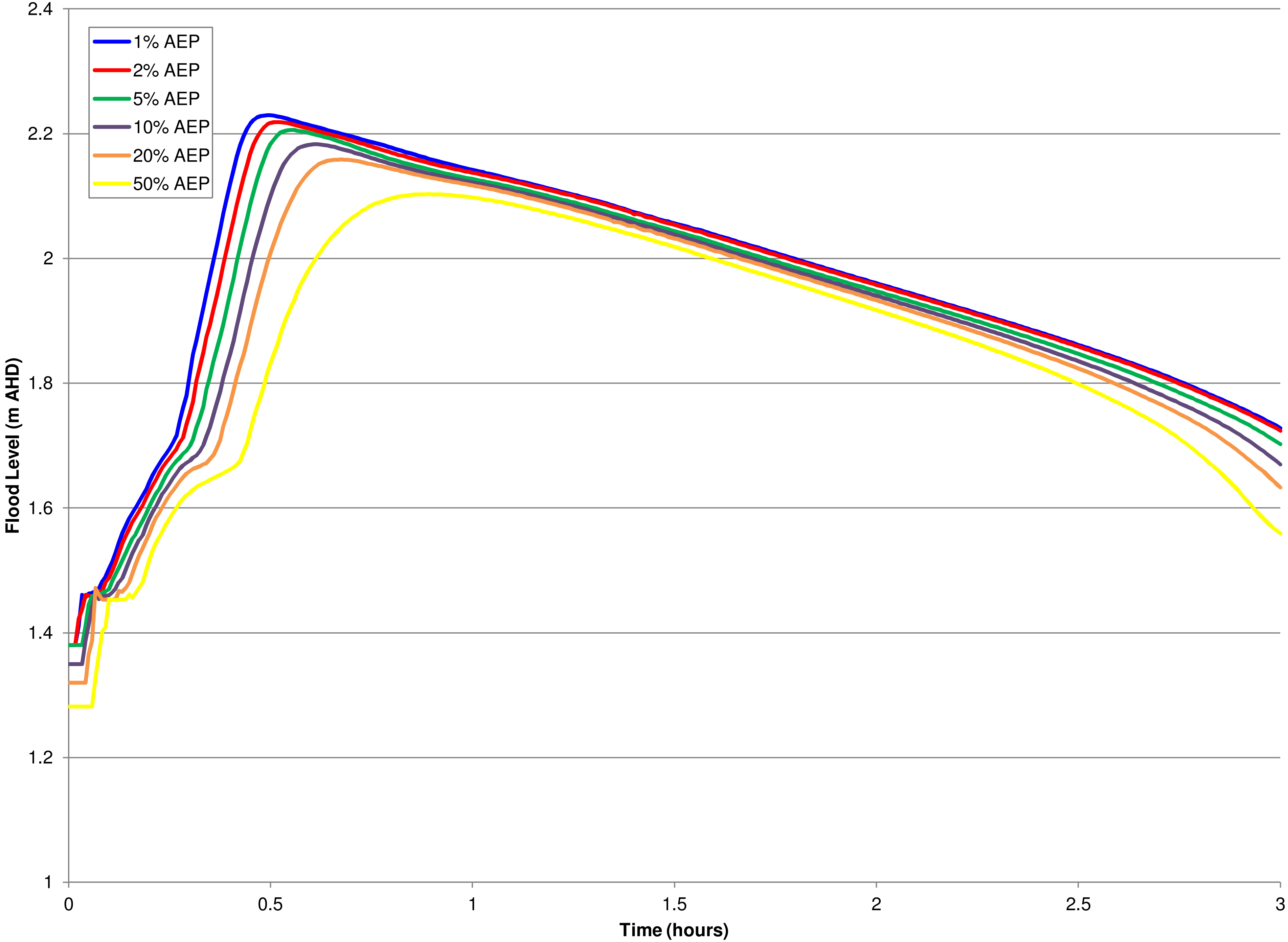

FIGURE 12A
DESIGN FLOOD LEVEL HYDROGRAPH
DOBROYD PARADE

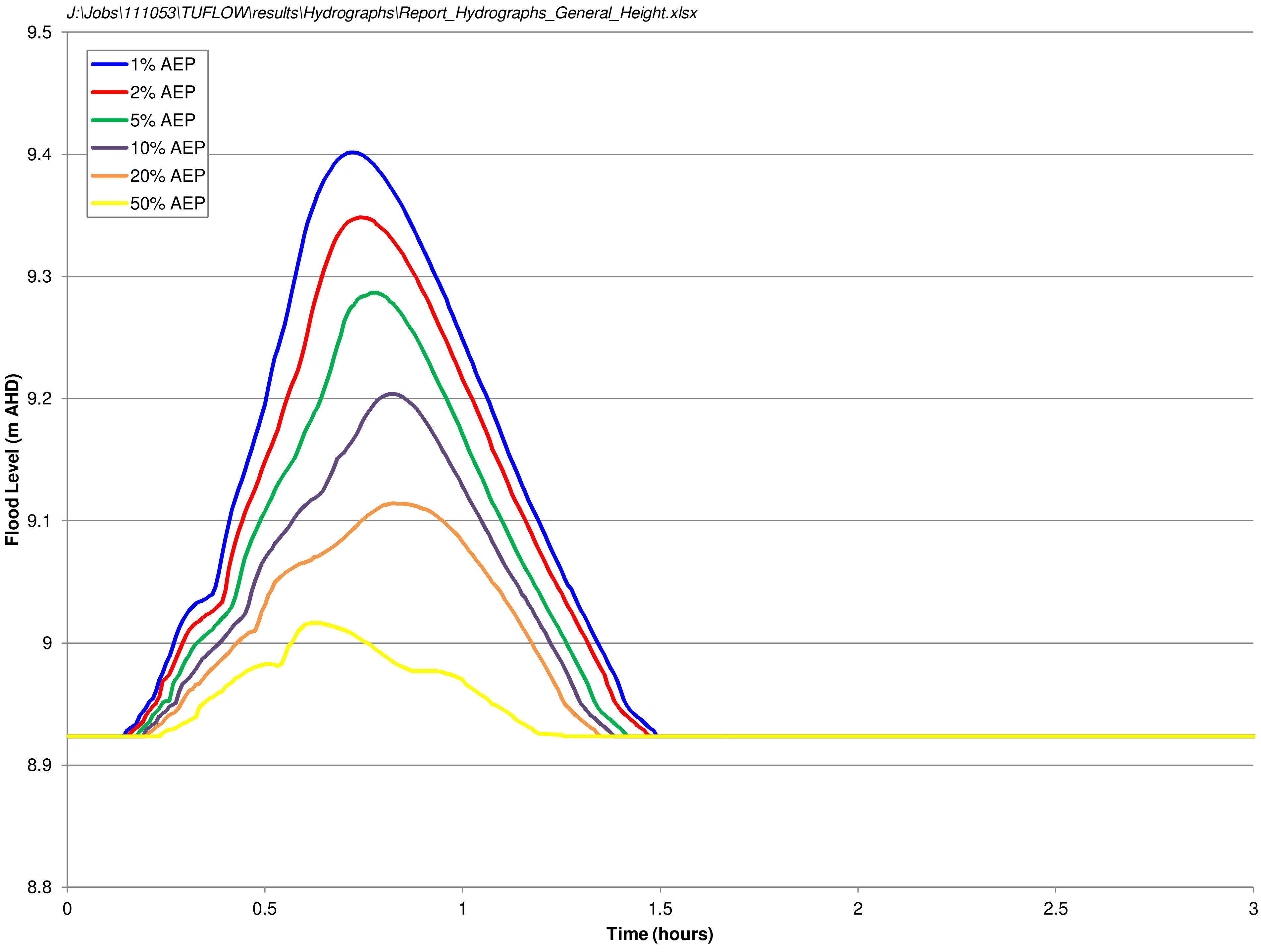

FIGURE 12B
DESIGN FLOOD LEVEL HYDROGRAPH
FREDERICK STREET

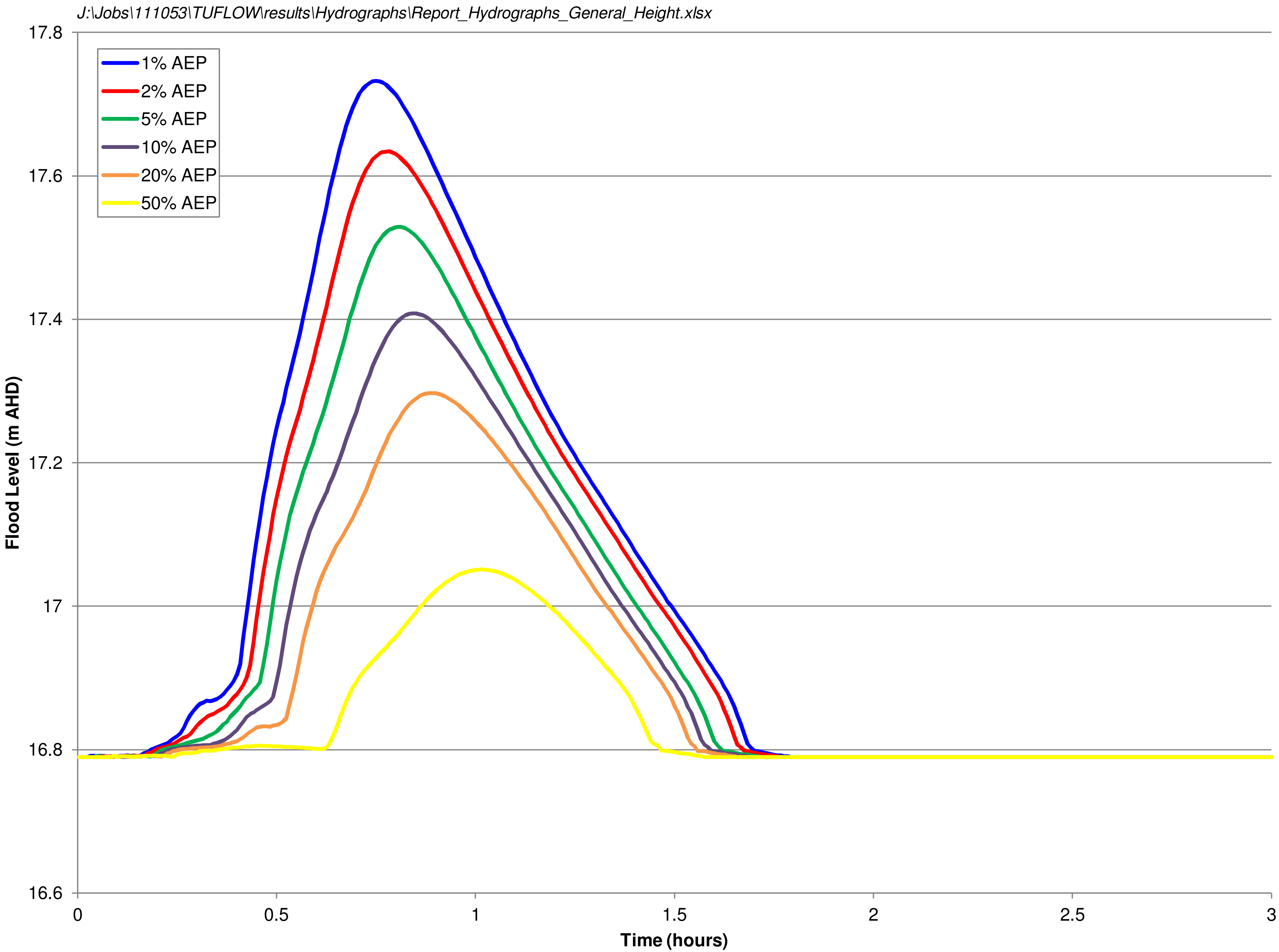

FIGURE 12C
DESIGN FLOOD LEVEL HYDROGRAPH
HUME HIGHWAY

FIGURE 13
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS 50% AEP

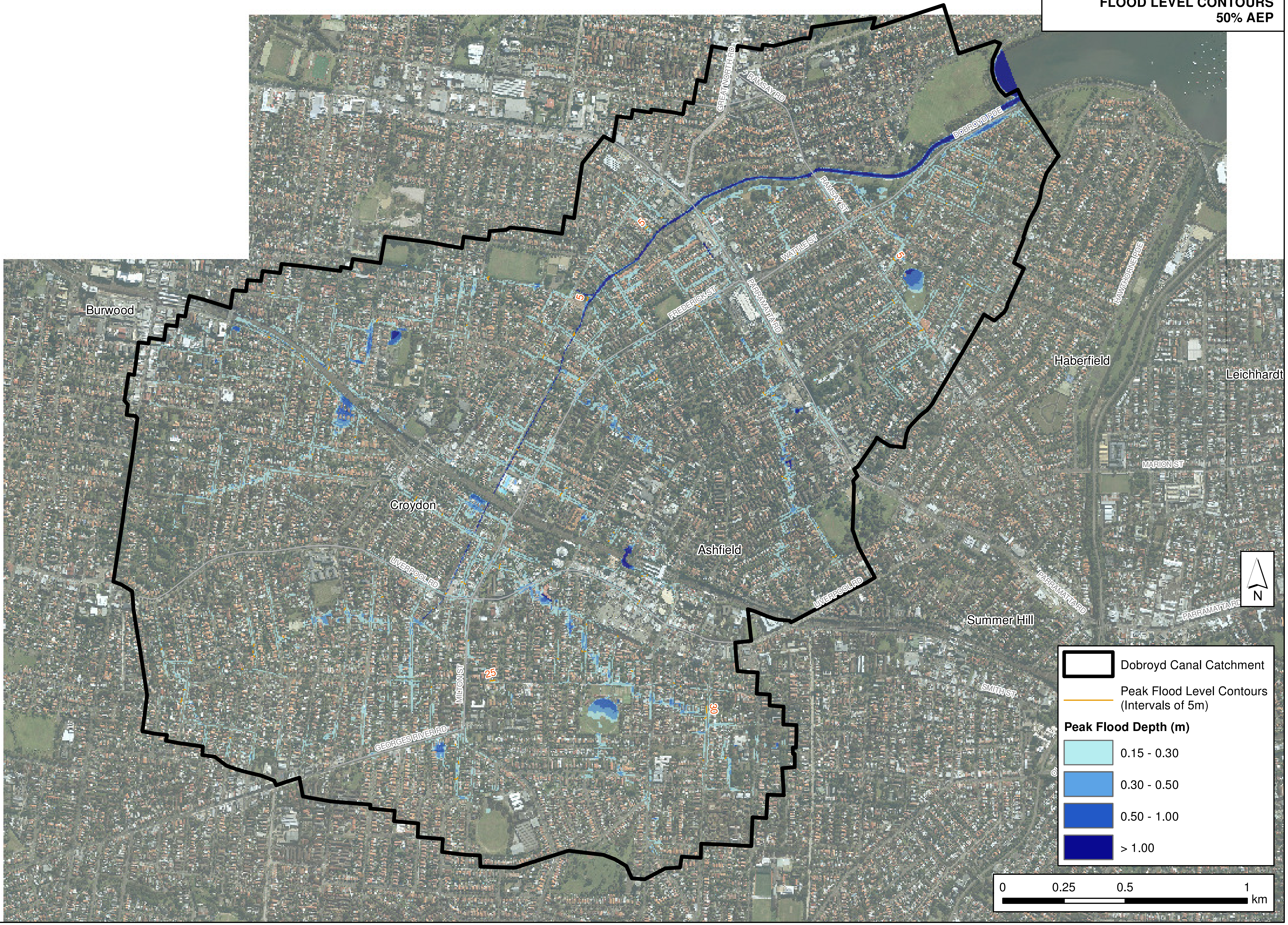

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FIGURE 14
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS 20% AEP

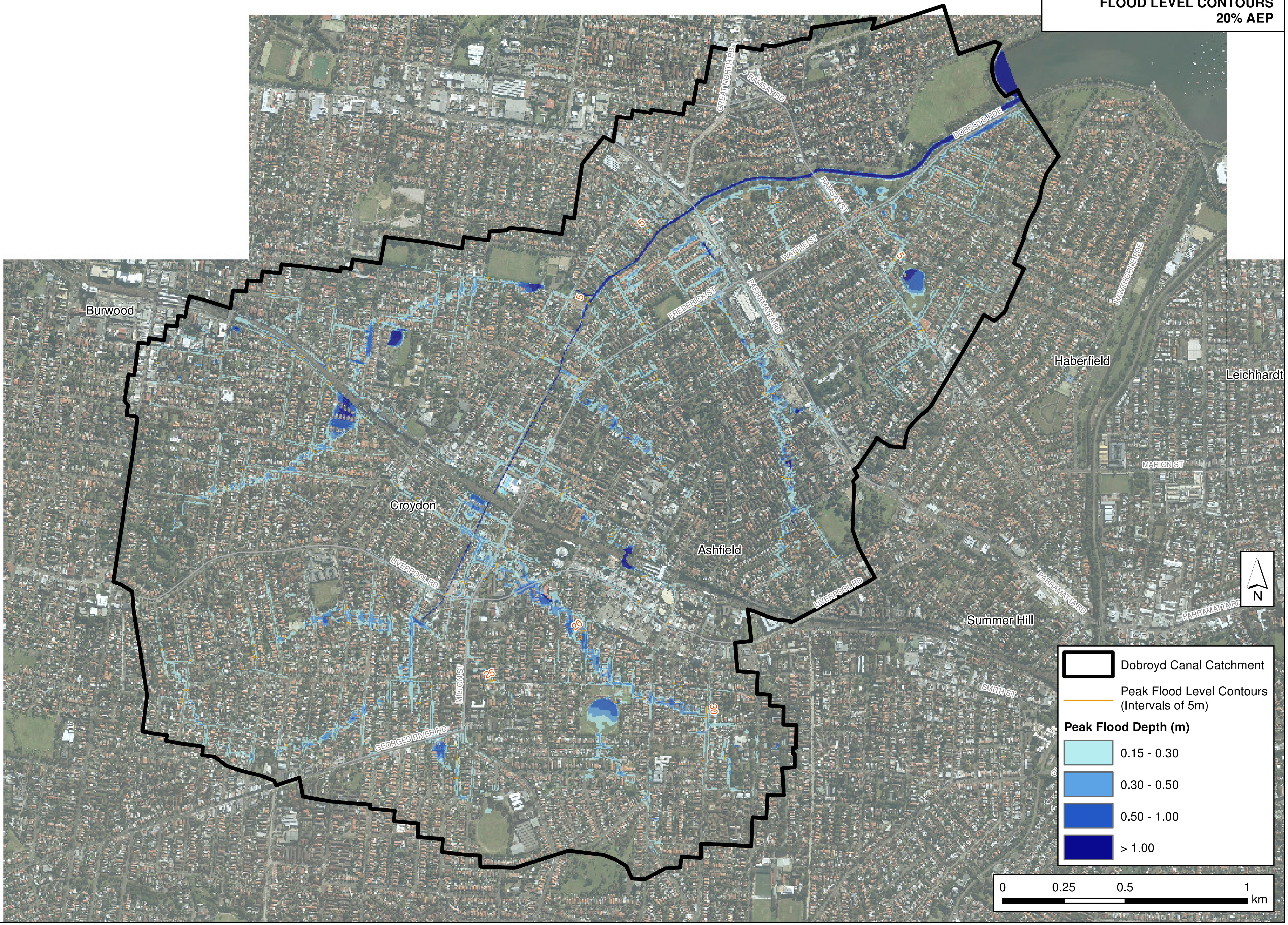

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FIGURE 15
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS
10% AEP

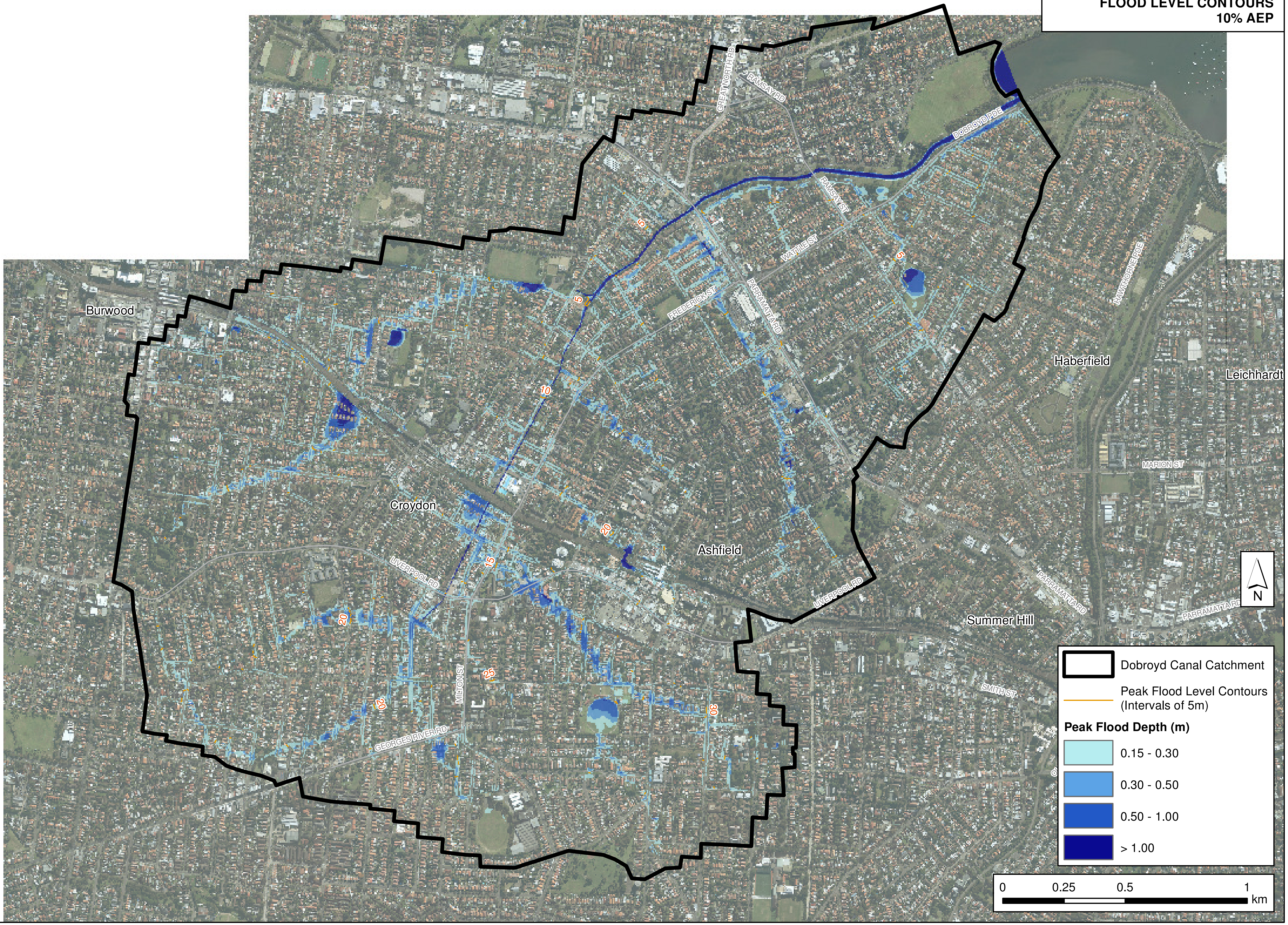

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FIGURE 16
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS 5% AEP

