POWELLS CREEK AND SALEYARDS CREEK DRAFT FLOOD STUDY

MARCH 2017

Level 2, 160 Clarence Street
Sydney, NSW, 2000

Tel: 9299 2855
Fax: 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

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MARCH, 2017

Project Powells Creek Flood Study	**Project Number** 115010
Client Burwood Council	**Client's Representative** Kundan Pokharel
Authors Richard Dewar Felix Taaffe Erika Taylor	**Prepared by** TO BE SIGNED FOR FINAL REPORT
Date 1 March 2017	**Verified by** TO BE SIGNED FOR FINAL REPORT

Revision	Description	Date
1	Draft Report	March 2016
2	Draft Report	June 2016
3	Draft Report	March 2017

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LIST OF ACRONYMS

AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ARI	Average Recurrence Interval
AR&R	Australian Rainfall and Runoff
ALS	Airborne Laser Scanning sometimes known as LiDAR
BoM	Bureau of Meteorology
CBD	Central Business District
CSIRO	Commonwealth Scientific and Industrial Research Organisation
CFERP	Community Flood Emergency Response Plan
DEM	Digital Elevation Model
DRAINS	Hydrologic computer model developed from ILSAX
EPR	Entire Period of Record of gauge data at Elva Street gauge)
EY	Exceedances per Year
FFA	Flood Frequency Analysis
GEV	Generalised Extreme Value probability distribution
GIS	Geographic Information System
GSDM	Generalised Short Duration Method
HEC-RAS	1D hydraulic computer model
HGL	Hydraulic Grade Line
ILSAX	Hydrologic model - a precursor to DRAINS
IFD	Intensity, Frequency and Duration of Rainfall
IPCC	Intergovernmental Panel on Climate Change
LEP	Local Environmental Plan
LGA	Local Government Area
LiDAR	Light Detection and Radar
LP3	Log Pearson III probability distribution
m	metre
MHL	Manly Hydraulics Laboratory
m^3/s	cubic metres per second (flow measurement)
m/s	metres per second (velocity measurement)
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SEPP	State Environmental Planning Policy
SMC	Strathfield Municipal Council
SWC	Sydney Water Corporation
TIN	Triangular Irregular Network
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software program (hydraulic computer model)
UNSW	University of New South Wales
1D	One dimensional hydraulic computer model
2D	Two dimensional hydraulic computer model

FOREWORD

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

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

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

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

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

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

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

The Powells Creek Flood Study constitutes the first stage of the management process and is based on the recent study for the wider catchment undertaken by Sydney Water Corporation.

EXECUTIVE SUMMARY

BACKGROUND

Powells Creek is a small southern tributary of the Parramatta River and Saleyards Creek is the major tributary of Powells Creek (Figure 1). The total catchment area of Powells Creek to Homebush Bay Drive is 8.1 km^2 and Saleyards Creek to the confluence with Powells Creek is 3.2 km^2.

The Powells Creek catchment is located in Sydney's Inner West region, approximately 12 kilometres west of the CBD. The catchment includes the suburbs (or parts) of Burwood, Concord West, Homebush, Homebush West, North Strathfield, Strathfield and Rookwood (cemetery). Approximately 77% of the catchment is within the Strathfield Municipal Council (SMC) local government area (LGA), 15% is within City of Canada Bay Council, 5% is within Burwood Council LGA and 3% (Rookwood cemetery) within Auburn LGA. Saleyards Creek is predominantly within the SMC LGA apart from Rookwood cemetery.

The Powells Creek catchment drains to Homebush Bay 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 pipe and pit networks are owned by the various councils.

The current study concerns the part of the Burwood LGA in the Powells Creek catchment.

OBJECTIVES

The purpose of this Flood Study is to identify mainstream and overland flow flooding (where there is no defined channel) in order to define the existing flood liability within the catchment. This objective is achieved through the development of a suitable hydrologic and hydraulic modelling platform that can subsequently be used as the basis for a future Floodplain Risk Management Study and Plan for the study area, and to assist 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 subsequent detailed overland flow studies and a 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; 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 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. Archival records also mention several prior large floods including a particularly severe event in 1860. More recently, reports of minor property inundation from overland flow in 2015 and 2016 in the Burwood LGA have been received.

A water level gauge at Elva Street was operated from 1958 to approximately 2010 by the University of New South Wales (UNSW). The records have been digitised up to 1997 and were used for calibration of the modelling system as well as flood frequency analysis.

PAST STUDIES

Initially a review of the available reports and data was undertaken. The previous Powells Creek Flood Study undertaken for SMC in 1998 is the only study covering the entire catchment and providing detailed flood levels. All relevant data from the 1998 Powells Creek Flood Study was obtained and used in the present study and the results compared. The other prior studies used hydrologic models (ILSAX) to determine pipe flows and assess mitigation measures and are of less relevance for the present study.

RAINFALL AND FLOOD HEIGHT DATA

There is a limited amount of rainfall data covering the catchment, particularly pluviometer data which is needed to describe the temporal pattern of historical events. A reasonable amount of historical flood height data is available from SWC records as well as the 1998 Powells Creek Flood Study. As no significant floods have occurred since the completion of the 1998 Flood Study, no further attempt of obtaining historical flood data from the residents was made as part of the present study.

HYDROLOGIC AND HYDRAULIC MODELLING PROCESS

The hydrologic modelling was undertaken using DRAINS and the hydraulic model was undertaken using TUFLOW. These models were verified by comparison to six historical events (3rd, 7th, 10th and 17th February 1990, 18th March 1990 and 2nd January 1996).

The design rainfall events modelled were the 0.5EY, 0.2EY, 10%, 5%, 1% AEP, 0.5% and 0.2% design events and the Probable Maximum Flood (PMF). The temporal patterns for the design events were sourced from Australian Rainfall and Runoff (1987) and the rainfall data was obtained from the Bureau of Meteorology's (BoM) internet-based tool. The PMP estimates were derived according to the BoM guidelines.

FLOOD FREQUENCY ANALYSIS

An extensive flood frequency analysis (FFA) was carried out which examined different rating curves and the use of different data sets. When compared to FFA design flow estimates, those from TUFLOW appear to overestimate flows for more frequent events and underestimate flow in the 2% AEP event or greater.

SENSITIVITY ANALYSIS, BLOCKAGE AND CLIMATE CHANGE

Sensitivity analysis and blockage assessments were undertaken to assess the effects of varying key model parameters. In addition, assessments of the effects of a sea level rise elevating the adopted design water levels in the Parramatta River and an increase in design rainfall intensities were undertaken. Sea level rise made little difference in the upstream developed areas; however, rainfall increases will produce a significant increase in flood levels.

OUTCOMES

The results from this study provide design flood data (levels, depths, velocity, hazard, hydraulic classification) which supersede those derived in the 1998 Powells Creek Flood Study.