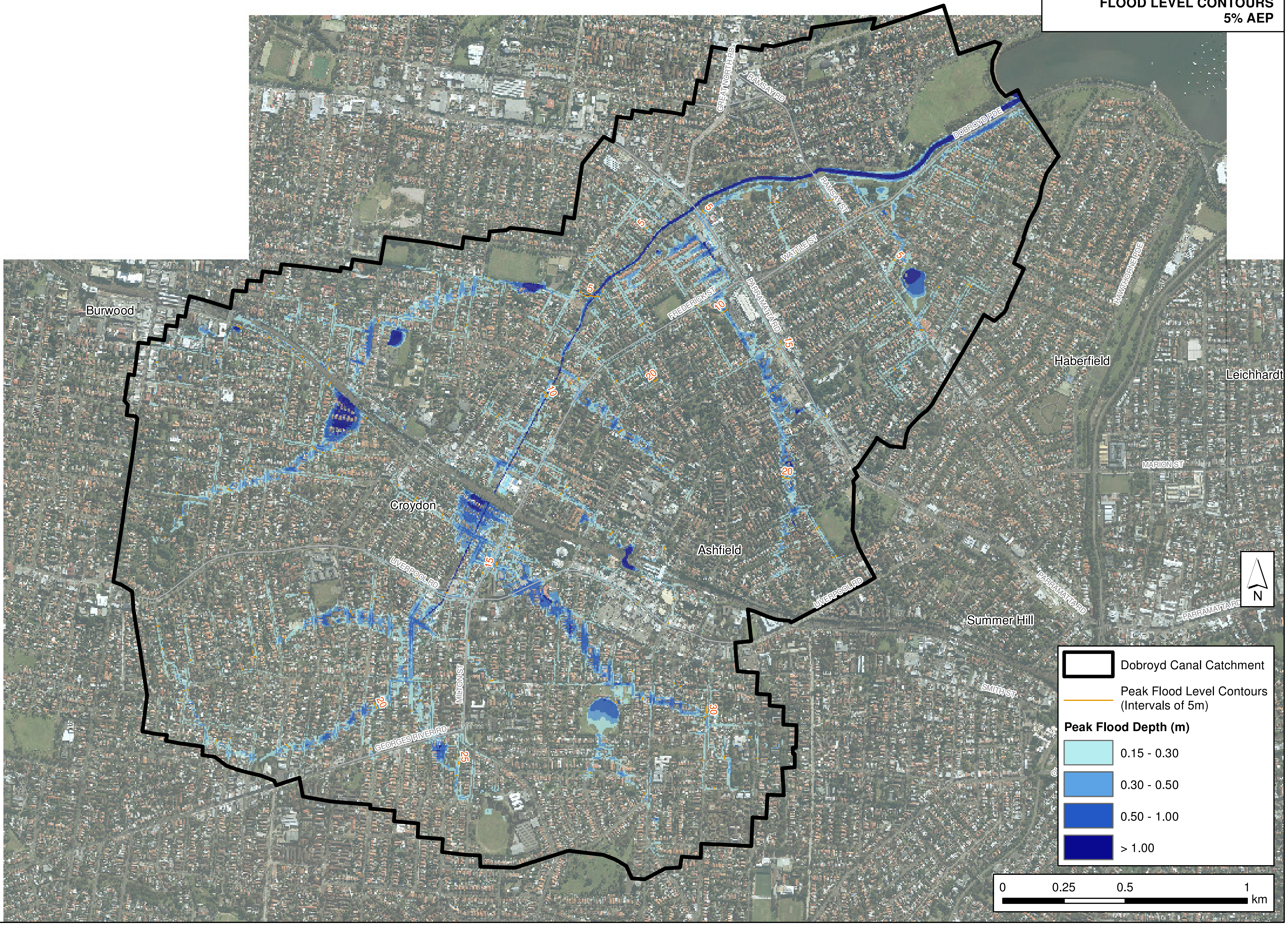

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FIGURE 17
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS 2% AEP

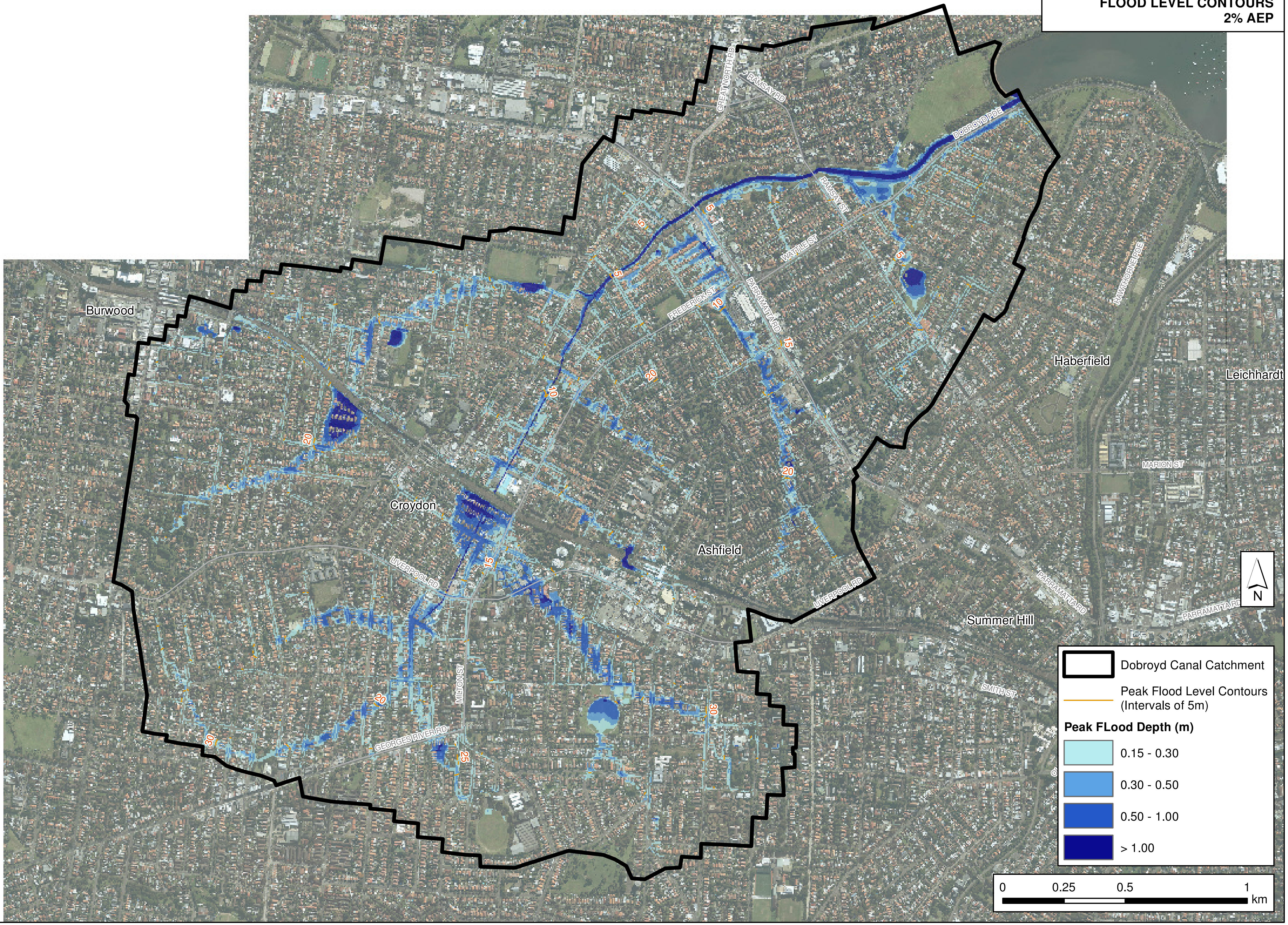

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FIGURE 18
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS 1% AEP

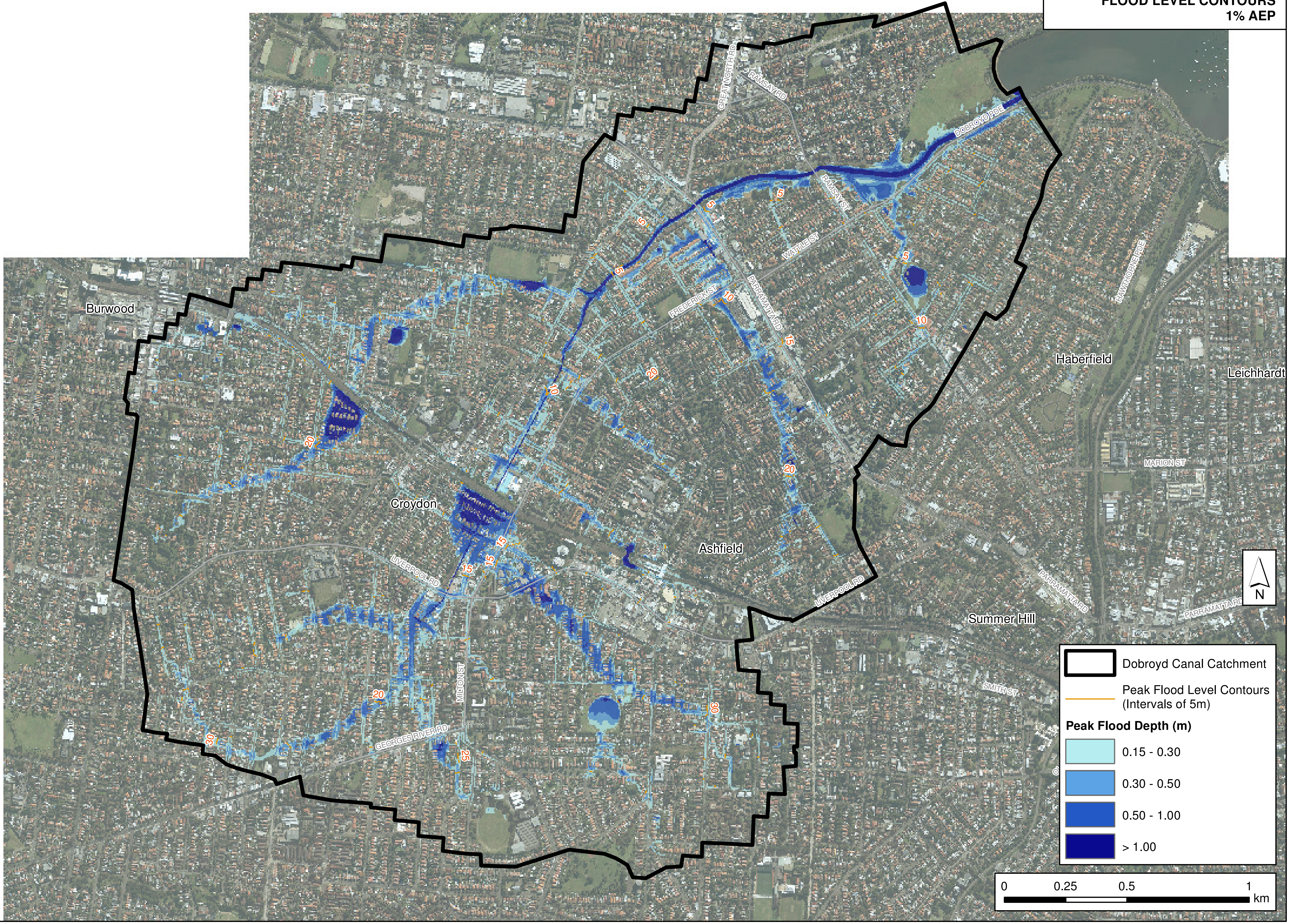

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FIGURE 19
PEAK FLOOD DEPTHS AND FLOOD LEVEL CONTOURS PMF

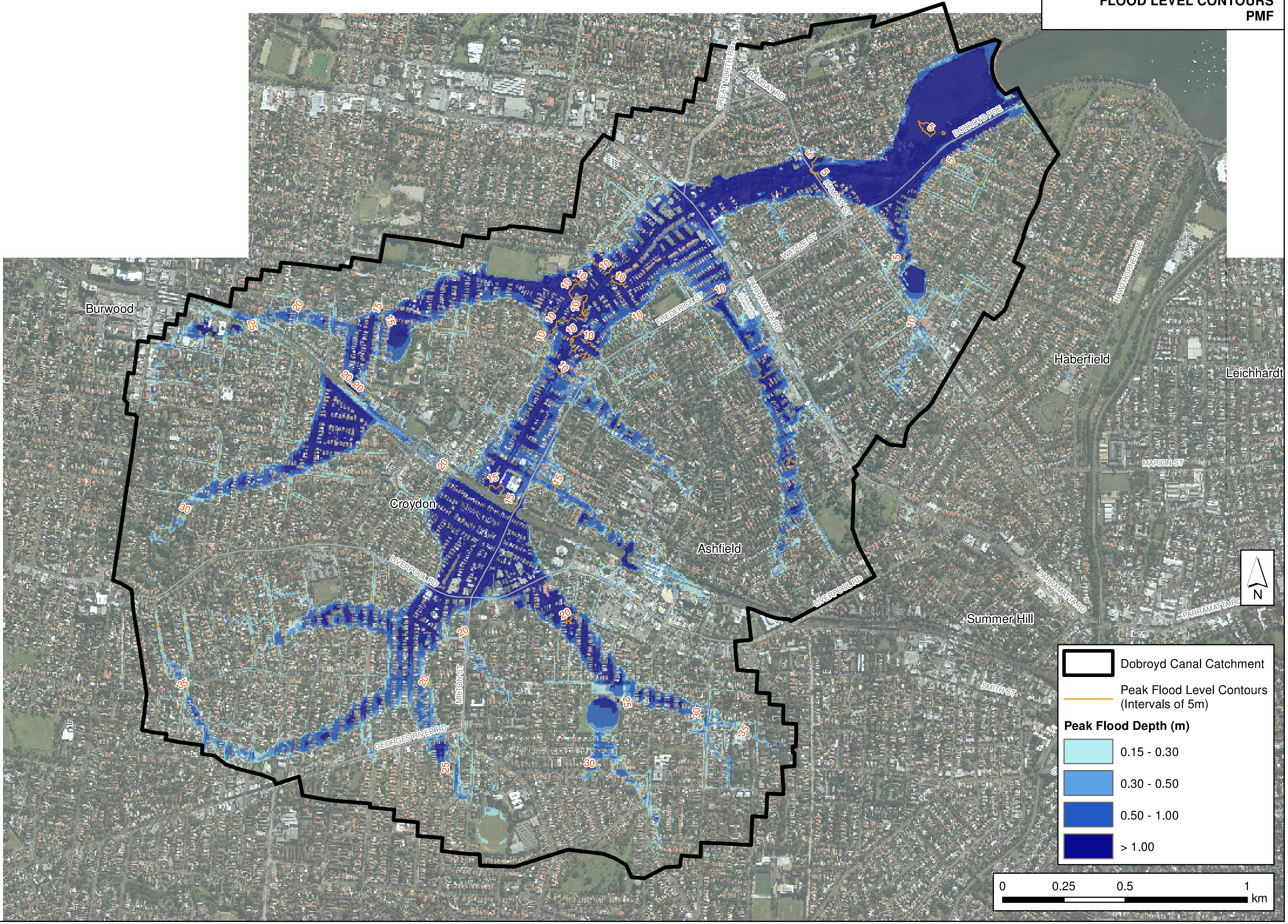

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FIGURE 20
PEAK FLOOD VELOCITY
1% AEP

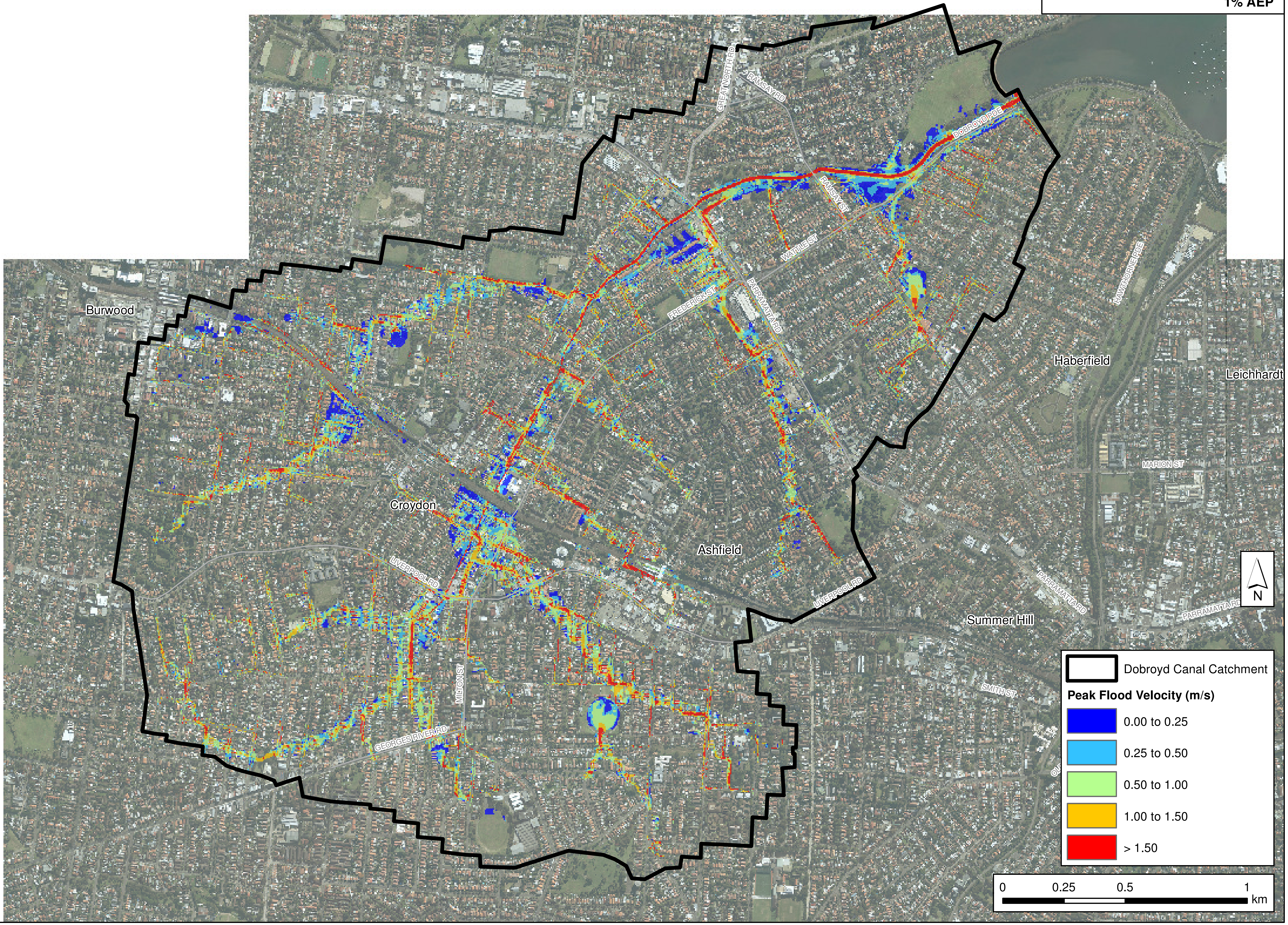

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FIGURE 21
PROVISIONAL HYDRAULIC HAZARD
20% AEP