Immediately following the next large flood event (10% AEP or greater) water level and rainfall data should be collected and used to verify the hydrologic and hydraulic model calibration.

1. INTRODUCTION

1.1. Background

The Powells Creek catchment (Figure 1) is located on the southern bank of the Parramatta River at Homebush Bay, approximately 12 kilometres west of the Sydney CBD. The main tributary of Powells Creek is Saleyards Creek which enters immediately upstream of Homebush Bay Drive. Downstream of Homebush Bay Drive, Powells Creek is a natural channel surrounded by dense mangrove vegetation on both sides. Upstream Powells and Saleyards Creeks are concrete lined channels with Powells Creek bounded on the east by the City of Canada Bay LGA; largely comprising of residential development with residential, light industry and open space on the western SMC side. Saleyards Creek is bounded on both sides by open space until reaching Underwood Road where it is largely bordered by commercial developments.

The total catchment area of Powells Creek to Homebush Bay Drive is 8.1 km^2 and Saleyards Creek to the confluence with Powells Creek is 3.2 km^2.

The catchment includes the suburbs (or parts) of Burwood, Concord West, Homebush, Homebush West, North Strathfield, Strathfield and Rookwood (cemetery). Approximately 77% of the catchment is within the SMC LGA, 15% is within City of Canada Bay Council, 5% is within Burwood Council LGA and 3% (Rookwood cemetery) within Auburn LGA (herein termed the Councils). Saleyards Creek is predominantly within the SMC LGA apart from Rookwood cemetery.

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 the Councils, with SWC owning the larger "trunk" elements. Amongst the drainage assets is a length of brickwork drain that was one of the first purpose-built stormwater drains in Sydney and constructed in the 1890's. Open channel sections extend from Powells Creek under the railway lines to Elva Street, to just beyond Ismay Avenue on the small tributary, and up Saleyards Creek under Flemington markets to upstream of the railway line.

The present study has been commissioned by Burwood Council to extend upon the previous study commissioned by SWC, to define mainstream and overland flood behaviour in the catchment. This report covers the part of the catchment lying in the Burwood LGA, and results and analysis are virtually the same as those presented in the SWC-commissioned study. Mainstream is generally defined as flooding occurring from open channels, either lined or natural, whereas overland is mainly flooding where there is no defined open channel and drainage is via the pit and pipe system or overland through private and public properties. However, there are exceptions to these definitions. The study area does not include any sections of open channel or mainstream flooding (they are located in the Strathfield LGA), however, results from these downstream areas have been presented for completeness.

1.2. Description of Catchment

The study area's catchment is fully urbanised. Within the Strathfield LGA approximately 79% of the catchment is zoned for residential development, 9% for special purpose, 6% for open space areas (parks and recreation areas) and the remaining 7% for business/commercial and industrial areas. Within the Burwood LGA, approximately 90% is zoned for residential development (mix of Low Density and General) with remaining areas containing mixed use, public recreation and infrastructure.

A land use zone map is provided as Figure 2. Upstream of the Parramatta railway Line both catchments are predominantly occupied by residential development with areas of open space, schools and active recreation. The residential developments are largely detached dwellings constructed prior to 1960 but there are also a number of recent higher density developments. Significant commercial development is located near Strathfield railway station at Strathfield Plaza.

Downstream of the railway line the catchments of both creeks are a mixture of residential, commercial (Flemington Markets) and light industrial developments. There are also significant areas of open space surrounding the lower parts of both creeks. The transport routes, M4 Motorway, Parramatta Road, Homebush Bay Drive and the railway lines have influenced the flow paths in the lower reaches.

Very little information is available in Council's records regarding the existing site drainage for the catchment in general (i.e. are there rubble pits? If so what size? Is the existing roof drainage connected directly to the street drainage?). On-site detention has been introduced by the Councils since the mid-1990s.

Diagram 1 indicates the significant change in alignment of Powells Creek with construction of the concrete lined SWC channel.

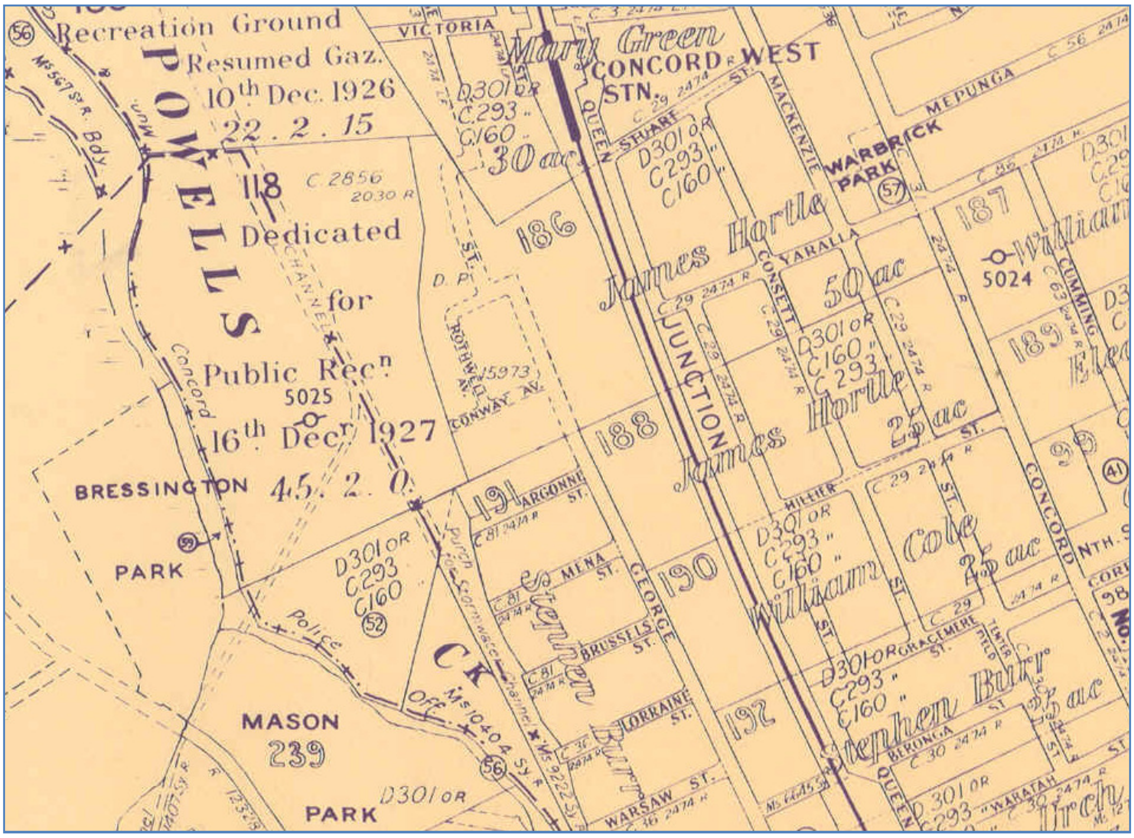


Diagram 1: Cadastral Plan near the time of Construction of the SWC Concrete Channel

Elevations in the upper part of the catchment (Figure 3) reach approximately 55 m AHD near Arthur Street and some reaches are relative steep with 2% to 4% grades. However, the 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 the lined channel width varies from approximately 2 m in the upper areas to 22 m at Homebush Bay Drive.