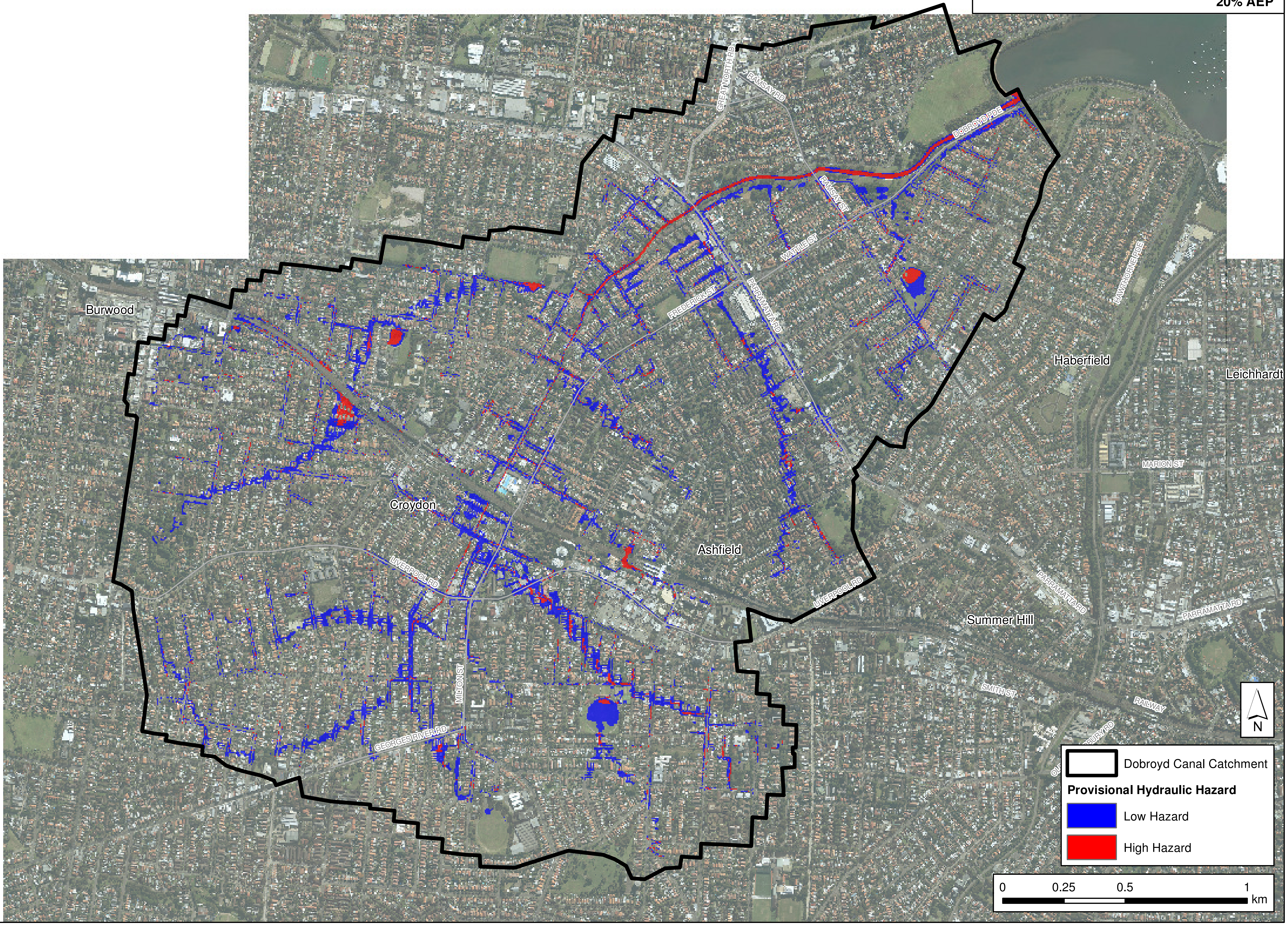

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FIGURE 22
PROVISIONAL HYDRAULIC HAZARD
5% AEP

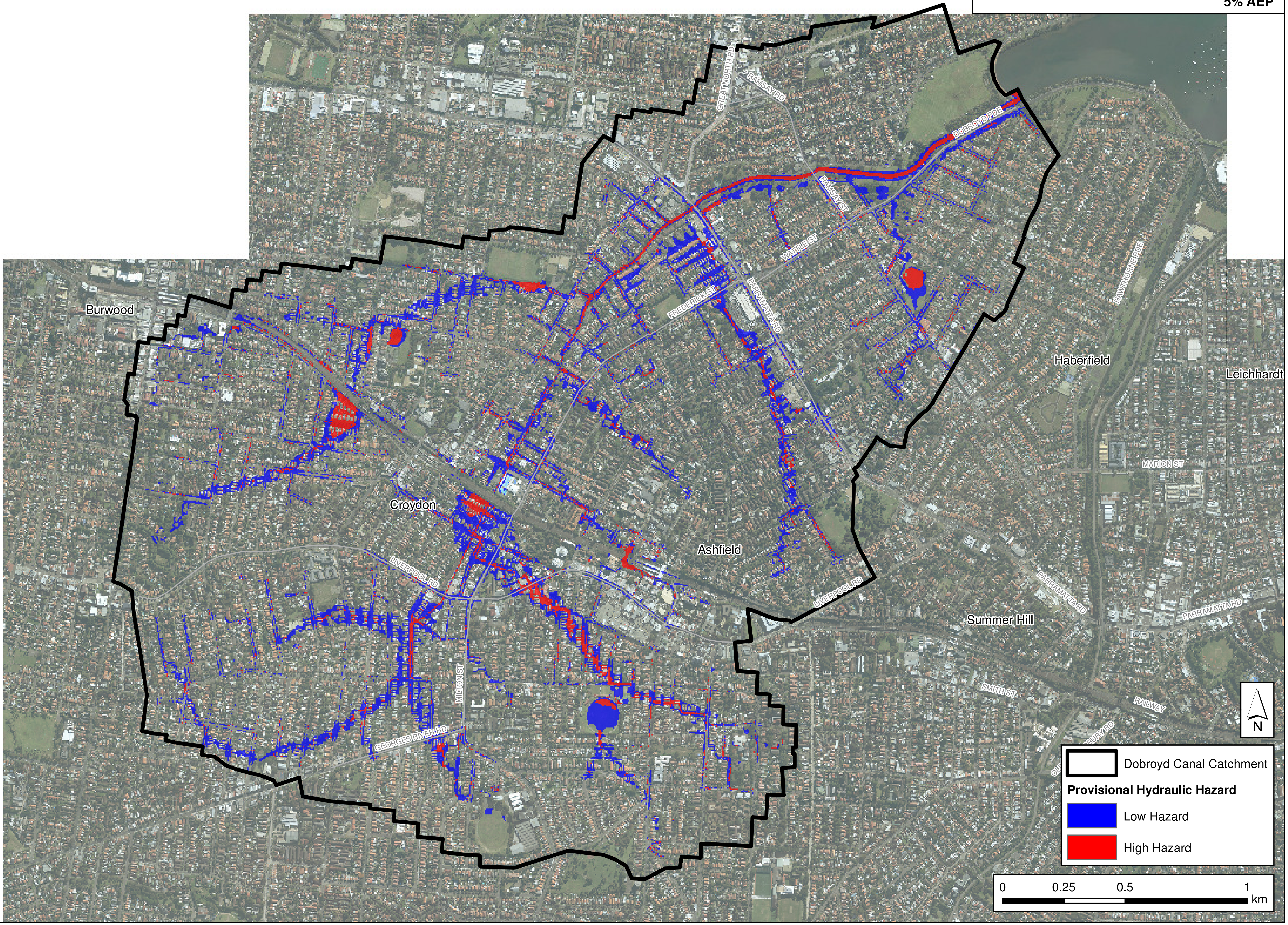

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FIGURE 23
PROVISIONAL HYDRAULIC HAZARD
1% AEP

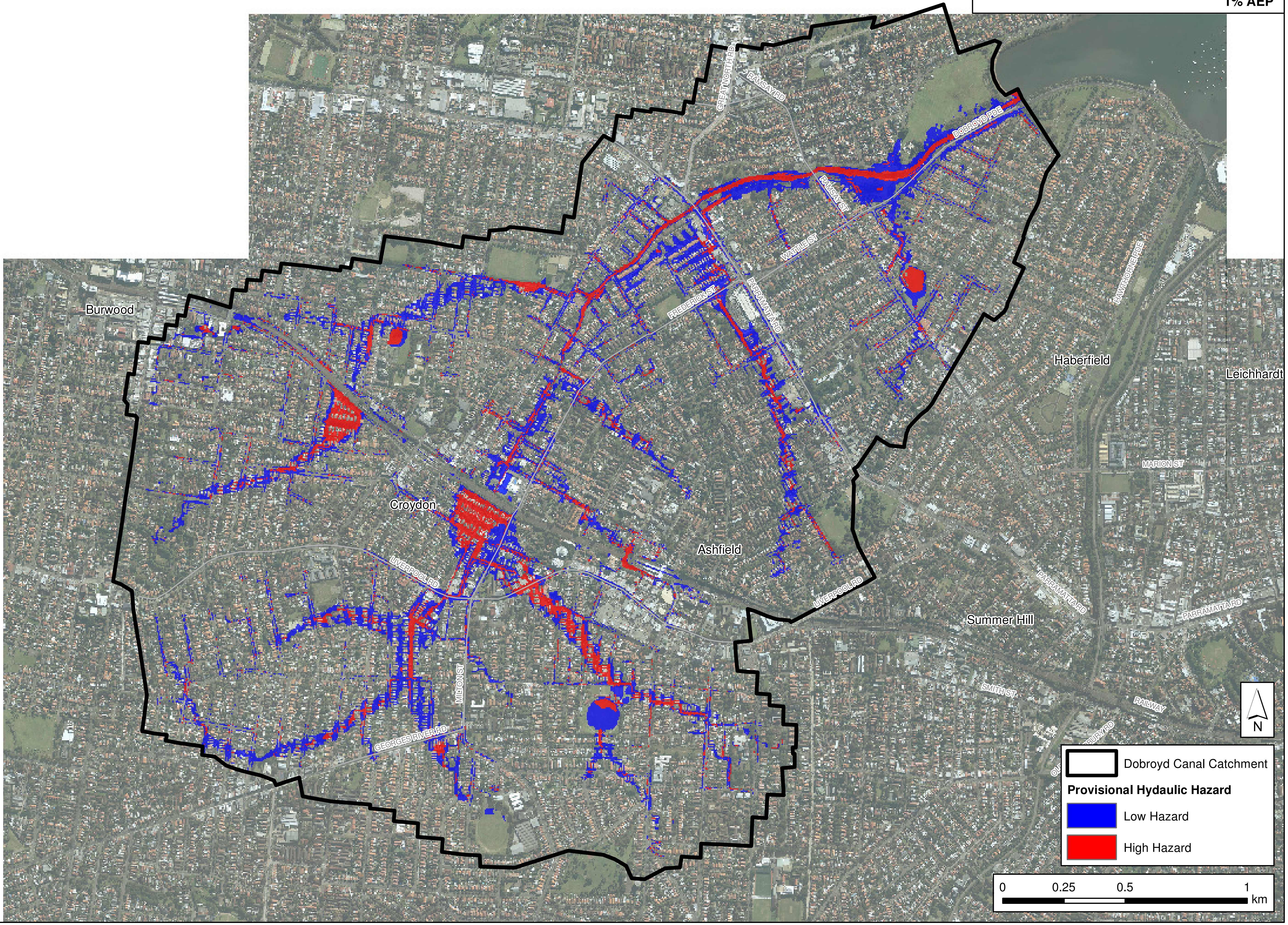

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FIGURE 24
PROVISIONAL HYDRAULIC HAZARD
PMF