Construction of buildings and structures over the open lined channel as shown on Figure 4 has significantly reduced the capacity of the natural waterways. As a result flooding has occurred in the past (Figure 5) causing significant tangible and intangible damages.

1.3. Objectives

The primary objective of the Flood Study was to develop a suitably robust hydrologic and hydraulic modelling system to be used to define flood behaviour, peak flood levels and inundation extents within the study area. This system may subsequently be used within a Floodplain Risk Management Study to assess the effectiveness and suitability of flood mitigation works.

The key stages in the flood study process are:

- undertake a comprehensive review of the available flood related data including previous studies, available survey data, historical rainfall and flood level data;
- establish a hydrologic model for the entire Powells Creek catchment to Homebush Bay Drive;

- develop a suitable hydraulic model of Powells Creek and major tributaries within the study area;
- calibration of the hydrologic and hydraulic models to historic flood data;
- define the flood behaviour and produce information on flood levels, velocities and flows for a full range of design flood events under existing conditions;
- assess the sensitivity of blockage and other assumptions on peak flood flows and levels;
- assess the impacts of sea level rise and increase in rainfall and runoff intensities due to climate change; and,
- prepare hydraulic hazard and category mapping.

This report details the results and findings of the above investigations.

1.4. Floodplain Risk Management Process

As described in the 2005 NSW Government's Floodplain Development Manual (Reference 1), the Floodplain Risk Management Process entails four sequential stages:

Stage 1:	*Flood Study*
Stage 2:	*Floodplain Risk Management Study*
Stage 3:	*Floodplain Risk Management Plan*
Stage 4:	*Implementation of the Plan*

The above first three stages were completed with publication of Powells Creek Flood Study (Reference 2) and the Powells Creek Floodplain Risk Management Study and Plan (Reference 3). Several other flood studies have also been undertaken for private developers and these are reviewed in Section 2.2.

This present document provides a review of the past flood studies and updates the design flood analysis to current best practice. A Flood Study is a technical document and is not always easily understood by the general public. A glossary of flood related terms is provided in Appendix A to assist. If more explanation of terms or a better understanding of the approach is required, type "*NSW Government Floodplain Development Manual*" into an internet search engine and you will be directed to the NSW Government web site which provides a copy of this manual (Reference 1) and further explanation.

Australian Rainfall and Runoff (AR&R) have produced a set of draft guidelines for appropriate terminology when referring to the probability of floods. In the past, Annual Exceedance Probability (AEP) has generally been used for those events with greater than 10% probability of occurring in any one year, and Average Recurrence Interval (ARI) used for events more frequent than this. However, the ARI terminology is to be replaced with a new term, EY.

AEP is expressed using percentage probability. It expresses the probability that an event of a certain size or larger will occur in any one year, thus a 1% AEP event has a 1% chance of being equalled or exceeded in any one year. For events smaller than the 10% AEP event however, an annualised exceedance probability can be misleading, especially where strong seasonality is experienced. Consequently, events more frequent than the 10% AEP event are expressed as

Exceedances per Year (EY). Statistically a 0.5 EY event is not the same as a 50% AEP event, and likewise an event with a 20% AEP is not the same as a 0.2 EY event. For example an event of 0.5 EY is an event which would, on average, occur every two years. A 2 EY event is equivalent to a design event with a 6 month average recurrence interval where there is no seasonality, or an event that is likely to occur twice in one year.

While AEP has long been used for larger events, the use of EY is to replace the use of ARI, which has previously been used in smaller magnitude events. The use of ARI, the Average Recurrence Interval, which indicates the long term average number of years between events, is now discouraged. It can incorrectly lead people to believe that because a 100-year ARI (1% AEP) event occurred last year it will not happen for another 99 years. For example there are several instances of 1% AEP events occurring within a short period, for example the 1949 and 1950 events at Kempsey.

The Probable Maximum Flood (PMF) is a term used in describing the largest possible flood and is related to the PMP, the Probable Maximum Precipitation.

This report has adopted the approach of the AR&R draft terminology guidelines and uses % AEP for all events greater than the 10% AEP and EY for all events smaller and more frequent than this.

All levels in this report are in metres to Australian Height Datum (AHD). Mean sea level is approximately 0 mAHD and an approximate tidal range in Homebush Bay is +0.6 mAHD to -0.4 mAHD.

1.5. Accuracy of Model Results

The accuracy of all model results provided in this report is dependent on the input data sets and the ability of the modelling approach to replicate recorded historical flood data. As modelling approaches improve over time and additional flood data becomes available from future flood events the accuracy of the results will improve.

A key input data set is the topographic information provided by SWC and the Councils for use in this study. The topographic information was derived from Airborne Laser Scanning (ALS) with an estimated accuracy of ± 0.15m in cleared areas, such as car parks or on roads. In locations with more complex terrain, such as vegetated areas, the accuracy is likely to be much lower and could vary significantly, by up to ± 1m. It is cost prohibitive to obtain detailed field survey throughout the entire study area and the ALS is assumed to be correct. However due to these potential accuracy limitations, some of the floodway extents, depth estimates and design flood levels may change if more accurate field survey is obtained. It is estimated that an order of accuracy of the design flood levels is ± 0.3 m where quality historical calibration data are available nearby and up to ± 0.5 m where no such data are available.

The results from the present study incorporate best practice in design flood estimation at this time but it is acknowledged that changes in approach in the future will cause changes to design flood levels. A good example of this is the collection of rainfall data which forms the basis of design flood estimation. As more rainfall data are collected and analysed (and particularly from continuously read gauges termed pluviometers) the BoM will provide new estimates of design rainfalls and design temporal patterns over NSW. An updated version of the 1987 edition of AR&R - Reference 4 will also introduce new approaches and guidelines which may change design flood levels.

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 or Parramatta Rivers there are generally stream height and historical records dating back to the early 1900's, or in some cases even further. However, in most small urban catchments there are no stream gauges or official historical records available.

The Powells Creek catchment is unique in Sydney because a stream gauge has been operated by the UNSW at Elva Street for a long period (50 years). The records from this gauge have been used for many technical papers and university undergraduate and graduate theses.