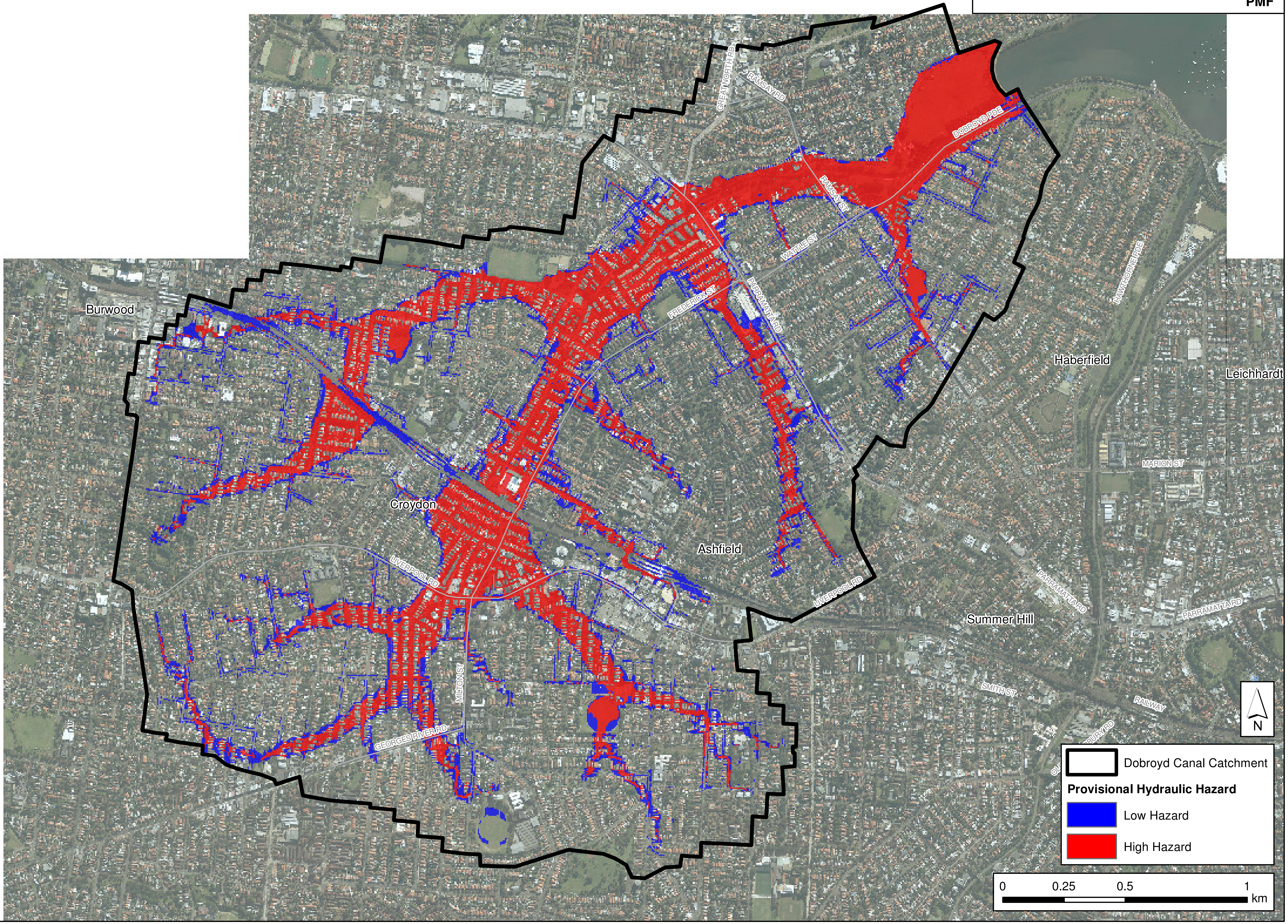

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FIGURE 25
PROVISIONAL HYDRAULIC CATEGORISATION
20% AEP

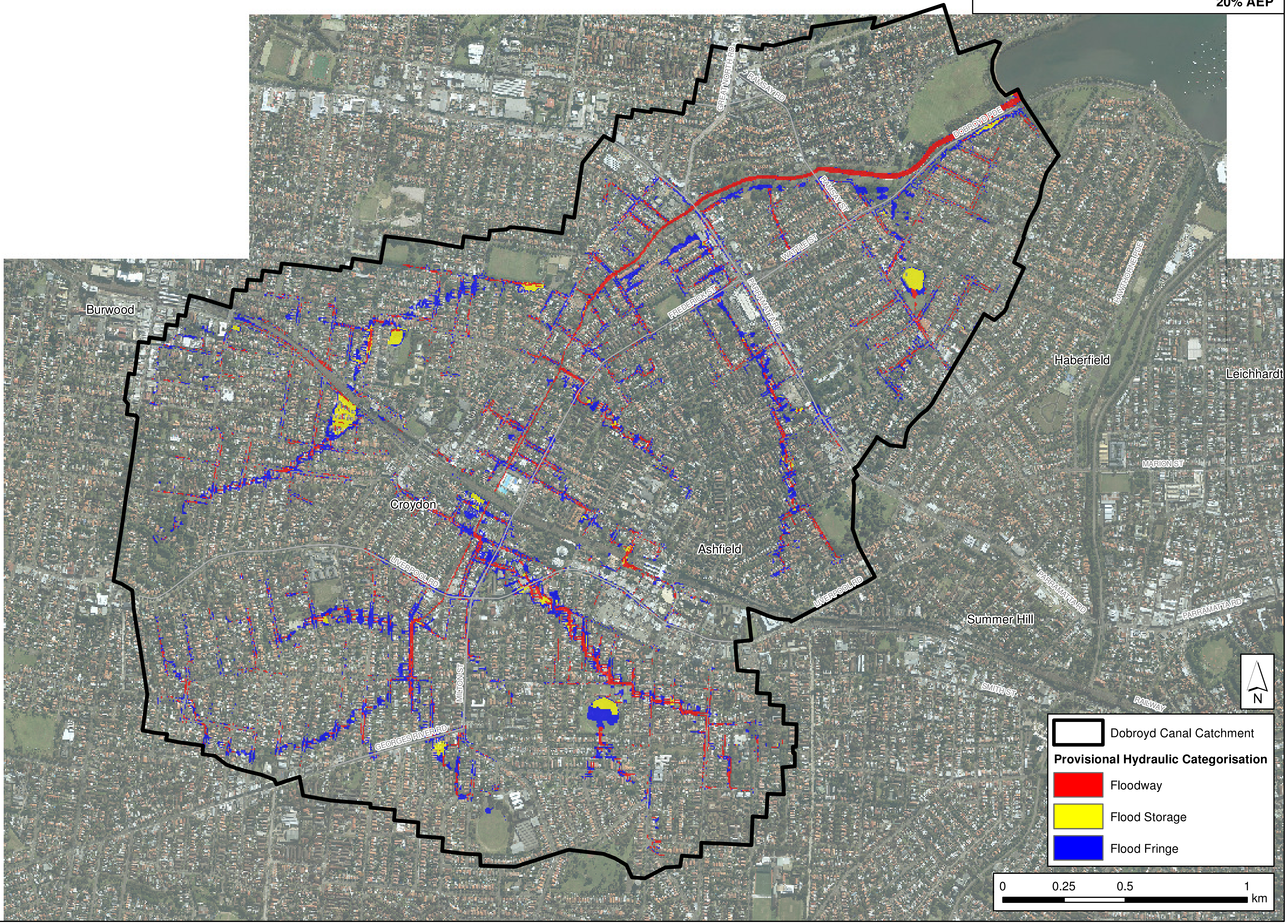

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FIGURE 26
PROVISIONAL HYDRAULIC CATEGORISATION
5% AEP

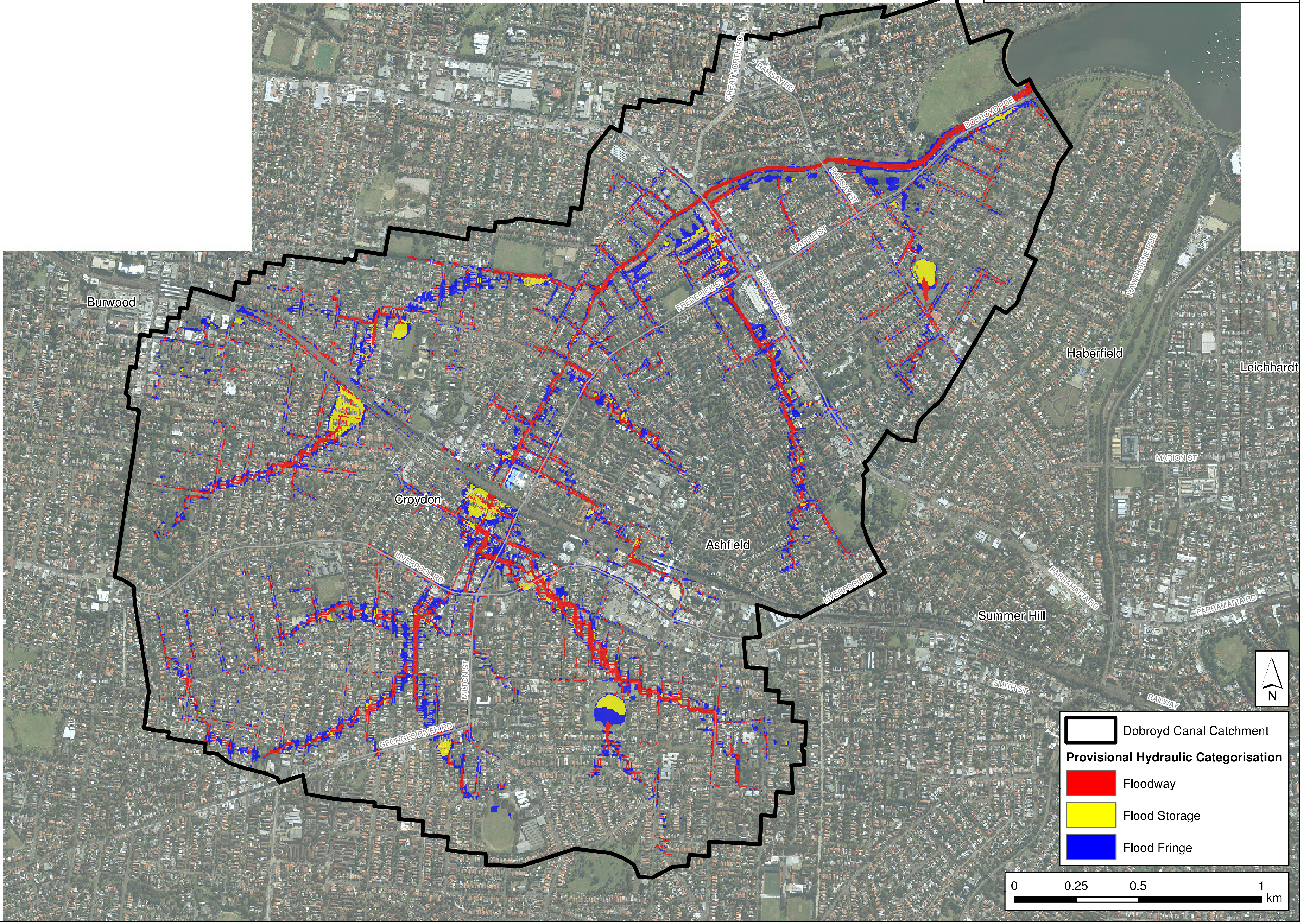

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FIGURE 27
PROVISIONAL HYDRAULIC CATEGORISATION
1% AEP

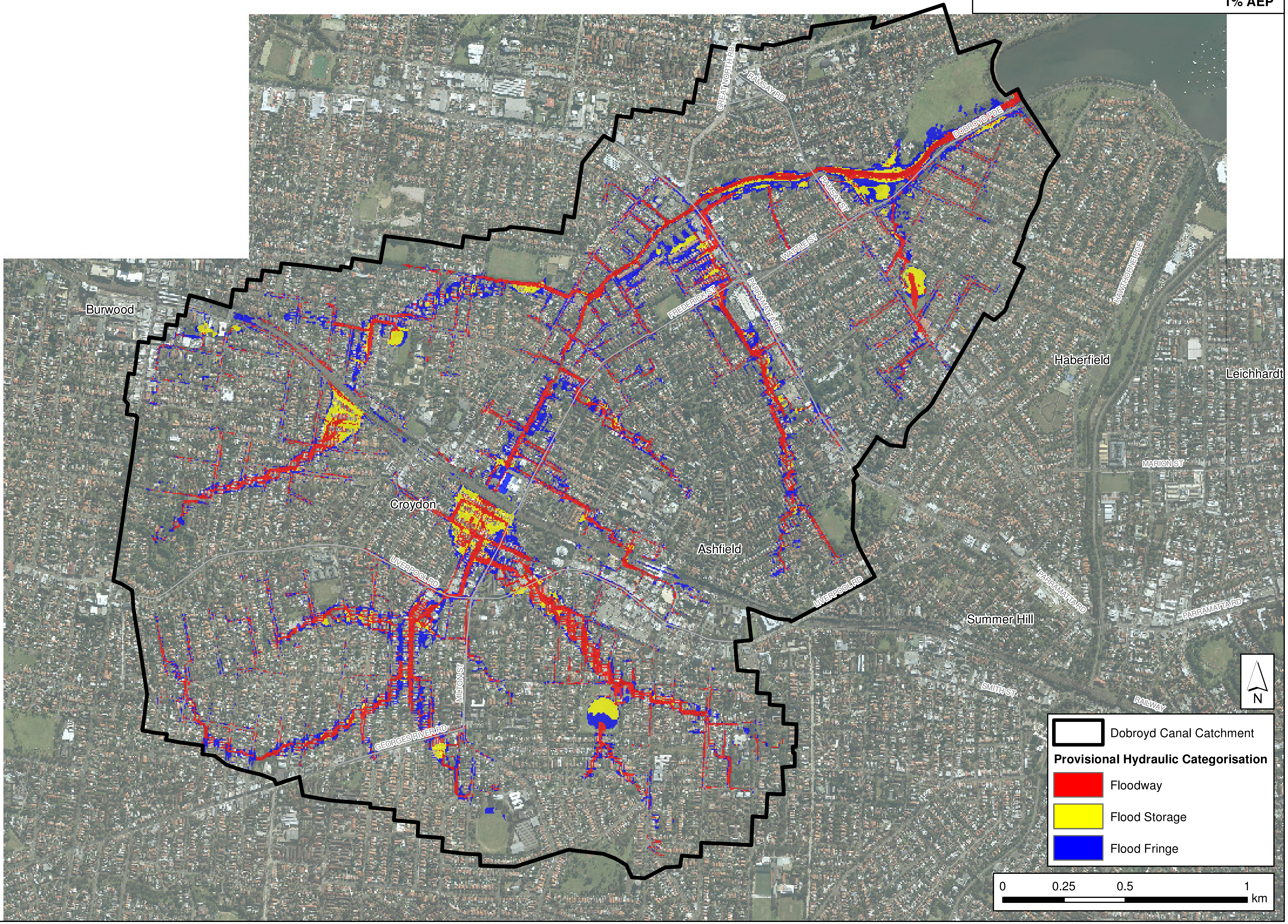

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FIGURE 28
PROVISIONAL HYDRAULIC CATEGORISATION
PMF

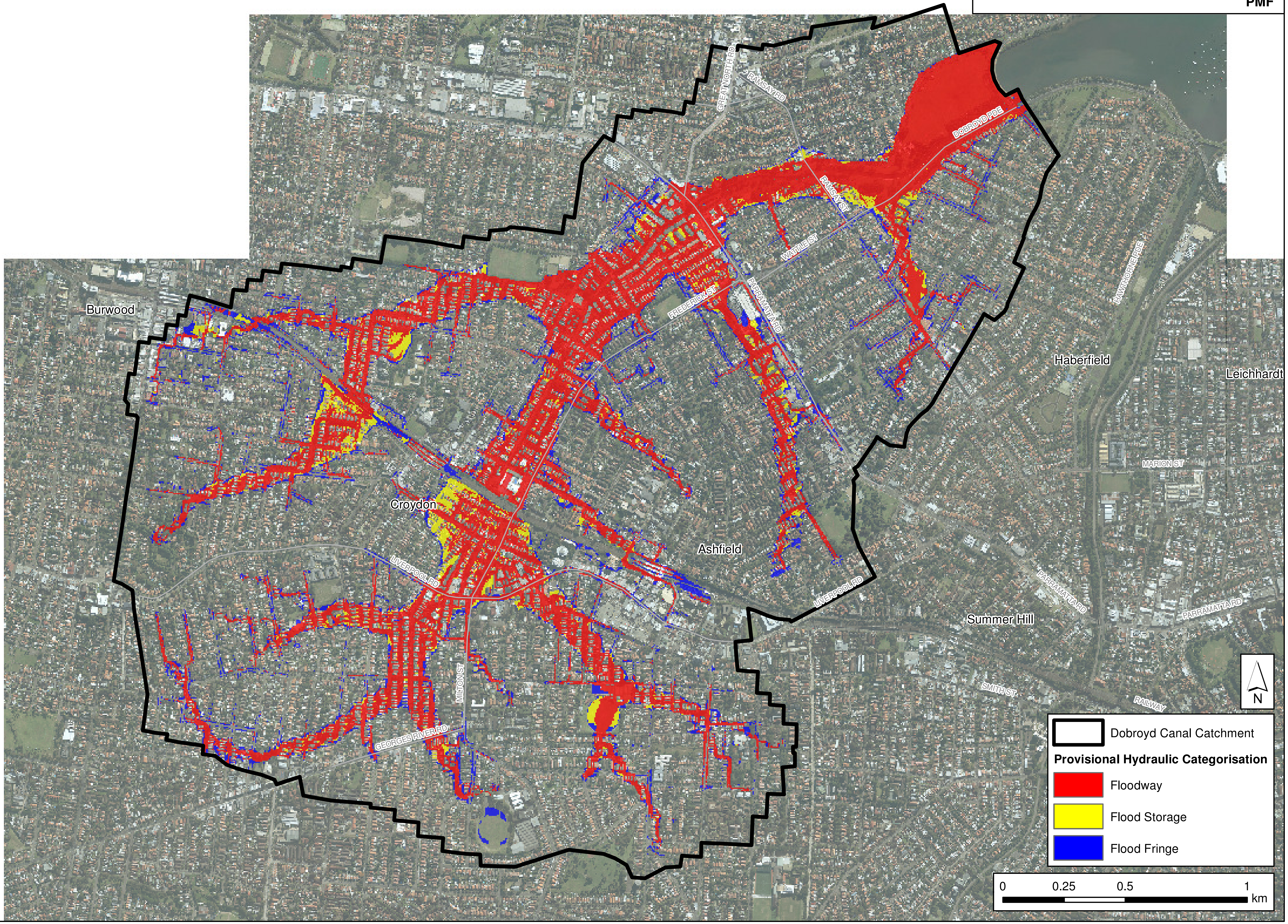

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FIGURE 29
PROVISIONAL HYDRAULIC CATEGORISATION
HIGH / LOW RISK
1% AEP

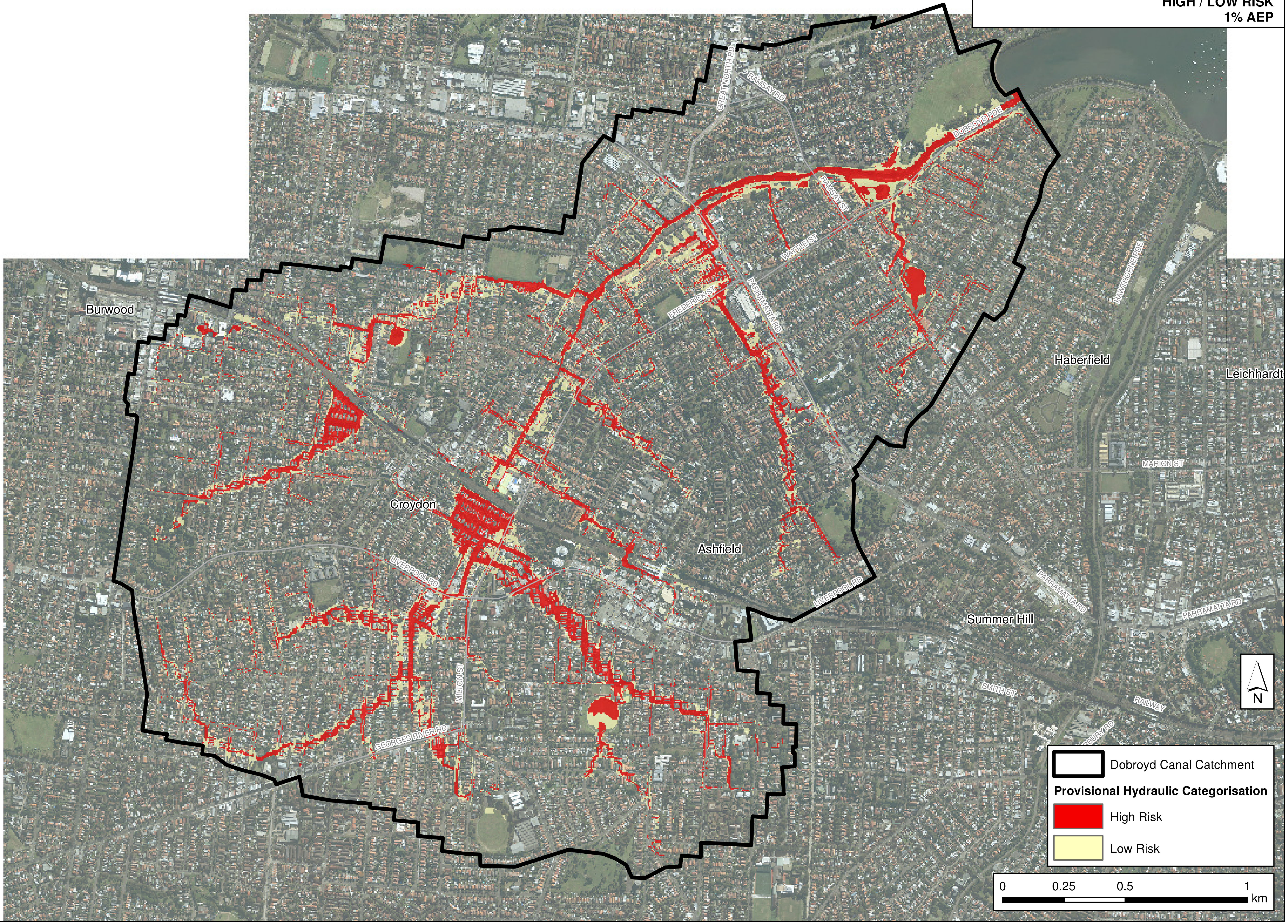

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FIGURE 30
PRELIMINARY FLOOD EMERGENCY RESPONSE CLASSIFICATION OF COMMUNITIES
1% AEP

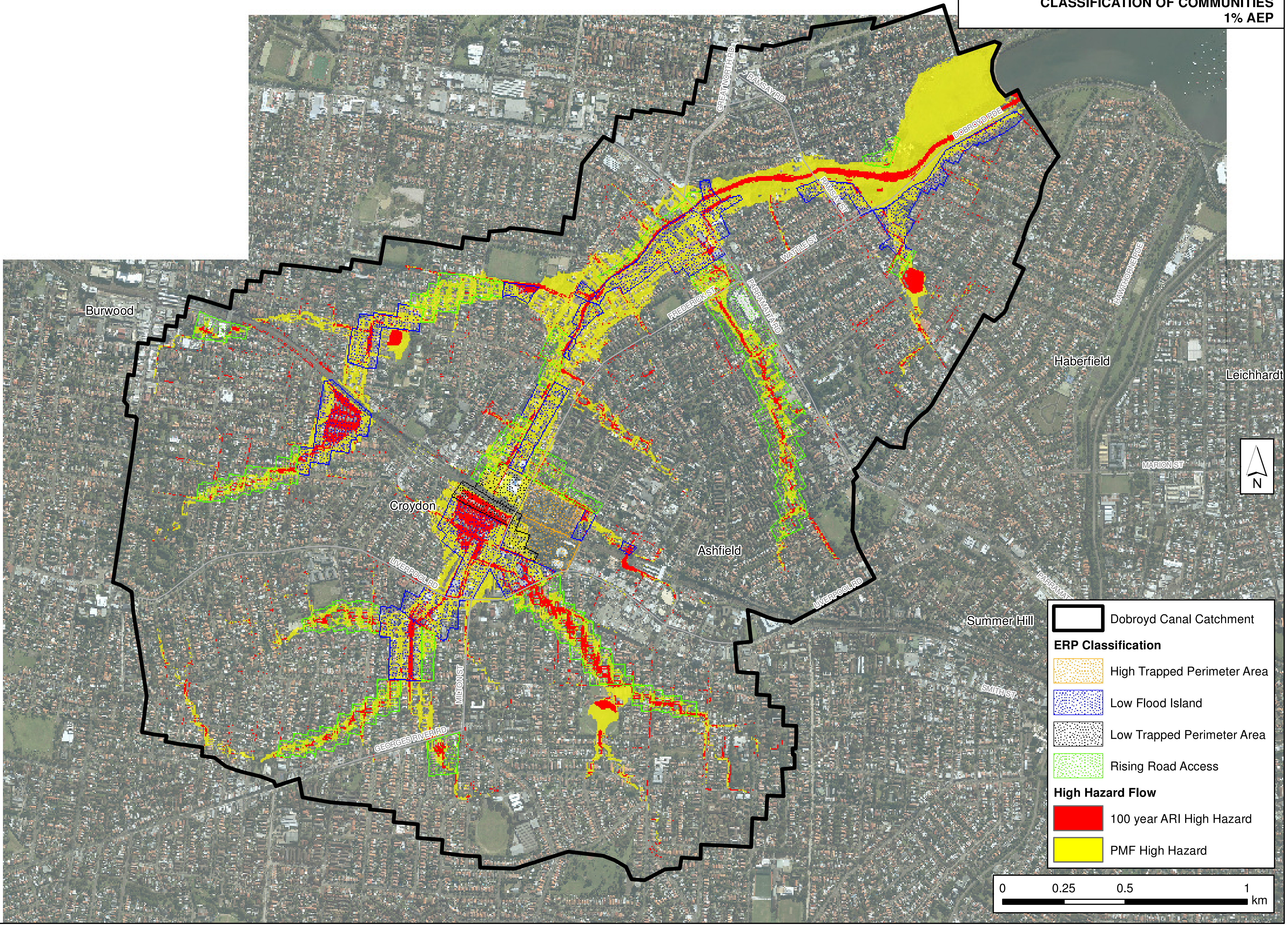

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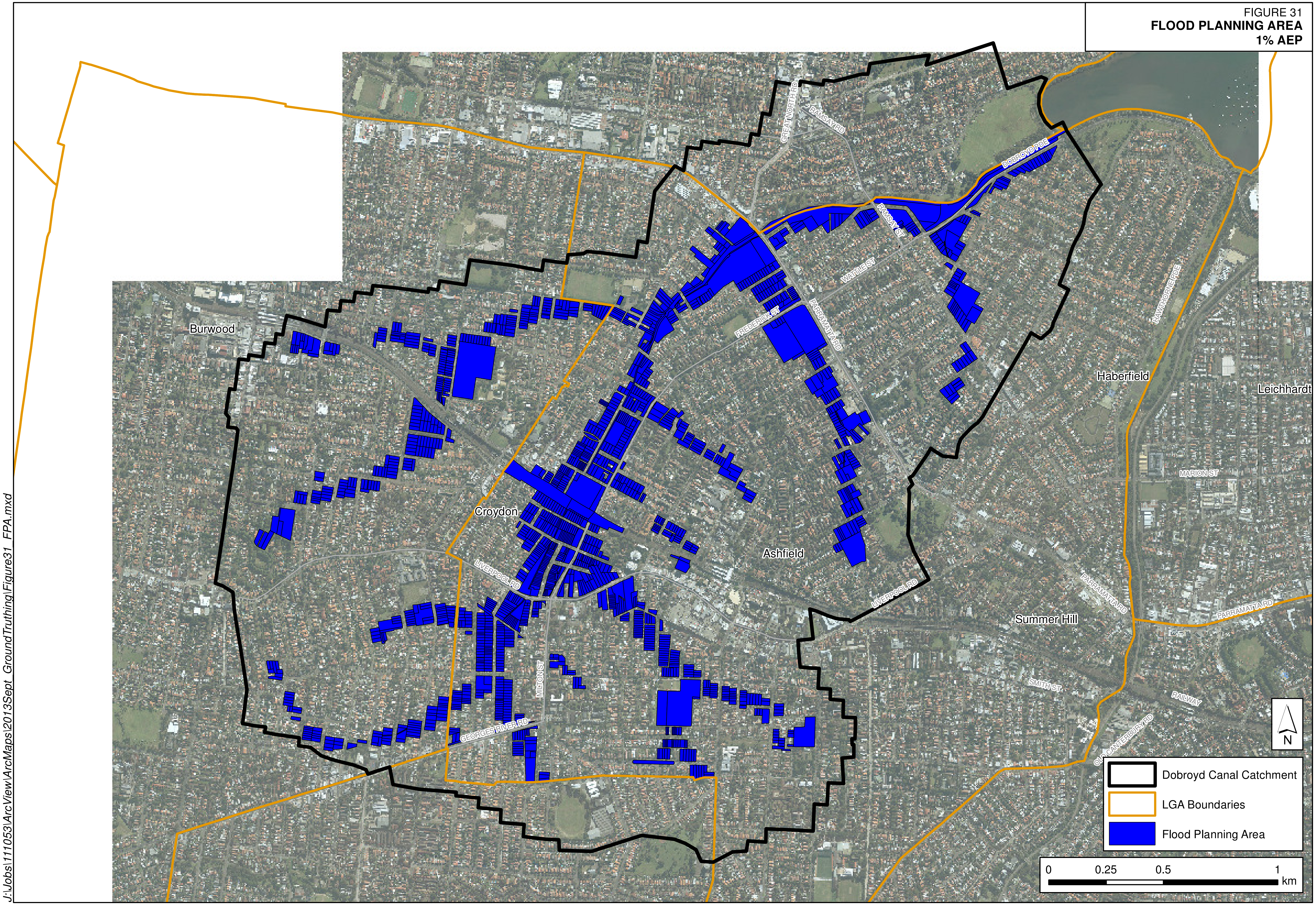
FIGURE 31
FLOOD PLANNING AREA
1% AEP



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Appendix A

APPENDIX A: GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

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

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

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

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

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

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

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

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

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

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

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

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

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

flood mitigation standard The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.