An overview of historical of flooding is also available from an examination of the Councils and SMC records, previous reports, internet search of newspapers, rainfall records and local knowledge.

2.2. Previous Studies

A number of previous studies (Table 1) have been undertaken as described in Reference 2. Numbers 1 to 6 used ILSAX hydrologic models to assess solutions to drainage problems with the majority distributing a questionnaire to the residents in order to obtain information about the drainage problems. Only numbers 7 to 11 determined design flood levels. No. 1 provides a summary of the more recent studies. No studies have been undertaken specifically on the study area (Burwood LGA in Powells Creek catchment).

Table 1: Previous Studies Listed in Reference 2

Title	Consultant	Branches	Date	Comment	No.
Strathfield Local Flooding Issues	Kinhill Engineers	Wentworth Rd, Strathfield Ck, Albyn Rd	March 1997	Expanded upon References 2 and 3. Undertook HGL.	1
Redmyre Road/Florence Street Catchment Study	Giammarco	Albyn Rd	November 1993	Undertook HGL.	2
Rochester Street Catchment Drainage Investigation	Bewsher Consulting	Strathfield Ck	December 1990	Undertook HGL.	3
Stormwater Drainage Upgrading Programme - Rochester Street Catchment - Feasibility Study and Design Report	Taylor, Thomson, Whitting	Strathfield Ck	1992	Expanded on Ref. 3. Undertook HGL.	4
Rochester Street Drainage Investigation Report	Rankine and Hill	Strathfield Ck	May 1985	Examined upgrading of pipe system.	5
Arthur Street Catchment Study	Bewsher Consulting	Saleyards Ck	July 1996	Only upstream of the railway line.	6
Saleyards Creek at Park Road, Flemington	Bewsher Consulting	Saleyards Ck	October 1996	Determined design flood levels.	7
12-14 Wentworth Road, Homebush	Bewsher Consulting	Saleyards Ck	February 1995	Determined design flood levels.	8

Title	Consultant	Branches	Date	Comment	No.
32-36 Burlington Road, Homebush	B Lysenko	Strathfield Ck	February 1994	Determined design flood levels.	9
Lower Parramatta River Flood Study	Willing & Partners	Powells Ck to approx.Pomeroy St	February 1986	Determined design flood levels.	10
Powells Creek at Underwood Street Site Flood Study	Tierney & Partners	Powells Ck at Pomeroy St	November 1993	Determined design flood levels.	11

However, the references listed in Table 1 are of little value in the current study as they provide little historical data and the results cannot be easily compared. The 1998 Powells Creek Flood Study (Reference 2), however, is a comparable study to the current one and extensive use has been made of the data contained and results.

2.3. 1998 Powells Creek Flood Study (Reference 2)

The 1998 Powells Creek Flood Study was undertaken under the NSW Government Floodplain Management Program and used best practice techniques available at the time. A field survey was undertaken to provide approximately 100 cross sections of the creek channel as well as to collect historical flood height data. Some of the cross section data have been used in the current study and the historical flood height data is provided in Section 2.11.

The study area for determination of design flood levels was taken as:

Powells Creek:

- open channel from Homebush Bay Drive to Elva Street;
- Wentworth Road branch from Powells Creek to the M4 overpass;
- Strathfield Creek branch from Powells Creek to Newton Road;
- Albyn Road branch from Powells Creek to Alviston Street, including the sub-branch from Alviston Street to Victoria Street (Florence Street sub-branch) and from Alviston Street to Llandilo Avenue (Llandilo Avenue sub-branch).

Saleyards Creek:

- open channel from Homebush Bay Drive to Hampstead Road, including the sub-surface section to Mitchell Road;
- the Edgar Street sub-branch from Airey Park to Edgar Street.

A comprehensive data search was undertaken including:

- a review of previous studies;
- interviews with local residents;
- discussions with Council Officers;
- contact with SWC, the then Roads & Traffic Authority, the then State Rail Authority, the then Department of Land & Water Conservation and the UNSW;
- review of aerial photographs;
- provision of a questionnaire and review of all previous questionnaires;
- obtaining height and rainfall data from the stream and rainfall gauges operated by the UNSW and SWC.

2.3.1. ILSAX Model

An ILSAX hydrologic model of the entire Powells and Saleyards Creeks catchment was constructed using ILSAX files from some of the studies listed in Table 1. Unfortunately there is no record of the 1130 sub catchment delineation. Inflows from ILSAX were then input into the 1D HEC-RAS hydraulic model which determined flood levels and velocities. Flood extents were not defined, however this has subsequently been undertaken using the peak levels and ALS in Reference 5.

The ILSAX model was calibrated to the events of 3rd February, 7th February, 10th February, 17th February and 18th March 1990 using rainfall from two pluviometers at St Sabina College and at the Elva Street gauge. Calibration to the Elva Street gauge for the January 1996 event could not be undertaken as the gauge malfunctioned. The results are provided in Table 2 and adopted the St Sabina pluviometer as being representative of the catchment rather than the Elva Street gauge, except for the 18th March 1990 event.

Table 2: ILSAX Calibration Results from Reference 2

Event	Peak Flow (m3/s)			Volume (ML)			Runoff Co-efficient		Rainfall (mm)
	Actual	Model	% Diff	Actual	Model	% Diff	Actual	Model	
3 February 1990	15.5	15.6	<1%	205	196	-4%	0.76	0.73	110
7 February 1990	15.6	16.8	+8%	85	190	+123%	0.34	0.76	102
10 February 1990	20.9	20.8	<1%	94	110	+17%	0.69	0.80	56
17 February 1990	11.8	12.1	+2%	30	52	+73%	0.38	0.67	32
18 March 1990 St Sabina pluvi	23.3	20.2	-13%	70	91	+30%	0.58	0.76	49
18 March 1990 Elva St pluvi	23.3	24.7	+6%	70	105	+50%	0.52	0.78	55

The main features of the calibration were stated as:

- *there is a good match to the peak flows for all the February 1990 events. For 18th March 1990 a flow midway between the results from the two pluviographs would provide a good match,*
- *the timing and rate of rise of the modelled hydrographs is generally good. The exceptions are 18th March 1990 and 7th February 1990 (timing of streamflow gauge is incorrect),*
- *ILSAX provides a poor match to the volume of runoff. For the majority of events (the exception is 3rd February 1990) ILSAX overestimates the volume by up to 123%. It could be that ILSAX does not accurately represent the losses during the recession limb of the hydrograph. The poor match to the volume of runoff is of less relevance in this type of study than the match to the peak flow,*
- *the results were obtained with identical rainfall loss parameters for each event. A slightly better match may be achieved by varying these parameters but this would make it difficult to decide upon those to be adopted for design,*

- *the variation in actual runoff co-efficient (0.76 to 0.34) is difficult to explain. There are a number of possible reasons including:*
    - *malfunctions in the instrumentation (rainfall and streamflow),*
    - *the recorded rainfall at the pluviometer does not reflect the catchment rainfall. Records show that the rainfall can vary significantly across a short distance (such as between the two UNSW pluviometers),*
    - *the actual losses over the catchment can vary significantly between events.*

Overall the calibration was considered satisfactory and the model appropriate for use in design analysis.

2.3.2. HEC-RAS Model

Approximately 160 cross sections were included in the HEC-RAS model with the majority based on field survey and the remainder interpolated (generally these were required to define upstream and downstream of a structure). The following tailwater levels in Homebush Bay were adopted:

1% AEP	1.40 mAHD;
2% AEP	1.35 mAHD;
5% AEP	1.30 mAHD;
10% AEP	1.25 mAHD;
0.2 EY	1.20 mAHD;
0.5 EY	1.15 mAHD.

Detailed investigation of the peak historical level data revealed a number of problems:

- the majority of the historical recorded levels were for the two most recent events (1990 and 1996) but it was concluded that is unlikely that these were the largest floods. A summary of the historical data shows:
    - January 1996 = 39 levels (rainfall data not available),
    - February 1990 = 21 levels (assumed to be 10th February 1990),
    - 1992 = 2 levels (rainfall data suggested this was only a minor event),
    - 1989 = 2 levels (rainfall data suggested this was only a minor event),
    - others = 9 levels (data unsuitable for calibration).
- as the depth of inundation was generally less than 0.4 m (the greatest depth was 0.8 m) a recorded level may reflect a local "low spot" or "ponding area" rather than being indicative of the level of the main flow,
- local structures (buildings, fences, gates, cars, drains blocked) are likely to have a significant effect upon the recorded levels. This effect may vary between floods (e.g. new fence, or gate open/closed),
- in many places there is a steep local gradient across a property which was not always represented by the survey data. This meant that it was difficult to match to data points not taken at cross-sections. The exact location of the recorded level within the property was also not always known.

The intention was to calibrate HEC-RAS to the January 1996 and February 1990 events but as the Elva Street gauge malfunctioned in January 1996 and rainfall data were not readily available calibration could only be undertaken for the February 1990 events.

The results of the calibration are shown in Table 3.

Table 3: HEC-RAS Calibration Results from Reference 2 for 10th February 1990

Location	Recorded Depth of Flow (m)	Recorded Level (mAHD)	Model Level (mAHD)	Model minus Recorded Level (m)
STRATHFIELD CREEK BRANCH:				
No. 56 Ismay Avenue	0.2	3.8	4.0	0.2
No. 41 Ismay Avenue	0.1	3.7	4.2	0.5
No. 51 Ismay Avenue	0.3	4.2	4.2	0.0
No. 55 Ismay Avenue	0.4	4.3	4.2	-0.1
No. 82 Underwood Road	0.5	5.0	4.9	-0.1
No. 12 Loftus Crescent	0.2	7.9	7.7	-0.2
No. 29 Burlington Road	not recorded	9.2	9.2	0.0
No. 38-46 Burlington Road	0.5	9.7	9.5	-0.2
No. 89 Rochester Street	0.1	12.8	12.7	-0.1
No. 28 Broughton Street	0.2	12.9	12.7	-0.2
No. 109 Rochester Street	0.4	14.3	14.3	0.0
No. 53 Beresford Road	0.1	15.3	15.4	0.1
No. 100 Beresford Road	0.1	15.9	15.9	0.0
No. 102 Beresford Road	0.1	16.4	16.4	0.0
No. 104 Beresford Road	0.6	17.0	16.7	-0.3
No. 108 Beresford Road	0.3	17.5	17.2	-0.3
No. 110 Beresford Road	0.4	17.5	17.2	-0.3
No. 137 Albert Street	not recorded	19.0	19.3	0.3
No. 137 Albert Street	not recorded	19.2	19.3	0.1
No. 141 Albert Street	0.3	19.5	19.3	-0.2
LLANDILO AVENUE BRANCH:				
No. 21 Llandilo Avenue d/s	0.8	28.8	28.9	0.1
No. 21 Llandilo Avenue u/s	0.1	29.9	30.6	0.7
SALEYARDS CREEK:				
No. 79 The Crescent	0.3	8.2	8.1	-0.1
No. 6 Kessell Avenue	not recorded	8.4	8.1	-0.3
POWELLS CREEK:				
No. 34 Ismay Avenue	0.4	2.6	2.4	-0.2
Elva Street Gauge	1.8	7.0	7.2	0.2

The main features of the calibration were stated as:

- *a reasonable match to all flood levels was obtained,*
- *the 0.6 m difference in level between adjacent properties at 102 and 104 Beresford Road could not be replicated. It is possible that there is an error with the records or the levels do not reflect the "mainstream" flow,*

• *the 1.1 m difference in level within 21 Llandillo Avenue could not be replicated.*

Overall the calibration was considered satisfactory and the model appropriate for use in design analysis. The report suggested that further calibration of the hydraulic model should be undertaken as more data become available. However, since 1998 there have been no significant floods suitable for model calibration.

The 2 hour duration was adopted as the critical storm duration for design events. Design flood results were provided in various formats and a comparison between runoff routing and flood frequency approaches (using the Elva Street gauge data) is shown in Table 4. The flood frequency analysis was undertaken by the University of New South Wales using the Flike program and fitting to a log Pearson III distribution.

Table 4: Comparison of Flood Frequency Analysis and Runoff Routing from Reference 2

AEP (%)	Flood Frequency		Runoff Routing (2h Duration)	
	Level (mAHD)	Flow (m^3/s) *	Level (mAHD)	Flow (m^3/s) *
20	7.2	23.8 (50%)	7.4	26.1 (55%)
10	7.5	29.3 (62%)	7.6	29.8 (63%)
5	7.8	34.6 (73%)	8.1	35.3 (75%)
2	#	41.7 (88%)	8.3	41.8 (89%)
1	#	47.2	8.9	47.2

Notes: * flow as a percentage of the 1% AEP event shown in brackets.
Levels for the flood frequency analysis are not provided for events greater than a 5% event. For such events the flow is above the coping of the channel and there are significant backwater influences from the bridges downstream. The extension of the UNSW rating curve used in their flood frequency analysis does not appear to reflect the backwater influence.