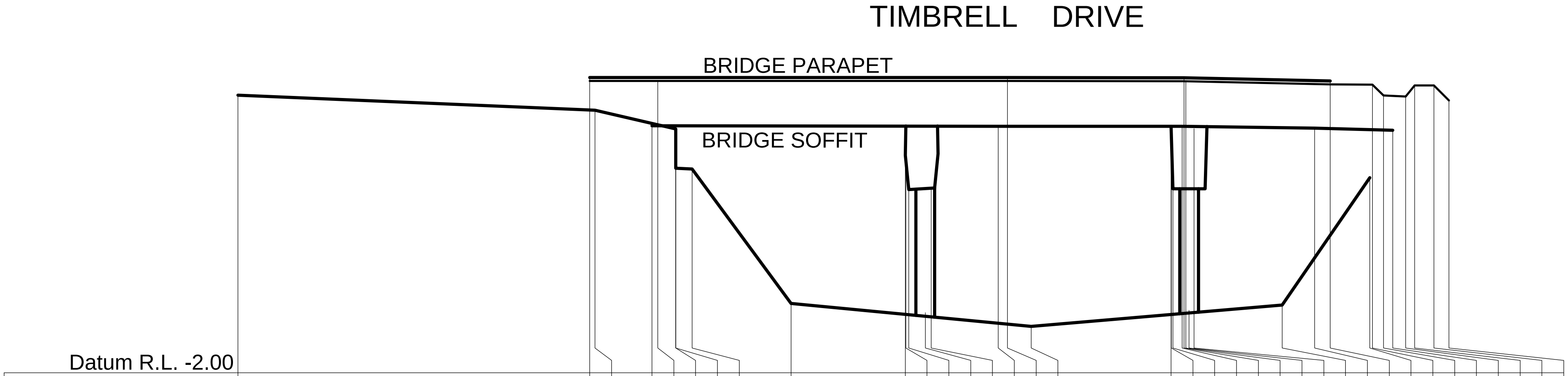
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain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

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

mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the States rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).

risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to water level. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B



CHAINAGE	SURFACE LEVEL
0.00	2.46
11.29	2.74
11.46	2.22
13.29	1.97
13.47	2.70
14.05	1.29
14.05	1.92
14.57	1.27
17.75	-0.89
21.41	1.50
21.43	1.92
21.53	0.94
22.06	-1.03
22.24	0.96
24.39	1.96
24.69	2.74
25.46	-1.25
29.94	1.49
29.95	1.92
30.00	0.96
30.30	1.96
30.36	2.74
30.42	2.68
30.52	-1.00
30.68	1.92
33.51	-0.91
34.55	1.93
35.05	2.69
36.32	1.13
36.40	2.63
36.76	2.46
37.05	1.90
37.47	2.44
37.76	2.61
38.37	2.62
38.86	2.37

D_St001 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:200 VERTICAL 1:100

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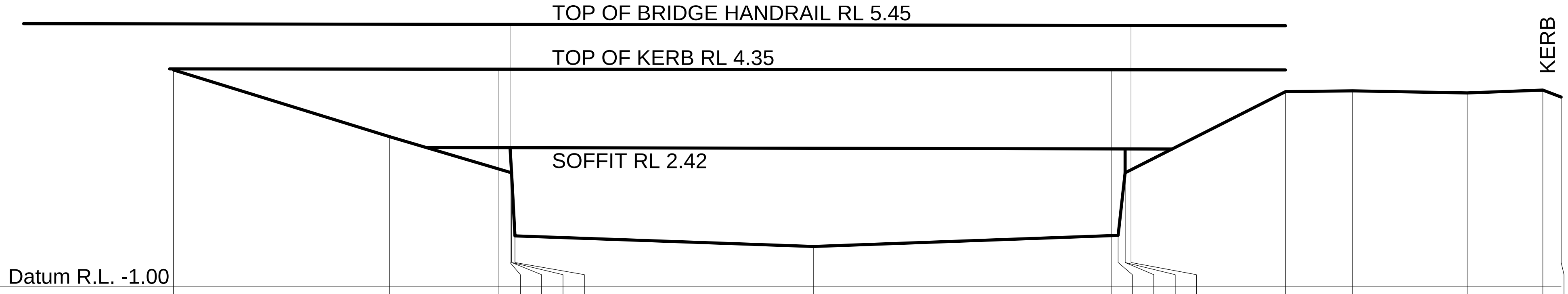
DOBROYD CHANNEL
BRIDGE CROSS SECTION D_St001
TIMBRELL DRIVE, FIVE DOCK



Datum R.L. -1.00

SURFACE LEVEL	4.33	2.69	4.35	5.45	1.80	2.42	0.25	-0.01	4.33	0.26	1.80	2.39	5.42	3.80	3.82	3.76	3.84	3.67
CHAINAGE	0.00	5.31	8.00	8.27	8.31	8.32	8.40	15.73	23.05	23.22	23.40	23.40	23.54	27.34	28.99	31.80	33.66	34.11

D_St002 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

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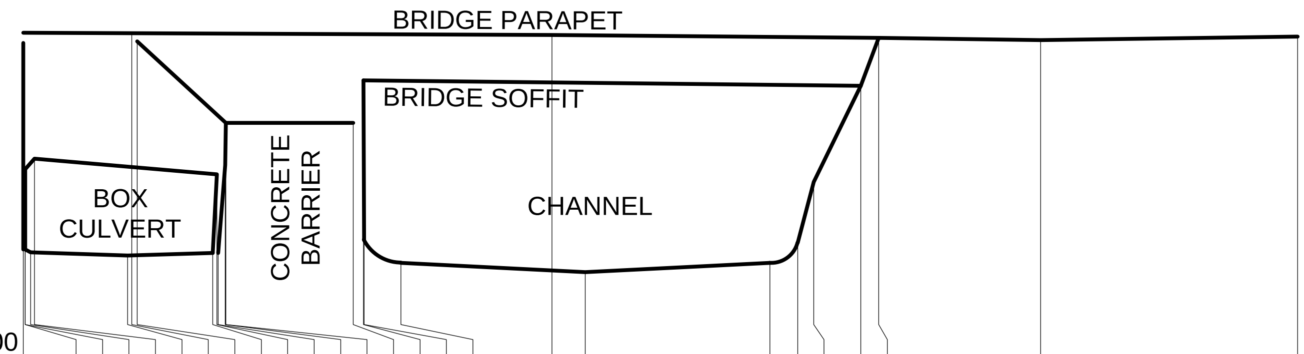
DOBROYD CHANNEL
BRIDGE CROSS SECTION D_St002
RAMSAY ROAD



Datum R.L. -1.00

SURFACE LEVEL	5.20	1.09	2.69	1.03	2.89	0.97	5.39	5.23	1.02	2.58	1.02	2.77	3.60	3.61	4.45	1.28	0.83	5.35	0.64	0.83	1.23	2.43	4.35	5.30	5.25	5.32
CHAINAGE	0.00	0.04	0.04	0.15	0.23	2.07	2.16	2.26	3.76	3.84	3.87	4.01	4.02	6.55	6.76	6.76	7.50	10.50	11.15	14.82	15.37	15.69	16.63	16.98	20.19	25.29

D_St003 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

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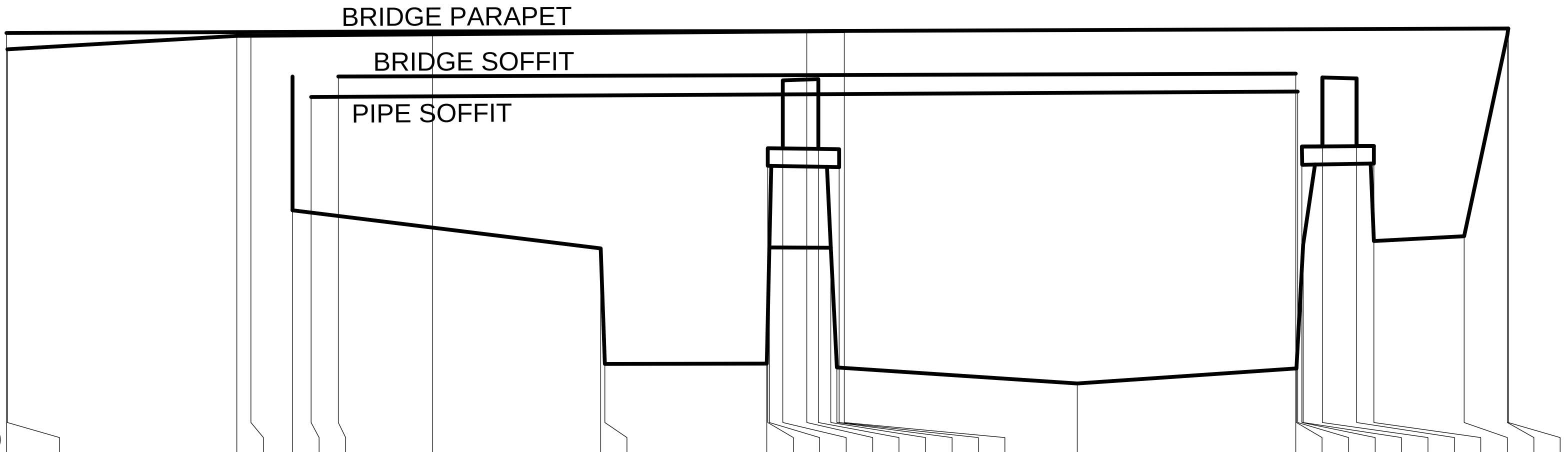
DOBROYD CHANNEL
BRIDGE CROSS SECTION D_St003
PARRAMATTA ROAD



Datum R.L. 1.00

CHAINAGE	SURFACE LEVEL
0.00	6.55
0.01	6.34
3.05	6.51
3.23	6.57
3.79	4.21
4.03	5.71
4.39	5.97
5.63	6.57
7.86	3.70
7.91	2.17
10.06	2.18
10.07	5.03
10.09	3.72
10.27	5.92
10.59	6.58
10.74	5.94
10.91	3.71
10.99	2.13
11.01	5.01
11.08	6.58
14.17	1.92
17.06	6.01
17.06	2.12
17.08	5.78
17.14	5.05
17.16	3.75
17.41	5.96
17.86	5.95
18.09	5.06
19.28	3.86
19.85	6.52
19.87	6.61

D_St004 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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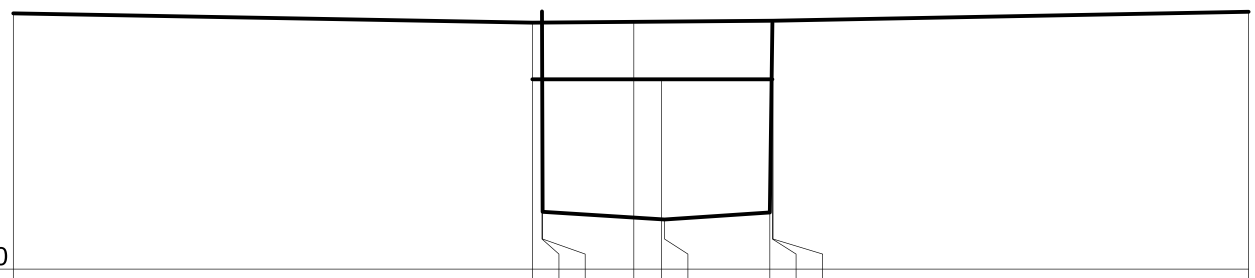
CULVERT CROSS SECTION D_St004
CHURCH STREET



Datum R.L. 2.00

SURFACE LEVEL	7.07	6.89	7.11	3.14	6.89	5.77	2.99	3.12	6.88	6.93	7.11
CHAINAGE	0.00	10.30	10.49	10.50	12.31	12.87	12.92	15.01	15.07	15.08	24.52

D_St005 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

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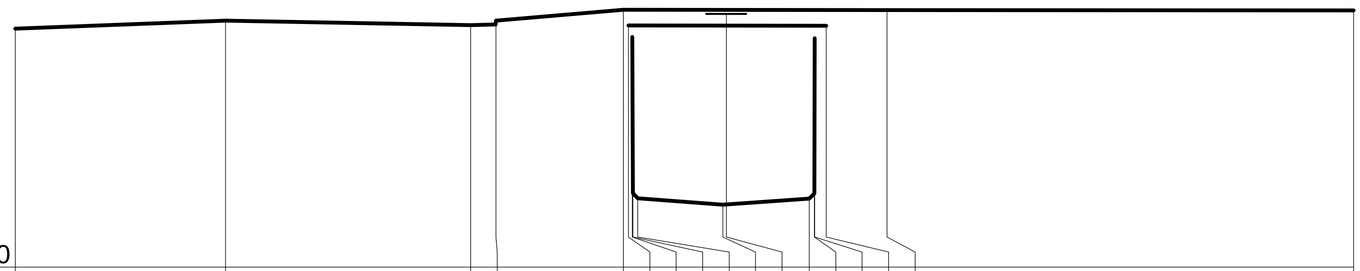
DOBROYD CHANNEL

BRIDGE CROSS SECTION D_St005

JOHN STREET



Datum R.L. 5.00

SURFACE LEVEL	9.73	9.89	9.80	9.89	10.11	9.79	9.57	6.47	6.37	6.24	10.03	6.36	6.46	9.54	9.79	10.10	10.09
CHAINAGE	0.00	4.17	9.04	9.54	12.07	12.17	12.24	12.25	12.35	14.04	14.11	15.76	15.85	15.86	16.09	17.30	26.56

D_St006 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

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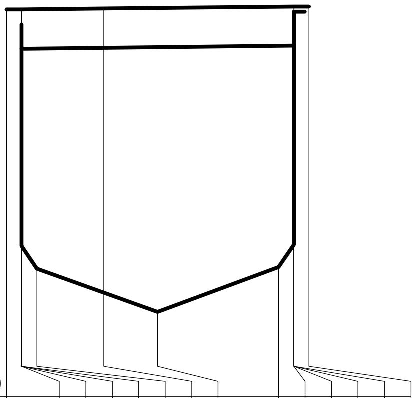
BRIDGE CROSS SECTION D_St006

BANKS STREET



Datum R.L. 7.00

SURFACE LEVEL	12.12	12.12	11.60	11.92	8.99	8.69	12.13	8.12	8.71	12.16	11.64	12.09	9.01	12.16
CHAINAGE	0.00	0.20	0.20	0.20	0.20	0.40	1.29	2.00	3.60	3.80	3.80	3.80	3.80	4.00

D_St007 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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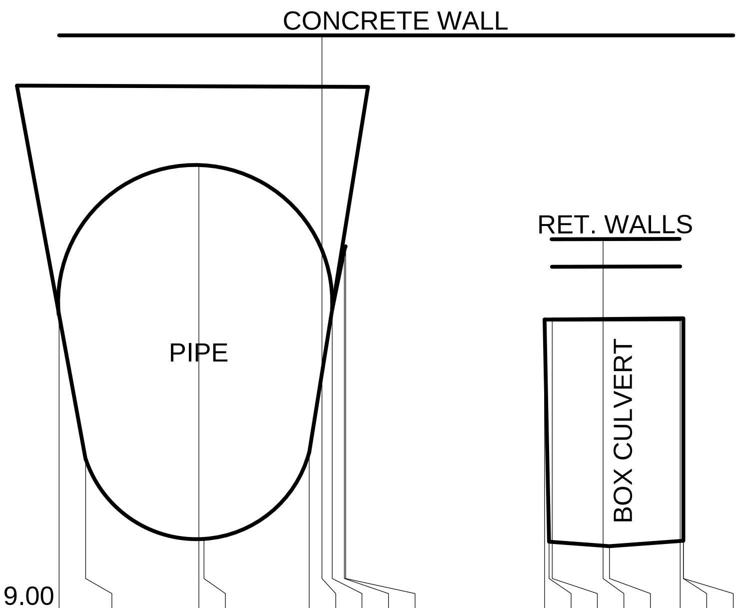
DOBROYD CHANNEL

CULVERT CROSS SECTION D_St007

ELIZABETH STREET

CULVERT DIAMETER	3.6m
CULVERT LENGTH	15.38m
UPSTREAM INVERT	RL 8.12
DOWNSTREAM INVERT	RL 7.43



Datum R.L. 9.00

SURFACE LEVEL	12.89	10.98	14.86	9.92	11.06	16.59	12.96	13.75	13.80	12.82	9.88	12.82	13.89	9.83	12.83	12.84	9.90
CHAINAGE	0.00	0.35	1.85	1.92	3.31	3.48	3.62	3.78	3.79	6.43	6.48	6.53	7.20	7.28	8.22	8.26	8.26

D_St008 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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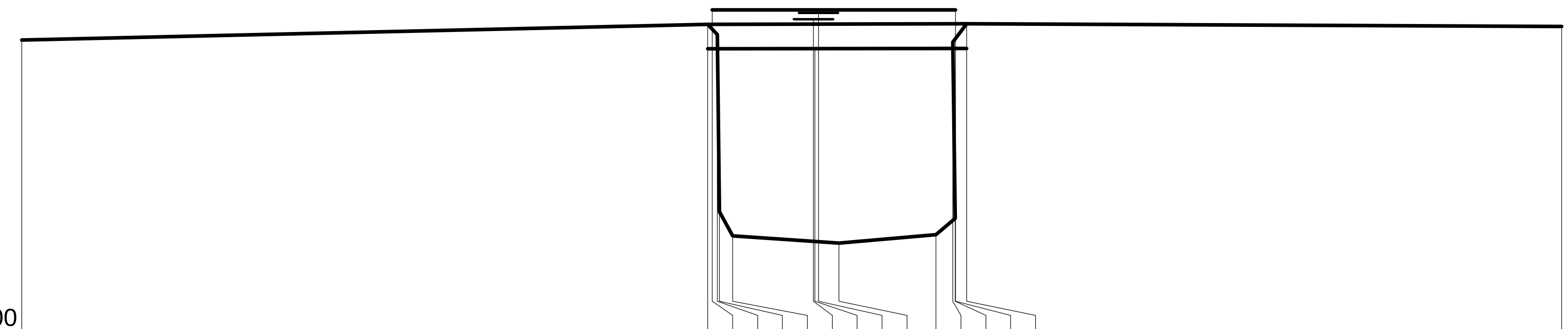
DOBROYD CHANNEL
CULVERT CROSS SECTION D_St008
RAILWAY LINE

PIPE DIAMETER	3.0m
PIPE LENGTH	37.02m
UPSTREAM INVERT	RL 9.92
DOWNSTREAM INVERT	RL 9.62



Datum R.L. 9.00

SURFACE LEVEL	13.07	13.29	13.49	13.15	10.66	10.32	13.36	12.95	13.45	10.22	10.33	13.04	10.56	13.50	13.30	13.26
CHAINAGE	0.00	9.65	9.71	9.78	9.81	10.00	11.13	11.15	11.20	11.49	12.85	13.09	13.12	13.13	13.29	21.65

D_St009 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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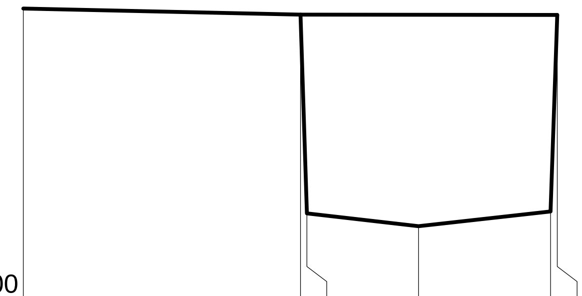
DOBROYD CHANNEL
CULVERT CROSS SECTION D_St009
HEIGHWAY AVENUE

CULVERT DIAMETER	3.31m
CULVERT LENGTH	14.10m
UPSTREAM INVERT	RL 10.22
DOWNSTREAM INVERT	RL 10.10



Datum R.L. 10.00

SURFACE LEVEL	13.81	13.73	11.10	10.93	11.13	13.73
CHAINAGE	0.00	3.67	3.75	5.24	6.98	7.07

D_St010 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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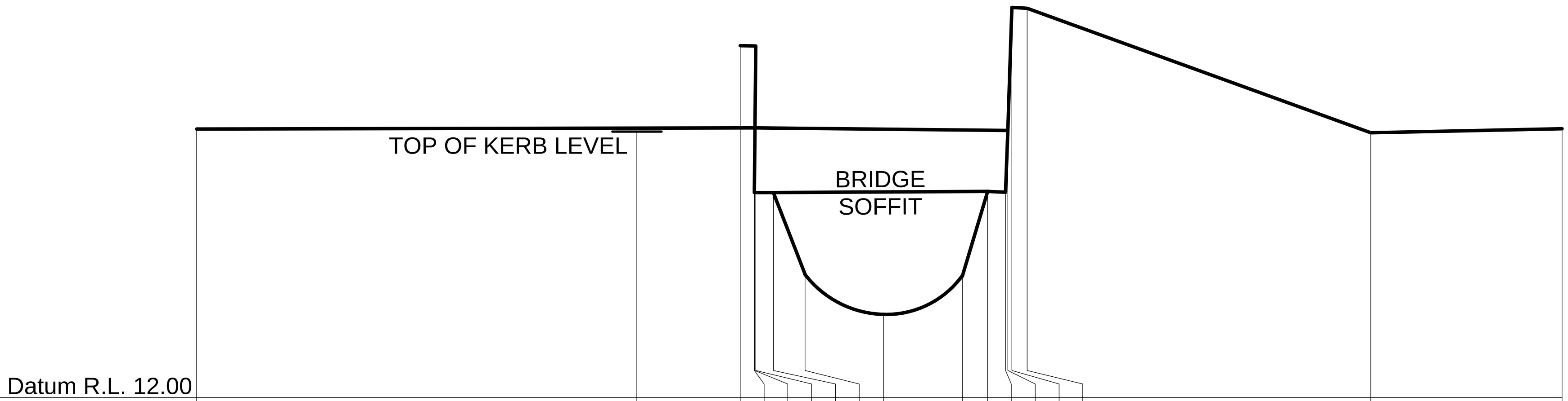
CHASE BURKE HARVEY

DOBROYD CHANNEL

CULVERT CROSS SECTION D_St010

THOMAS STREET



SURFACE LEVEL	15.96	15.92	17.18	15.02	15.97	17.18	15.02	13.82	13.22	13.79	15.03	15.02	15.93	17.74	17.73	15.90	15.96
CHAINAGE	0.00	6.47	7.99	8.19	8.20	8.22	8.48	8.94	10.10	11.25	11.62	11.89	11.92	11.98	12.20	17.26	20.07

D_St011 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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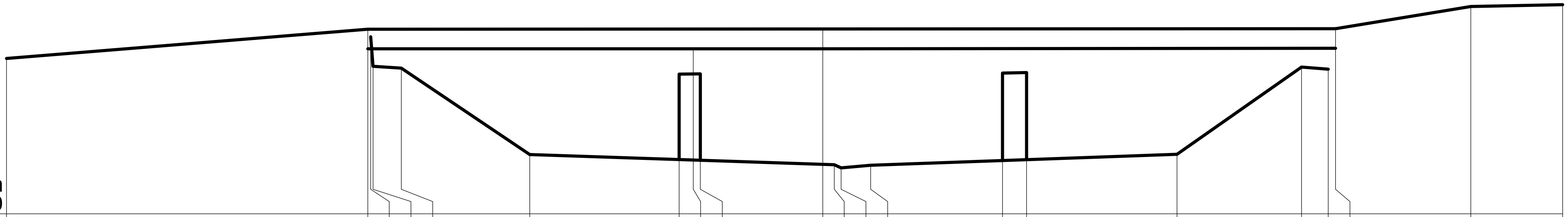
CHASE BURKE HARVEY

DOBROYD CHANNEL
CULVERT CROSS SECTION D_St011
HUME HIGHWAY



Datum
R.L. -2.00

SURFACE LEVEL	1.78	2.49	2.31	1.59	1.54	-0.55	1.40	2.02	1.40	2.50	-0.80	-0.88	-0.81	1.43	1.43	-0.55	1.57	1.52	2.51	3.04	3.09
CHAINAGE	0.00	8.77	8.84	8.90	9.58	12.70	16.32	16.67	16.84	19.81	20.09	20.26	20.98	24.18	24.76	28.41	31.43	32.08	32.26	35.54	37.77

D_St012 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:150 VERTICAL 1:150

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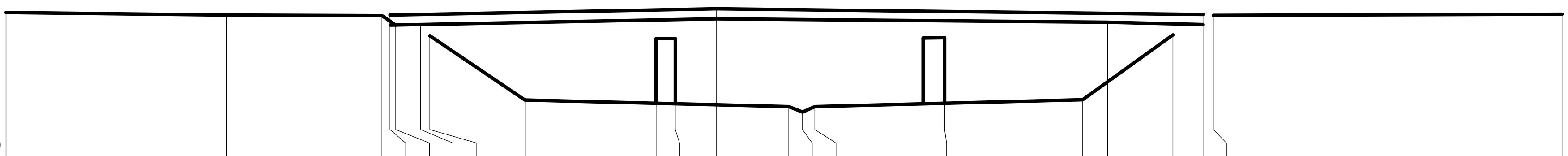
DOBROYD CHANNEL

FOOTBRIDGE CROSS SECTION D_St012

CRANE AVENUE, FIVE DOCK



Datum R.L. -2.00

SURFACE LEVEL	2.25	2.17	2.16	2.17	1.88	1.89	1.57	-0.33	1.48	1.48	2.36	-0.53	-0.68	-0.52	1.50	1.51	-0.32	1.96	1.59	2.19	2.17	2.20
CHAINAGE	0.00	6.52	11.11	11.35	11.52	12.25	12.52	15.33	19.21	19.78	21.00	23.13	23.54	23.90	27.10	27.74	31.82	32.56	34.48	35.37	35.68	45.99

D_St013 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:200 VERTICAL 1:200

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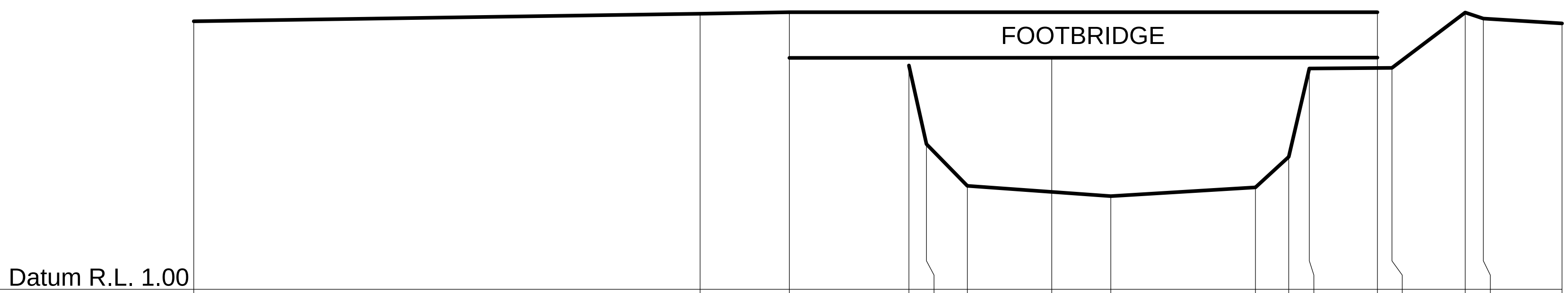
DOBROYD CHANNEL

FOOTBRIDGE CROSS SECTION D_St013

WARATAH STREET, FIVE DOCK



SURFACE LEVEL	4.75	4.86	4.88	4.14	3.03	2.45	4.24	2.30	2.43	2.85	4.09	4.88	4.10	4.87	4.79	4.72
CHAINAGE	0.00	7.08	8.33	10.00	10.25	10.82	12.00	12.83	14.85	15.32	15.61	16.56	16.76	17.79	18.04	19.14

D_St014 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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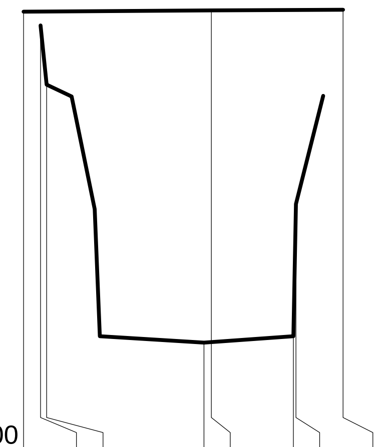
FOOTBRIDGE CROSS SECTION D_St014
ALEXANDRA STREET



Datum R.L. 3.00

SURFACE LEVEL	8.77	8.59	7.80	4.39	8.78	4.47	6.22	8.79
CHAINAGE	0.00	0.23	0.31	2.38	2.48	3.56	3.60	4.22

D_St015 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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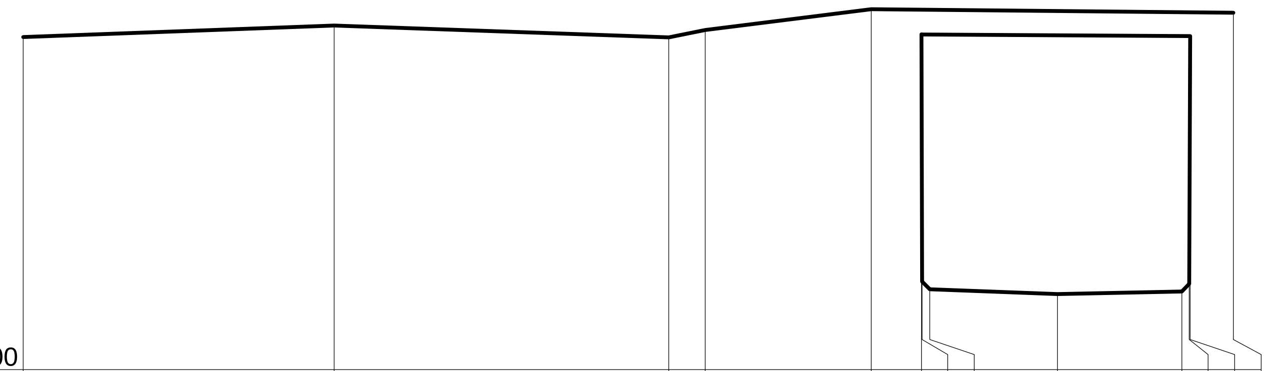
DOBROYD CHANNEL

FOOTBRIDGE CROSS SECTION D_St015

BETWEEN GREGORY AVE & HEDGER AVE



Datum R.L. 6.00

SURFACE LEVEL	10.40	10.55	10.40	10.49	10.77	10.43	7.17	7.07	7.00	7.04	7.14	10.41	10.72
CHAINAGE	0.00	4.12	8.54	9.03	11.23	11.89	11.90	12.00	13.69	15.33	15.43	15.44	16.02

D_St016 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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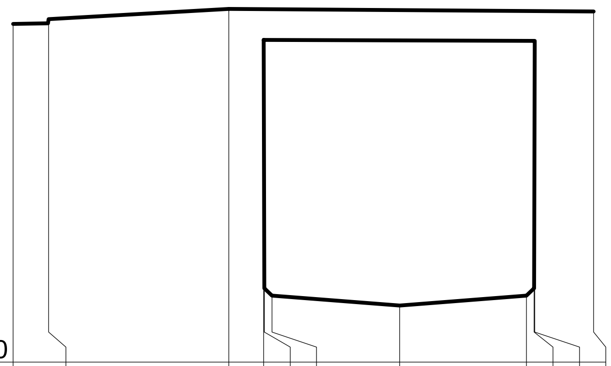
CHASE BURKE HARVEY

DOBROYD CHANNEL

FOOTBRIDGE CROSS SECTION D_St016

ETONVILLE PARADE NEAR ANTHONY STREET



Datum R.L. 7.00

SURFACE LEVEL	11.48	11.54	11.68	11.27	7.98	7.88	7.75	7.88	7.98	11.26	11.65
CHAINAGE	0.00	0.47	2.86	3.32	3.33	3.43	5.12	6.79	6.89	6.90	7.68

D_St017 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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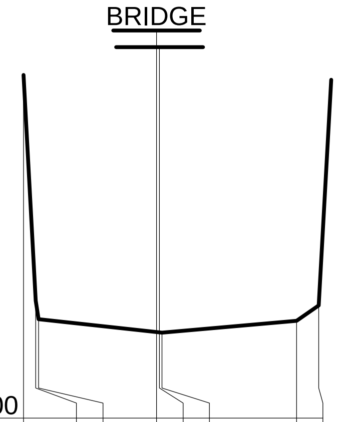
DOBROYD CHANNEL

FOOTBRIDGE CROSS SECTION D_St017

ETONVILLE PARADE



Datum R.L. 8.00

SURFACE LEVEL	12.54	9.56	9.31	13.13	12.91	9.13	9.29	9.49
CHAINAGE	0.00	0.16	0.20	1.76	1.80	1.83	3.61	3.90

D_St018 (UPSTREAM APPROACH)

SCALE HORIZONTAL 1:100 VERTICAL 1:100

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CHASE BURKE HARVEY

DOBROYD CHANNEL

FOOTBRIDGE CROSS SECTION D_St018

NORTH OF RAILWAY LINE

Appendix C

FIGURE C1
HOTSPOT AND AREA OF INTEREST LOCATIONS

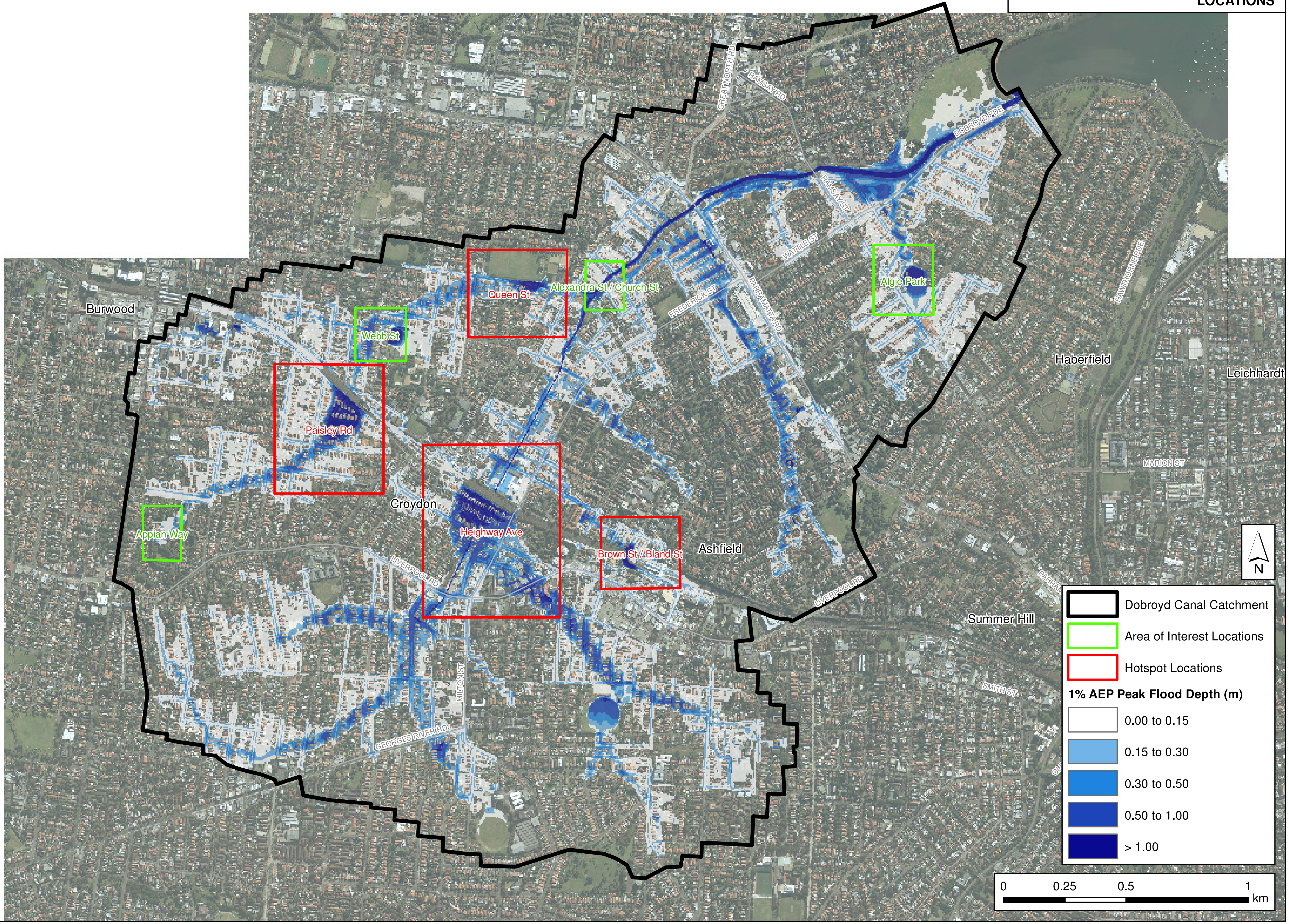

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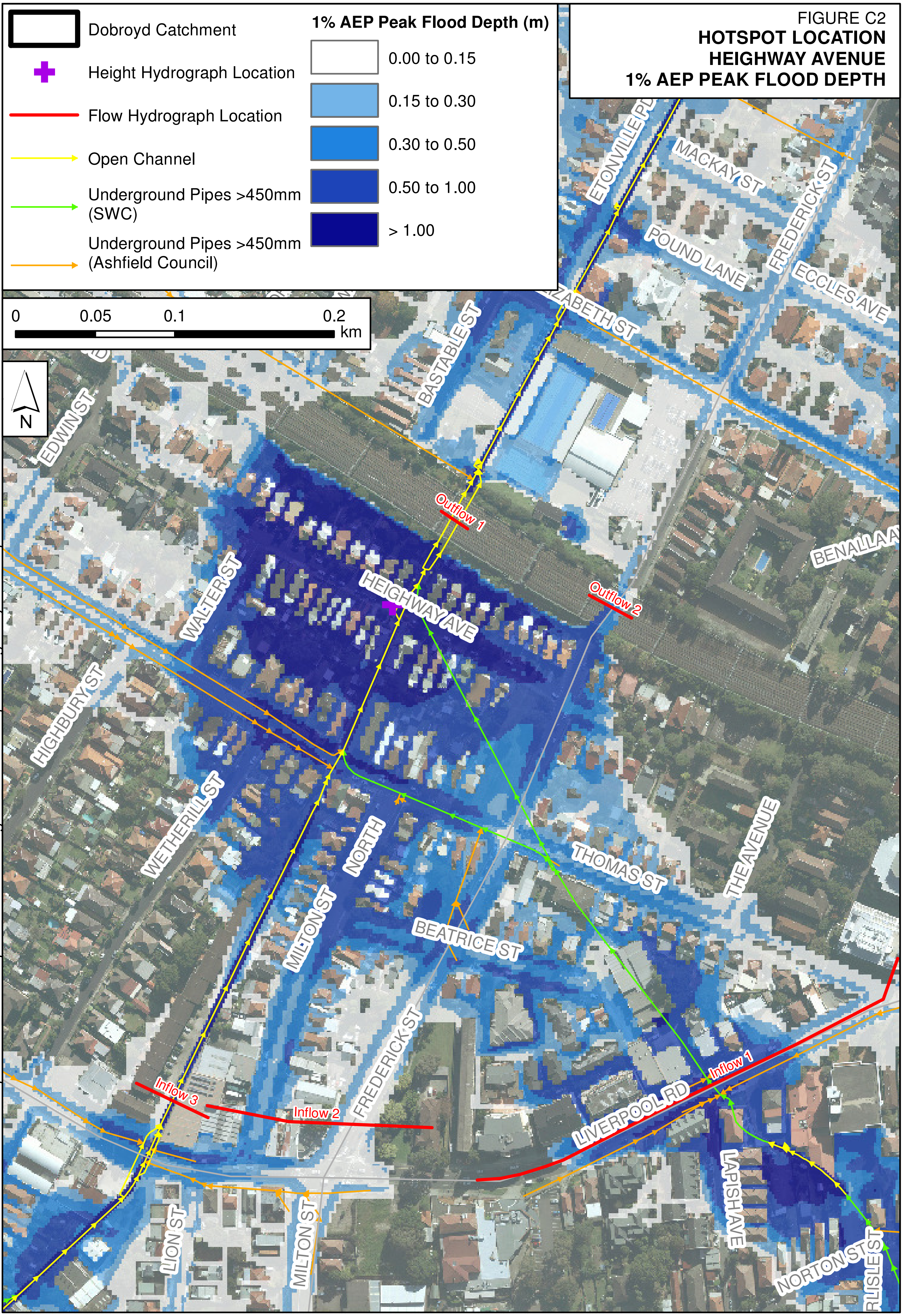
FIGURE C2
HOTSPOT LOCATION
HEIGHWAY AVENUE
1% AEP PEAK FLOOD DEPTH



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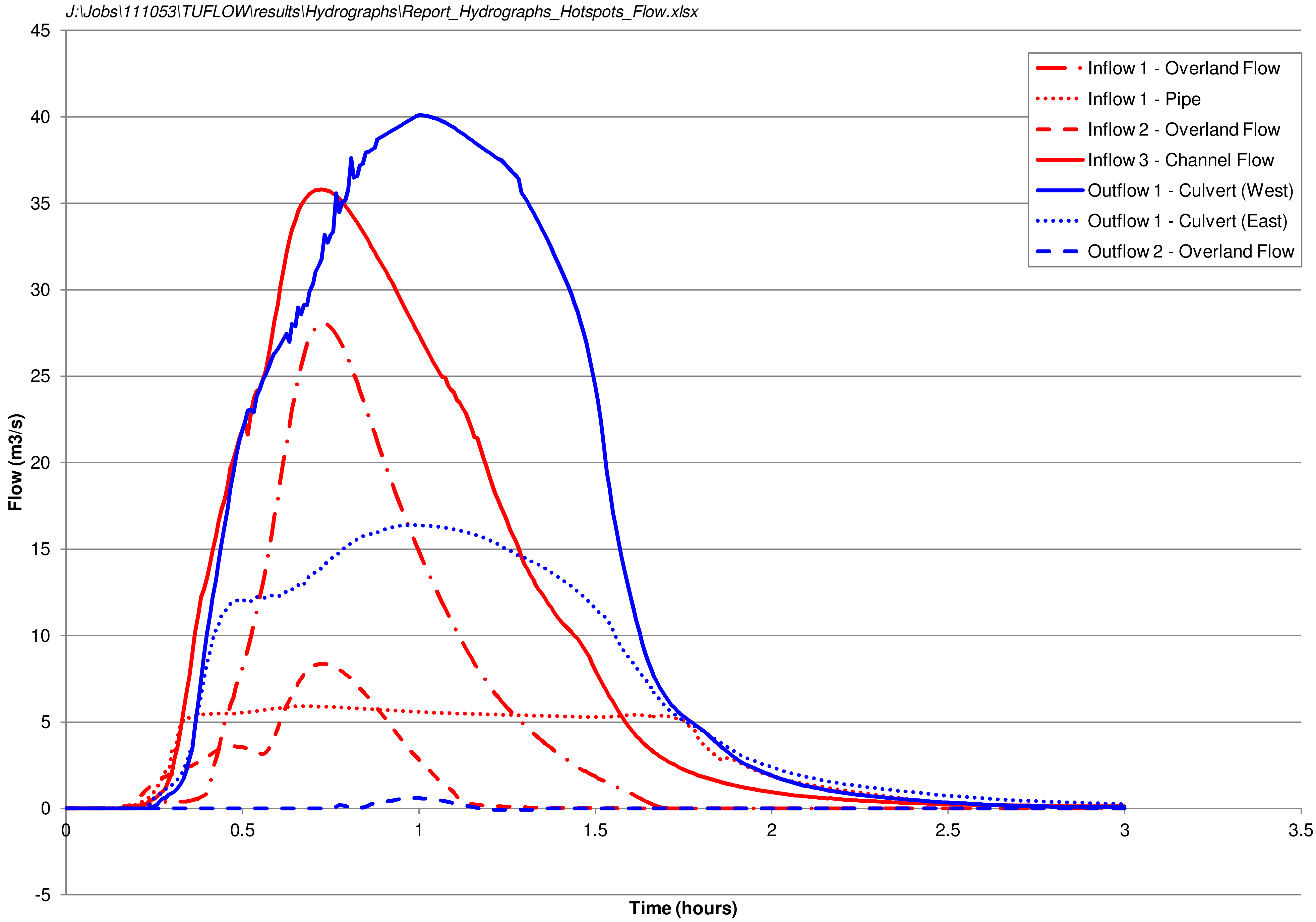