The following are some general comments regarding the results:

• *high velocities in the lined channel make it difficult to determine the true velocity and therefore the flow,*
• *the gauging station is well sited to reflect in channel flows but less so for overland flows, the majority of which may enter downstream of the gauge,*
• *the properties of the channel (area, Mannings "n" value, wetted perimeter) can be precisely measured but values above the channel are subject to considerable variation,*
• *ILSAX does not explicitly account for the considerable floodplain storage which occurs within most road reserves and within private property. The only exceptions to this are at Leicester Avenue and Airey Park where detention basins were explicitly included in the model.*

It was concluded that the differences between the design flood levels obtained from flood frequency and runoff routing could only be resolved once high flow calibration data are obtained. These data are difficult to obtain due to the rapid rise (1.8 m in 30 minutes) and fall of the water level. Flood levels from the runoff routing analysis have been adopted for design as the HEC-RAS model should provide a more accurate definition of the channel hydraulics at high flows.

2.3.3. Accuracy of the Design Flood Data

The study concluded that accuracy of the design flood data depended upon a number of factors including:

- **quality of the survey data.** How well do these data represent the floodplain? In an urban catchment the flow path can change dramatically over a short distance (fences, buildings, trees). In this study sections have been located to be representative of the typical flow path in the region.
- **downstream boundary conditions.** Changing the downstream boundary will affect flood levels upstream. This issue is not significant in this study as the main areas of interest are not affected by the downstream boundary.
- **accuracy of design rainfall data.** As the most up to date rainfall data have been used in this study this issue is unlikely to be significant. They may change as a result of climate change.
- **ability of the models to accurately represent the channel hydraulics.** This is likely to be a significant factor.
- **quantity and quality of available historical data.** The calibration of ILSAX to the flow data from the stream gauge provides a high degree of confidence in the results from the hydrological model at the gauge. Calibration of the HEC-RAS model is satisfactory but can be significantly improved if peak height data from future events can be replicated.

The main factors affecting the accuracy of the design data were considered to be the ability of the models to simulate the channel hydraulics and the quantity and quality of the historical data. Based upon the above considerations the accuracy of the design flood levels were considered to be ±0.4 m. This could be improved if further calibration of the models to future flood events was undertaken.

2.4. Comparison of Results with Previous Studies

A comparison of design peak flows from Reference 2 with other studies was shown in Table 5 and Table 6.

Table 5: Comparison of Design Peak Flows (m^3/s) from Reference 2

Location/Reference and ILSAX Branch/Reach (N/P = data not provided)	1% AEP		2% AEP		5% AEP	
	Ref	Ref 2	Ref	Ref 2	Ref	Ref 2
Strathfield Creek Branch: Railway Line (R/49T) (Ref. 3)	38	32	N/P	29	31	26
Saleyards Creek: Park Road (0/25Q) (Ref. 7)	76	47	68	43	61	38
Saleyards Creek: Wentworth Road (0/27Q) (Ref. 8)	76	55	N/P	50	N/P	44
Powells Creek: Homebush Bay (A/41Q) (Ref. 10)	140	182	120	165	105	148
Powells Creek: Pomeroy Street (A/38S) (Ref. 11)	77	97	N/P	90	59	82
Powells Creek: Confluence with Saleyards Creek (A/40S) (Ref. 11)	82	106	N/P	98	67	89
Powells Creek: d/s of conf. with Saleyards Creek (A/41Q) (Ref. 11)	139	182	N/P	165	101	148

Table 6: Comparison of Design Peak Levels (mAHD) from Reference 2

Location/Reference and HEC-RAS River Station Number (N/P = data not provided)	Design Events (AEP)					
	1%		2%		5%	
	Ref	Ref 2	Ref	Ref 2	Ref	Ref 2
Saleyards Creek: Park Road * (49) (Ref. 7)	4.60	4.31	4.50	4.12	4.10	3.91
Saleyards Creek: Wentworth Road (46) (Ref. 8)	2.90	2.59	N/P	2.58	N/P	2.57
Strathfield Creek Branch: 32-36 Burlington Road (84) (Ref. 9)	9.91	9.80	N/P	9.74	N/P	9.68
Powells Creek: Saleyards Creek confluence (1.5) (Ref. 10)	2.50	1.75	2.23	1.62	2.15	1.51
Powells Creek: Pomeroy Street (4.5) (Ref. 10)	3.00	3.15	2.85	3.11	2.75	3.05
Powells Creek: Lemnos Street (approx) (6) (Ref. 10)	3.25	3.11	3.14	3.07	3.03	3.03
Powells Creek: d/s of Pomeroy Street (4) (Ref. 11)	2.11	2.18	N/P	2.06	1.84	1.97
Powells Creek: u/s of Pomeroy Street(4.5) (Ref. 11)	2.80	3.15	N/P	3.11	N/P	3.05

Sensitivity analyses to changes in design rainfall intensities and parameters in ILSAX were also undertaken. Blockage was considered and was summarised in the following statement. *In the absence of any conclusive data on this issue and the fact that all previous studies assumed nil blockage, nil blockage was adopted for this study. As only a small percentage of flow is within the pipe system in a large flood, varying this parameter will have little impact on design flood levels.*

2.5. Data Sources

Data utilised in the present study has been sourced from a variety of organisations. Table 7 lists the type of data sourced and from where it has been extracted.

Table 7: Data Sources

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

2.6. Topographic Data

Airborne Laser Scanning (ALS) or Light Detection and Ranging (LiDAR) survey of the catchment and its immediate surroundings was provided for the study by SWC and SMC. 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.

2.7. Structure Survey

All bridges and structures within the open channel extent of the study area were inspected in May 2014. Survey data collected as part of Reference 2 were used to define the structures. Photographs on Figure 4 provide a descriptive overview of the key characteristics of the open channel system.

2.8. Rainfall Data

2.8.1. 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:

- 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 4 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 12 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. However, 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 Powells Creek 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.

2.8.2. Rainfall Stations

There are a number of daily read rainfall stations within the catchment and surrounding area. Data were not collected from these stations as more suitable data were available from six pluviometers (Table 8). The two UNSW pluviometers have operated since approximately 1977 but the dates shown in Table 8 are the periods for which digital data are available. No correction has been made in the digital records for the UNSW gauges to account for errors in the clock speed. Thus the time of the recorded rainfall can be out by several hours. This has not been corrected for in this report; however, Reference 6 provides an approach that can be used.

Table 8: Pluviometers

Gauge No.	Operator	Operating Period	Location
566005	UNSW	Mar 1981 to Feb 1996 (period when digital records available)	St Sabina College (Russell St, The Boulevarde)
566004	UNSW	Dec 1980 to June 1993 (period when digital records available)	Stream gauge at Elva St/Beresford Rd
566022	SWC	May 1969 to August 1983, July 1990 to Present	Homebush Bowling Club (Pomeroy St)
566020	SWC	Oct 1958 to Present	Enfield (Belfield Bowling Club - Margaret St)
566036	SWC	February 1970 to Present	Potts Hill Reservoir
566064	SWC	June 1988 to Present	Concord (Western Suburbs Club).