FIGURE C3
HOTSPOT LOCATION
HEIGHWAY AVENUE
1% AEP FLOW HYDROGRAPHS

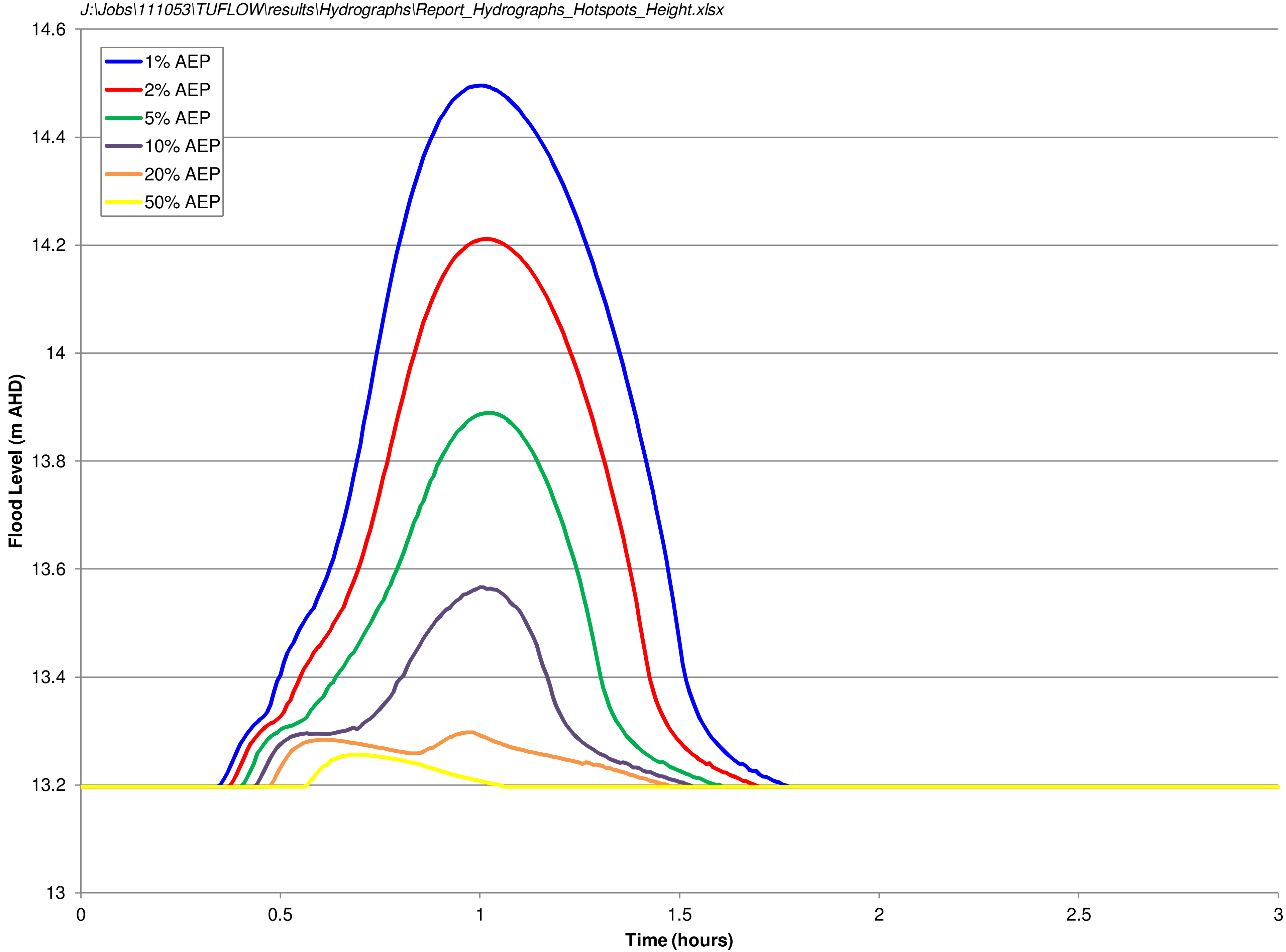

FIGURE C4
HOTSPOT LOCATION
HEIGHWAY AVENUE
FLOOD LEVEL HYDROGRAPH

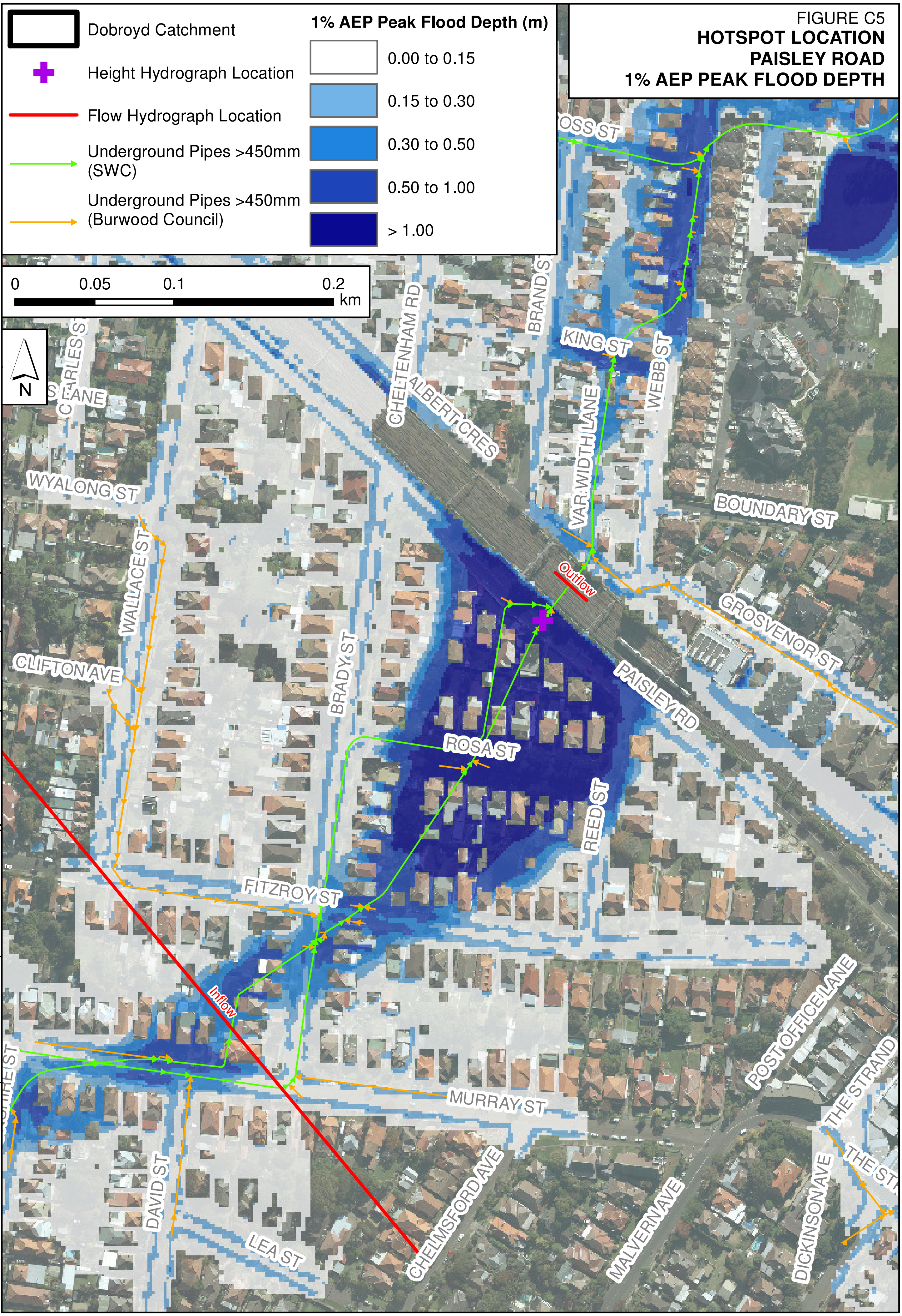

FIGURE C5
HOTSPOT LOCATION
PAISLEY ROAD
1% AEP PEAK FLOOD DEPTH

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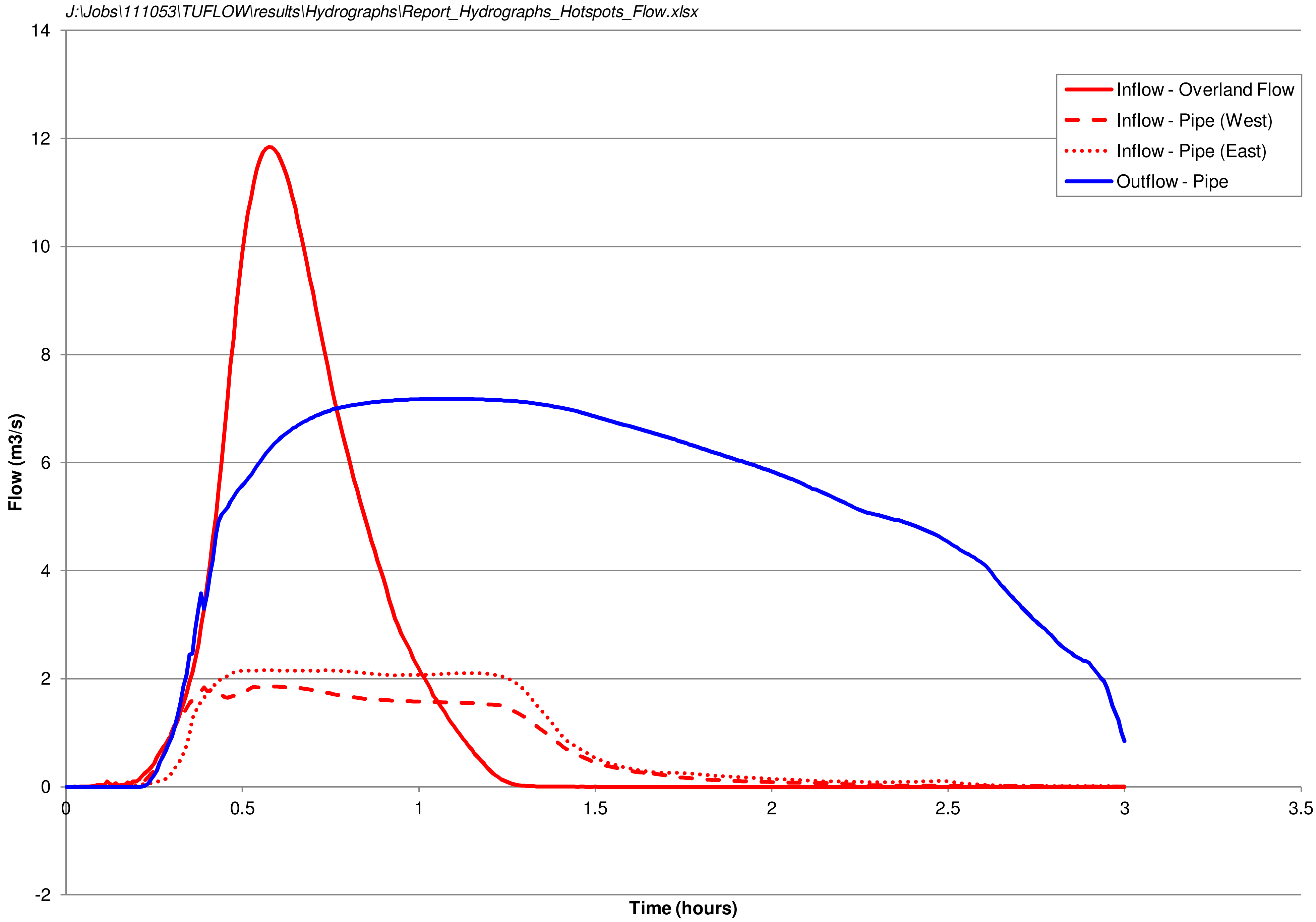


FIGURE C6
HOTSPOT LOCATION
PAISLEY ROAD
1% AEP FLOW HYDROGRAPHS

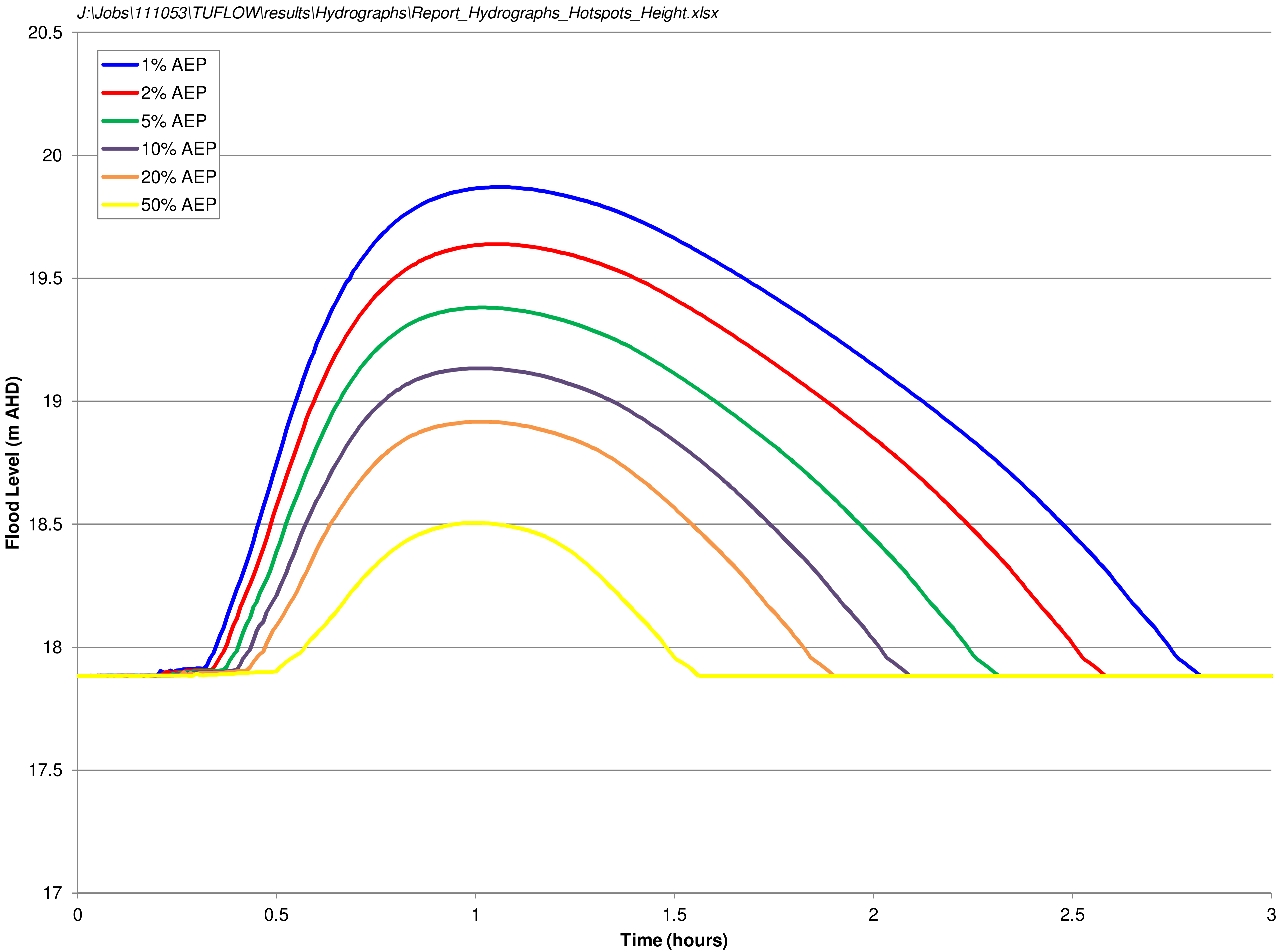
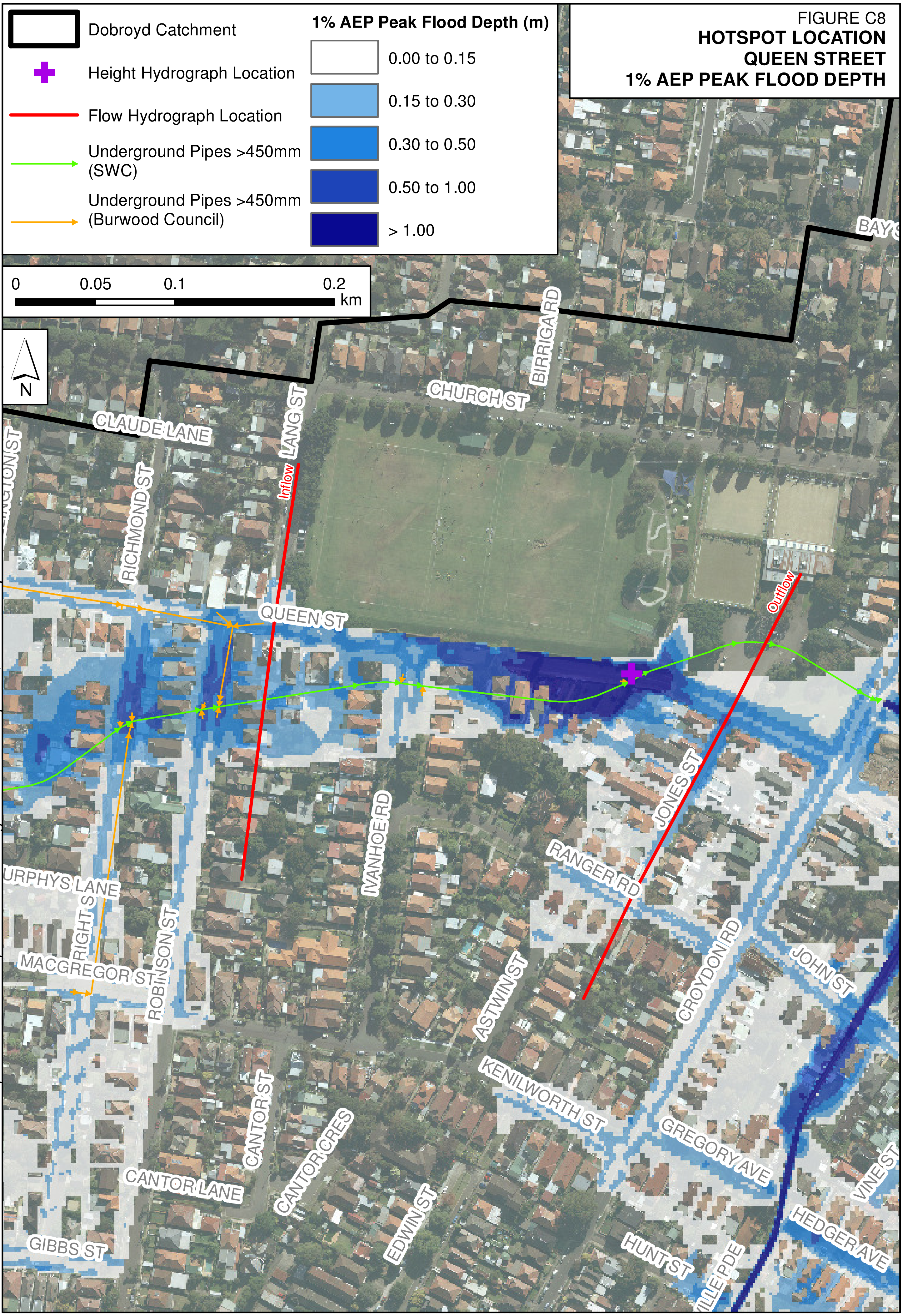


FIGURE C7
HOTSPOT LOCATION
PAISLEY ROAD
FLOOD LEVEL HYDROGRAPH



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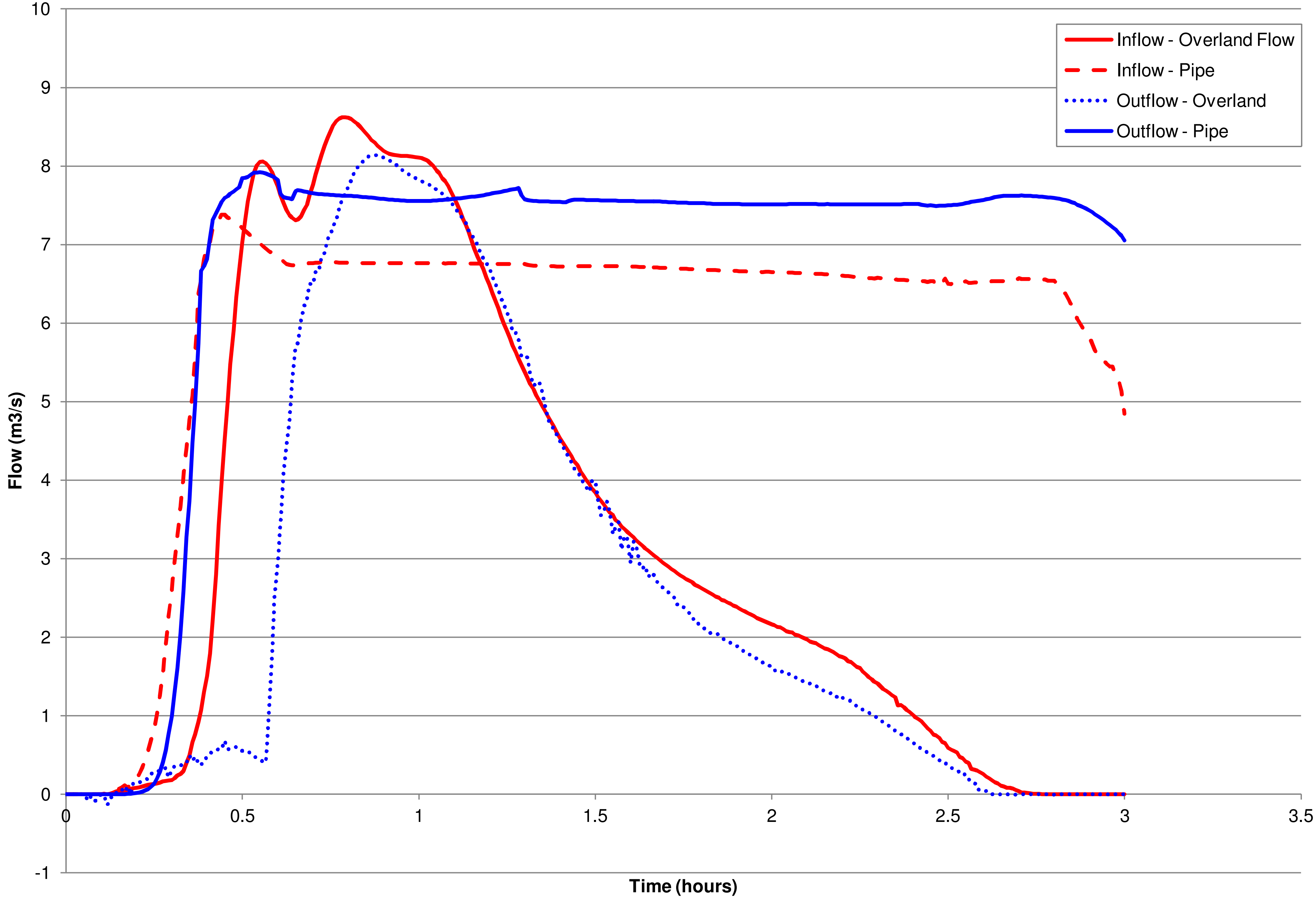