2.8.3. Analysis of Pluviometer Data

Rainfall data were collected from some of the available pluviometers for the significant recent flood events with the peak bursts provided in Table 9 and Figure 9. An estimate of the rainfall frequency for each event can be obtained from comparison with the design rainfalls (Table 10).

Table 9: Historical Rainfall - Maximum Rainfall Depths (mm)

	Duration						
	5 or 6 min	10 min	20 min	30 min	60 min	90 min	120 min
			2nd January 1996:				
Homebush	15	23	36	44	52	54	58
Enfield	17	25	45	57	81	83	88
Potts Hill	11	17	31	42	49	52	54
Concord	7	11	21	30	46	49	52
Elva Street	Instrument Failed						
St Sabina	11	22	37	50	64	n/a	71
			8th February 1992:				
Homebush	Instrument Failed						
Enfield	4	6	10	13	22	28	33
Elva Street	Instrument Failed						
St Sabina	2	5	6	11	16	n/a	n/a
			11th March 1991:				
Homebush	No Significant Rain						
Enfield	13	19	34	37	-	-	-
Potts Hill	11	18	33	35	-	-	-
Concord	10	16	24	24	-	-	-
Elva Street	Instrument Failed						
St Sabina	Instrument Failed						
			18th March 1990:				
Elva Street	20	34	41	44	45	47	50
St Sabina	8	23	26	31	36	43	46
			10th February 1990:				
Homebush	Gauge Not in Operation						
Enfield	11	15	23	26	40	45	50
Potts Hill	12	19	31	36	44	48	52
Concord	7	11	17	25	31	33	38
Elva Street	9	13	22	28	39	n/a	50
St Sabina	6	11	21	31	42	n/a	52
			4-6th August 1986:				
Homebush	Gauge Not in Operation						
Enfield	12	17	27	36	50	59	64
Potts Hill	11	16	27	37	52	60	64
Concord	Gauge Not in Operation						
Elva Street	10	13	17	21			
St Sabina	Very Little Rain						

Note: Data for January 1989 are not shown as the Enfield pluviometer record indicated no significant rainfall events.
Data from other pluviometers may be available but were not collected.

2.9. Design Rainfall

Design rainfall intensities were based on procedures in AR&R 1987 (Reference 4). Design rainfall intensities at the centre of the catchment are provided in Table 10.

Table 10: Design Rainfall Intensities at the Catchment Centroid (mm/hr)

	Duration						
Event	5 min	10 min	20 min	30 min	60 min	90 min	120 min
0.2 EY	144	111	81	66	45.4	35.6	29.9
10% AEP	161	124	91	74	51	40.2	33.8
5% AEP	184	142	104	85	59	46.3	39
2% AEP	213	165	121	99	69	54	45.7
1% AEP	236	182	134	110	76	60	51
0.5% AEP	258	200	148	121	84	66	56
0.2% AEP	288	224	165	135	94	75	63
PMP				440	326	248	208

Probable Maximum Precipitation (PMP) design rainfall depths were calculated using the 2003 BoM Generalised Short Duration Method (Reference 7) for durations up to 6 hours.

2.10. Stream Gauges

2.10.1. UNSW (Elva Street Gauge)

Flood levels have been recorded continuously from September 1958 at the Elva Street gauge (Photo 1) until 2010. Apart from this gauge there are no other long term flood records for the catchment. SWC operated a gauge on Powells Creek (under the M4) but records are only available from October 1995.

Photo 1: Powells Creek gauge at Elva Street

At the time of completion of the 1998 Powells Creek Flood Study (Reference 2) only a limited amount of water level and rainfall data were available from the UNSW as only parts of the historical records were digitised or quality checked.

Subsequently the entire water level and pluviometer record (both at St Sabina and at Elva Street) have been digitised and a rating table adopted to assign flows to the recorded levels.

However there are many gaps in the digital record and this means that the record is only complete to November 1997. The digital record has also not been corrected for timing errors. This timing error correction has not been undertaken for this study.

A summary of the water level data is provided on Figure 6 and below indicates the number of days where the water level has exceeded a threshold (1958 to November 1997):

- >3m - 1 day;
- >2.5m - 3 days;
- >2m - 6 days;
- >1.5m - 31 days;
- >1m - 116 days.

The coping of the channel is approximately 3m above the invert and thus only one event (February 1959) has exceeded the capacity of the channel in approximately 55 years of record (1958 to 2014). A review of Figure 6 indicates that since 1974 (40 years) no event has exceeded 2m on the gauge but 5 events did in the period from 1958 to 1974. Unfortunately this means that calibration can only be undertaken on events smaller than 2m gauge height as the two UNSW pluviometers were not in operation until 1980.

Reference 2 included Table 11 which listed the largest events recorded on the UNSW gauge above 2.0 m. These height data were obtained from inspection of the gauge charts or estimated from debris (Reference 6). The corresponding digital records are shown alongside in Table 11.

Table 11: UNSW Gauge at Elva Street - Major Floods (> 2.0 m) taken from Reference 2

Rank	Year	Date	Gauge Height (m)	RL(mAHD)	Gauge Height (m) from Digital Record
1	1961	18 Nov	4.18 *	9.43	**No Record**
2	1964	10 Jun	3.52 *	8.77	**1.8**
3	1959	18 Feb	3.29 *	8.54	**3.26**
4	1972	29 Oct	3.20	8.45	**0.9**
5	1970	9 Dec	3.09	8.34	**Gauge failed**
6	1963	13 Dec	2.40	7.65	**2.47**
7	1973	9 Apr	2.35	7.60	**0.7**
8	1974	25 May	2.34	7.59	**2.23**

* estimated from debris.
Gauge zero is RL 5.25 mAHD.

A limited number of gaugings (height v velocity measurements) have been undertaken enabling the construction of a rating curve (height versus flow). Whilst in theory this approach appears very simple it becomes complex for a number of reasons, including:

- the events occur within a few hours and thus it was very hard for the UNSW staff to get to the gauge whilst a flood was in progress;
- the above means that there are several low flow gaugings but very few high flow gaugings which are more relevant for use in a flood study;

- a gauging was taken by the UNSW at high flows which produced velocities above the rating of the instrument (say above 5 m/s). Thus even this gauging could not confidently determine the peak flow.

Rating curves from various sources are provided on Figure 7.

2.10.2. Sydney Water Gauge

This gauge, which is located on Powells Creek under the M4, has only recorded one significant flood (January 1996) since it was installed in 1995. The gauge zero is RL 2.15 mAHD and the January 1996 flood peaked at 2.04 m (4.19 mAHD) at 1405 hours. Three streamflow gaugings have been undertaken. All gaugings are below 0.1 m gauge height (flow <2 m^3/s). Extrapolation of the rating curve based on these data is not appropriate and as a result flow data from this gauge have not been used for calibration of the hydrologic model.