FIGURE C9
HOTSPOT LOCATION
QUEEN STREET
1% AEP FLOW HYDROGRAPHS

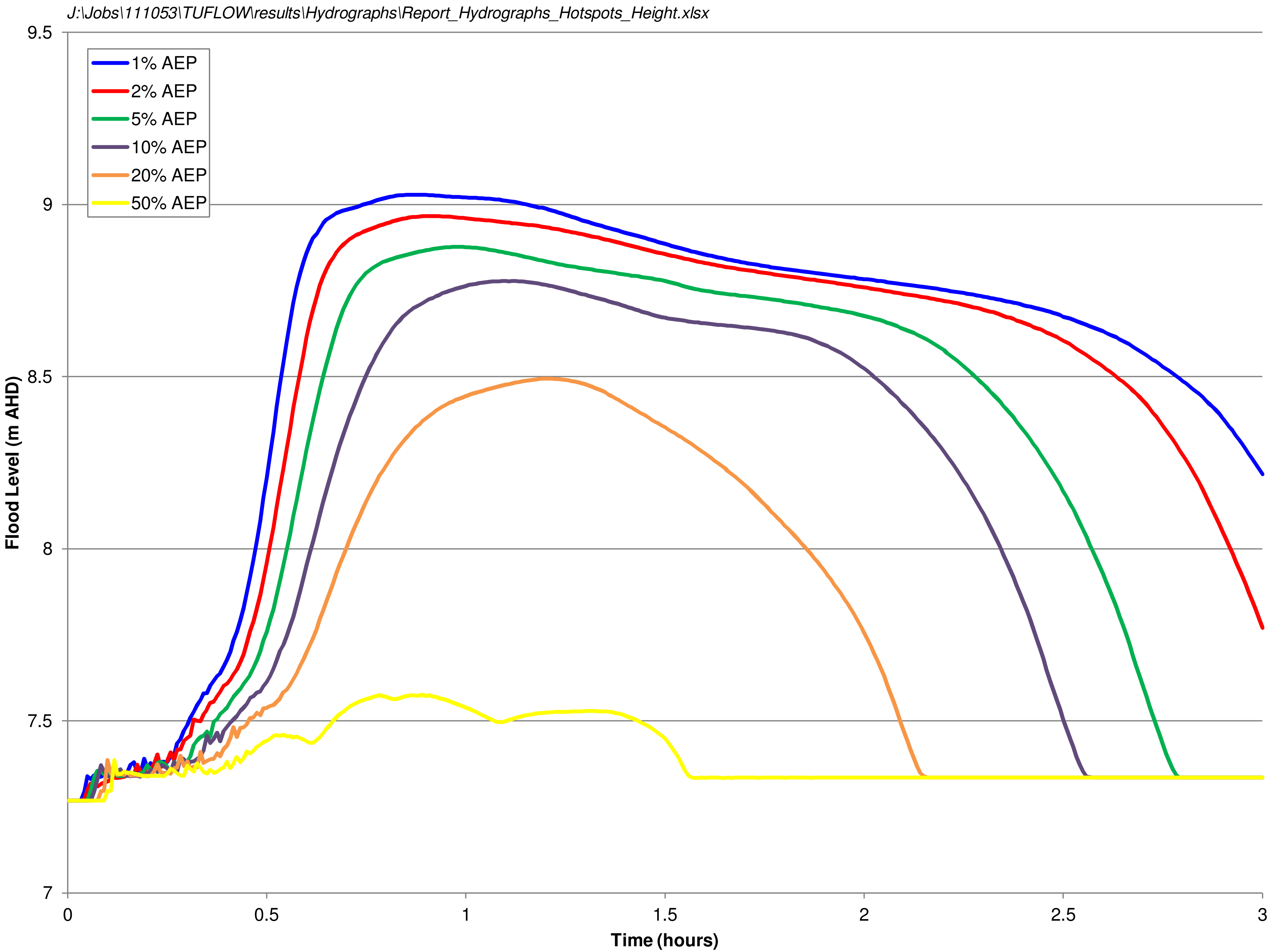

FIGURE C10
HOTSPOT LOCATION
QUEEN STREET
FLOOD LEVEL HYDROGRAPH

FIGURE C11
HOTSPOT LOCATION
BROWN STREET / BLAND STREET
1% AEP PEAK FLOOD DEPTH

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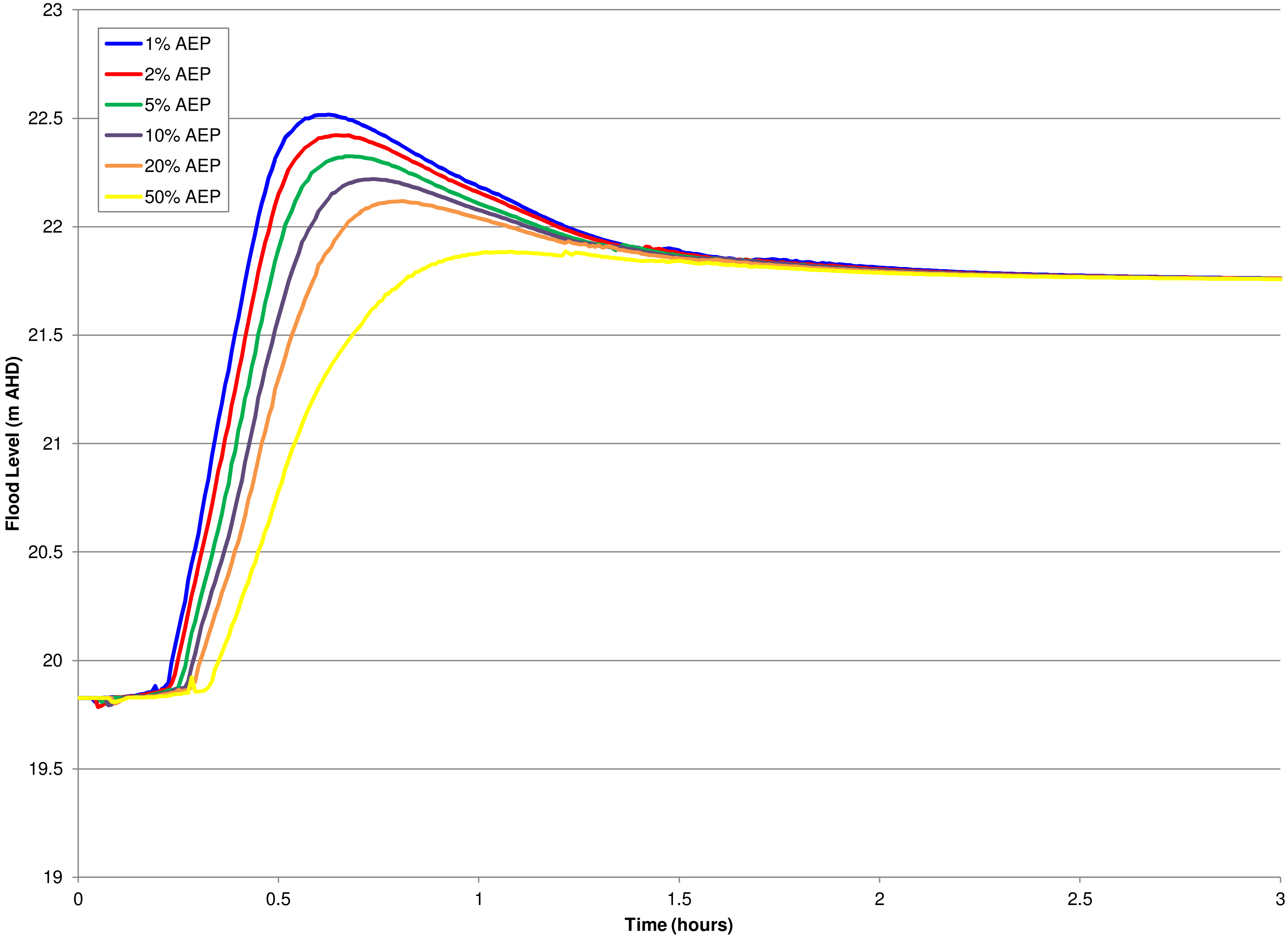

FIGURE C12
HOTSPOT LOCATION
BROWN STREET
FLOOD LEVEL HYDROGRAPH

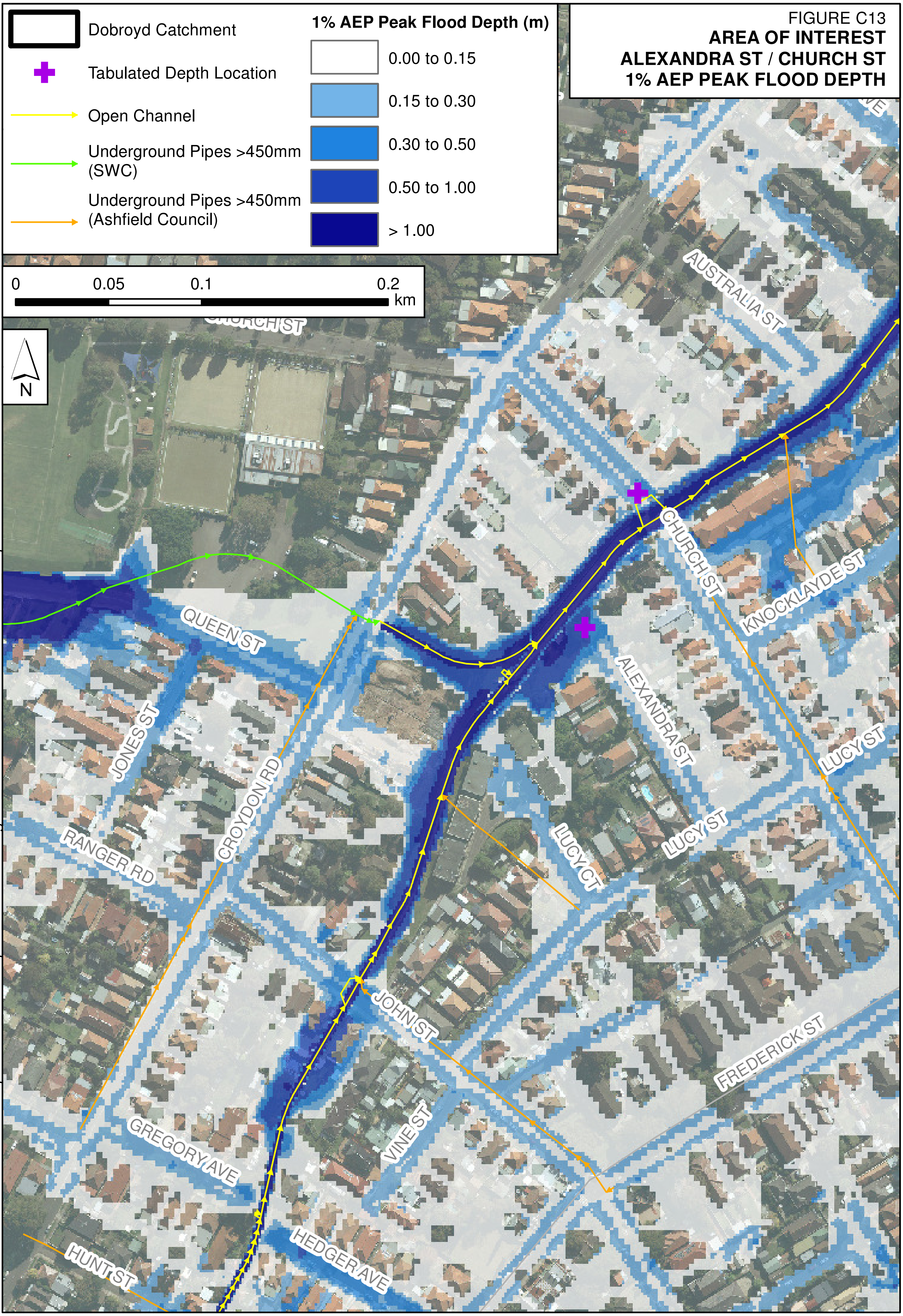

FIGURE C13
AREA OF INTEREST
ALEXANDRA ST / CHURCH ST
1% AEP PEAK FLOOD DEPTH

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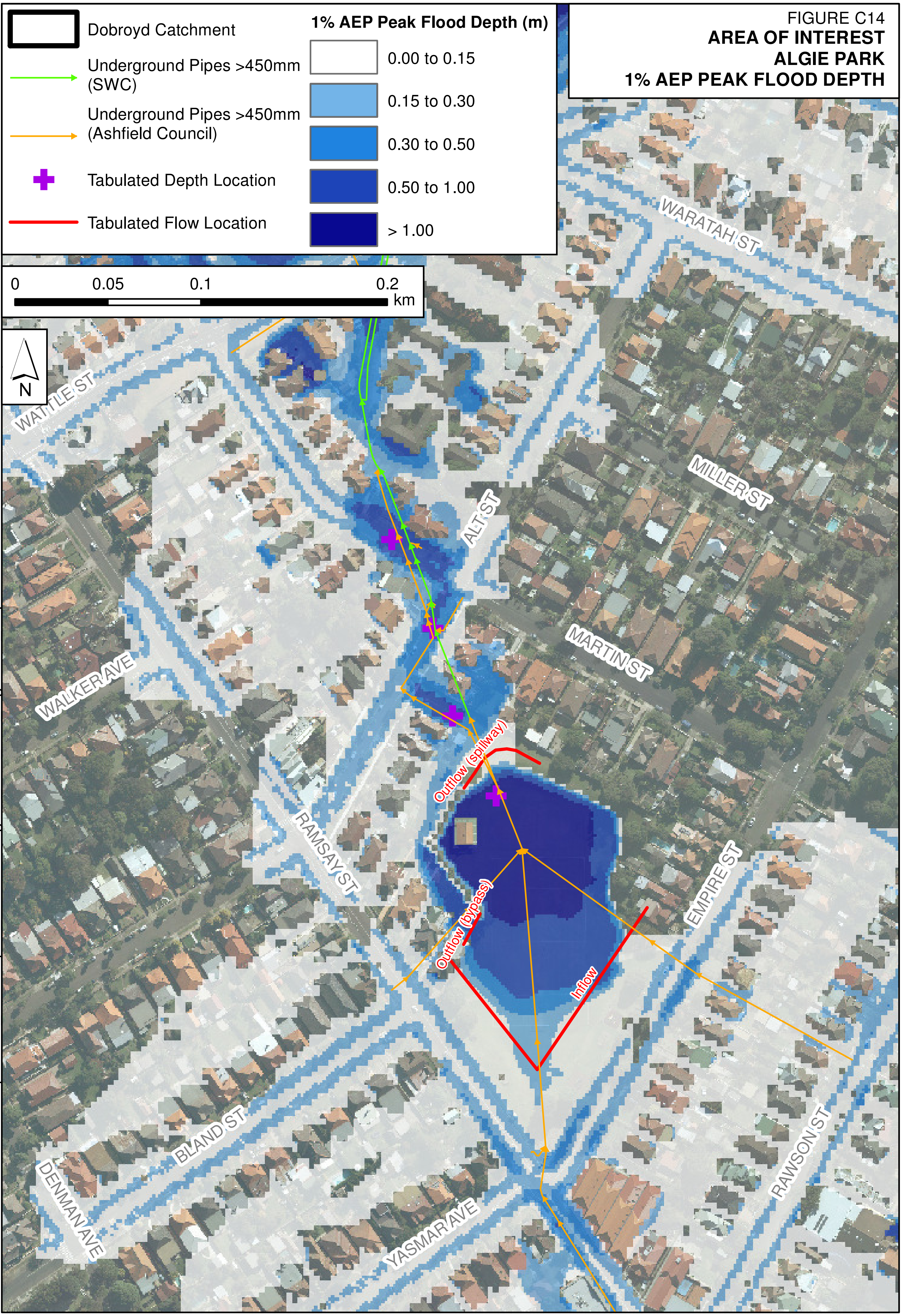

FIGURE C14
AREA OF INTEREST
ALGIE PARK
1% AEP PEAK FLOOD DEPTH

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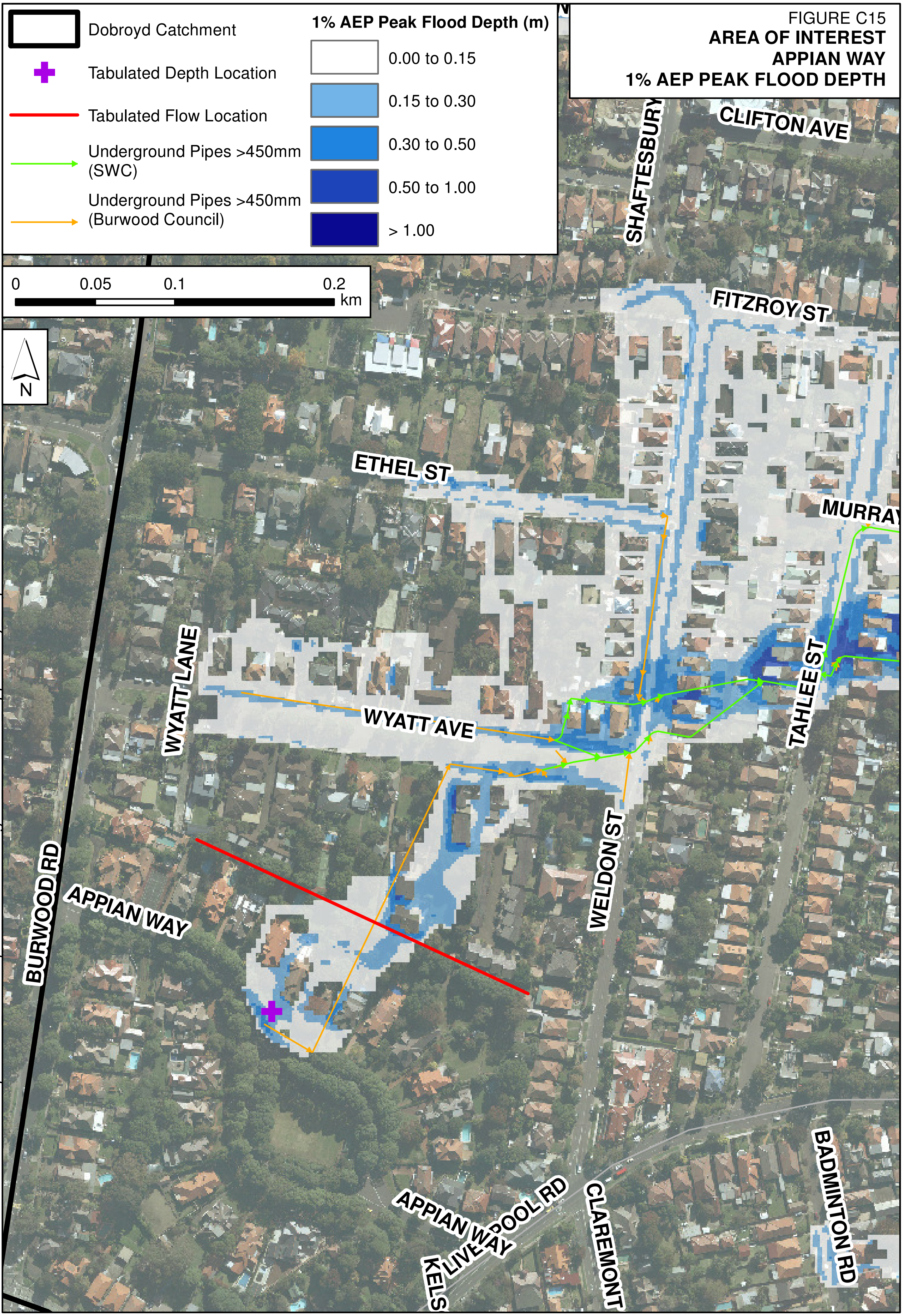

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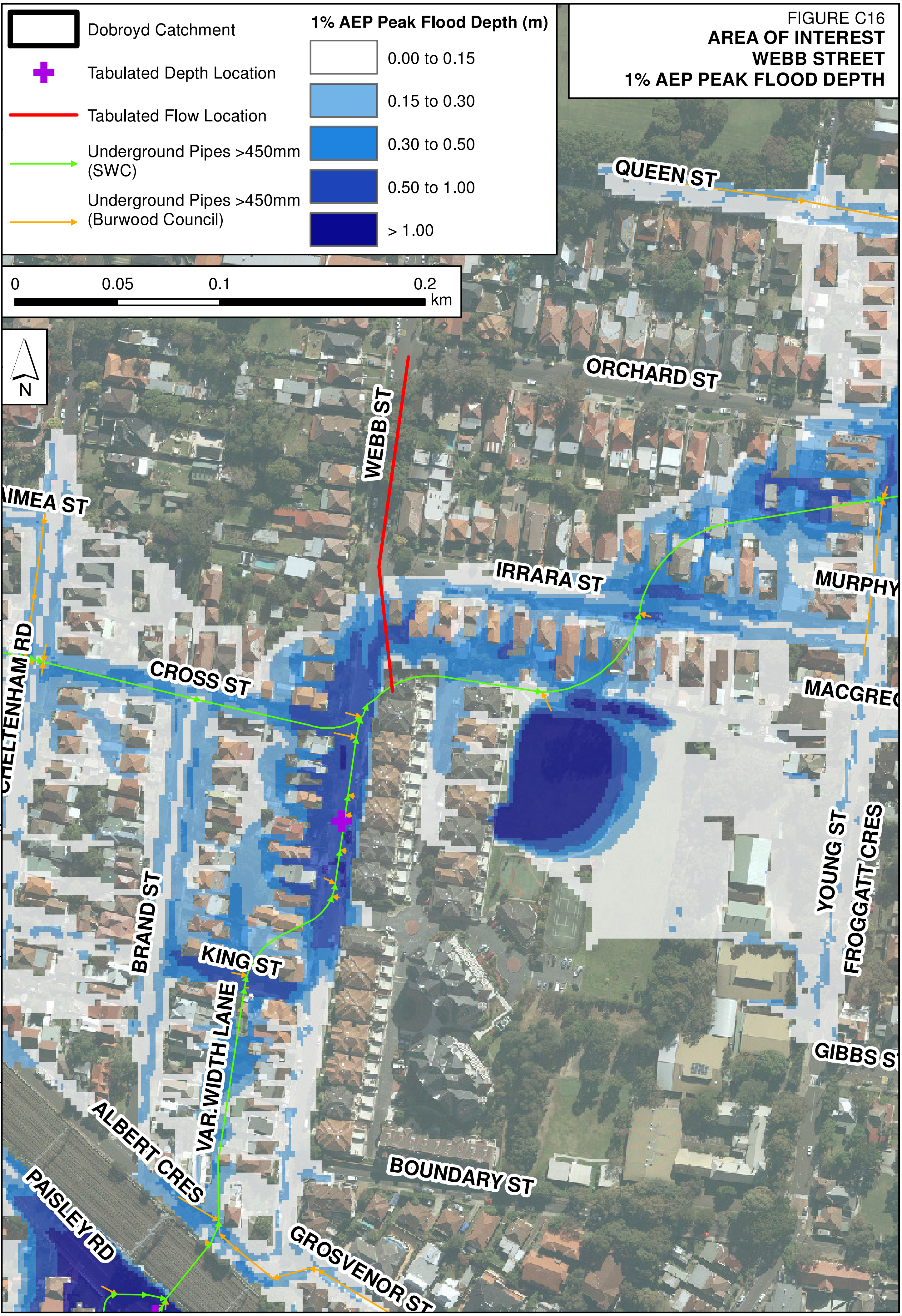

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