2.11. Flood Levels from Debris or Other Marks

2.11.1. Resident Interviews

As part of the 1998 Powells Creek Flood Study (Reference 2) and earlier studies (refer Table 1) questionnaires were distributed to local residents in order to collect information about past flood events. Prior to the 1998 Powells Creek Flood Study the responses were generally concerned with drainage issues (blocked pits, minor overland flow) and not with identifying historical flood levels. The only exception to this was at Airey Park (Saleyards Creek) for the January 1996 event.

Data obtained from residents should be used with caution for a number of reasons, including:

- residents may have only been in the study area for a short period;
- residents may have "missed" a flood whilst they were away;
- the more recent events are remembered more clearly than (say) a larger event several years ago;
- some events noted by residents may be as a result of a blocked drain or other local factors and are more typically referred to as local drainage problems rather than flood related;
- residents can easily forget the date of a flood or become confused about the extent and nature of the problem. Experience has shown that water entering a house may have resulted from a leak in the gutter or a local drainage problem in the yard rather than overbank flow from the main creek.

Table 12 provides the most widely remembered events (obtained from the results of the 1998 Powells Creek Flood Study (Reference 2) and previous questionnaire surveys).

Table 12: Significant Floods Obtained from 1998 Flood Study Questionnaire

Approximate Date	Comment
? 1930's	Infrequently mentioned.
1943	Infrequently mentioned.

18 February 1959	Infrequently mentioned.
? 1960's	Infrequently mentioned.
November 1961	Infrequently mentioned.
? 1964	Infrequently mentioned.
? 1973	Infrequently mentioned.
August 1986	Appears to be the largest event in the last 30 years
March/April and July 1988	Infrequently mentioned.
January 1989	Widely remembered.
February 1990	Widely remembered, larger than 1996 in Saleyards Creek
March 1990	Infrequently mentioned.
April 1990	Infrequently mentioned.
March 1991	Widely remembered.
2 December 1992	Infrequently mentioned.
February 1995	Infrequently mentioned.
October 1995	Infrequently mentioned.
June 1995	Infrequently mentioned.
December 1995	Infrequently mentioned.

Table 12 indicates that 50% of the most widely remembered events are in the 1990's. This figure could suggest that flooding in the 1990's has been a major issue compared to other periods. This is unlikely to be the case, and merely reflects some of the points noted previously regarding obtaining data from residents. Clearly the gauge record (Figure 6) indicates the period from 1958 to 1974 had more large floods.

As part of the 1998 Powells Creek Flood Study (Reference 2) 125 questionnaires were returned out of approximately 800 hand delivered or mailed (to non-resident owners) with some followed up by telephone or field interview. Table 13 summarises the results from this survey.

Table 13: 1998 Flood Study Questionnaire Results

Total number of questionnaires returned	**125 (approx.15%)**
Number who responded indicating that their property had been inundated by a water depth greater than 100 mm.	60 (49%)
Number not inundated.	65 (52%)
Number who could indicate a historical flood level.	39 (31%)
Number of buildings inundated above floor level*.	6 (5%)

Note: * Previous questionnaire surveys have indicated that other buildings have been inundated above floor level.

A questionnaire was distributed as part of the current study with several responses identified recent occurrences of flooding. The reported flooding was generally less than 0.1 m and would be considered nuisance flooding. It has been for general verification of model results. Further details of the community consultation are given in Section 2.13.

2.11.2. Surveyed Levels

A number of historical flood levels were collected from field interviews as part of the 1998 Powells Creek Flood Study (Reference 2). The majority of levels were for either the January

1996 or the February 1990 events. These are shown in Table 14 and on Figure 8.

Table 14: Historical Flood Data from Field Interviews in August 1997 as part of Reference 2

Address	Date of Flood	Depth (m)	Description	Flood Level (mAHD)
No. 21 Llandilo Avenue	Approx 1990	0.05-0.08	Garage Floor Level	29.96
	Approx. 1990	0.8	North-West Corner	28.8
No. 8 Agnes Street	Jan-96	0.1	Driveway and Front Boundary	26.71
No. 41 Albyn Road	Jan-96	0.08	Crest of Driveway	22.54
	Jan-96	0.35	Low Point along West. Boundary	21.64
No. 47 Albyn Road	Jan-96	0.25	Garage Floor Level	21.18
No. 35 Redmyre Road	Jan-96	0.05-0.1	Crest of Driveway	13.26
	Jan-96	0.5	Ground Level at Back Fence	12.13
No. 37 Redmyre Road	Jan-96	0.05-0.1	Crest of Driveway	13.27
	Jan-96	0.3	Ground Level at Garage	12.21
No. 45 Churchill Avenue	Jan-96	0.1	Base Steps at Front House	10.74
No. 60 Churchill Avenue	Jan-96	0.2	Ground Level at Path Granny Flat	11.49
No. 66 Churchill Avenue	18th February 1959	0.3	Floor Level	12.06
Upstream Railway crossing near Elva Street	Unknown		Top coping LHS looking Downstream	8.1
			Top coping RHS looking Downstream	7.83
Pharmacy adjoining Plaza Entrance, The Boulevarde	Jan-96		Floor Level - water entered shop	12.29
No. 11 The Boulevarde (Gumbleys Butchery - now gone)	Nov-61	0.3	Estimated Floor Level	12.55
No. 26 Barker Road	Regularly	0.1	Drive at Boundary	25.83
No. 65 Oxford Street	Jan-96	0.45	Carport Slab	24.16
No. 63 Oxford Street	Jan-96	0.3	South-West corner of house	23.75
No. 61 Oxford Street	Jan-96	0.5	Garage Floor Level	23.24
No. 59 Oxford Street	Jan-96	-	Patio Level	23.14
No. 141 Albert Street	Approx. 1990	0.3	Ground level along eastern fence	19.51
No. 135 Albert Street	Approx. 1990	0.5	Bottom steps rear of house	18.49
No. 137 Albert Street	Feb-90	-	Crest of driveway	19.24
	Feb-90	-	Water reached floor level	19.01
No. 100 Beresford Road	Feb-90	0.1	Driveway at entrance to house	15.91
No. 102 Beresford Road	Feb-90	0.12	Ground level at back door	16.43
No. 104 Beresford Road	Feb-90	0.55	Ground level rear house	17
No. 110 Beresford	Feb-90	0.35	Midway along eastern	17.5

Address	Date of Flood	Depth (m)	Description	Flood Level (mAHD)
Road			fence	
No. 53 Beresford Road	Feb-90	0.05	Garage floor level	15.29
No. 108 Beresford Road	Feb-90	0.34	Base steps rear house	17.49
No. 89 Rochester Street	Feb-90	0.1	Floor level shop	12.84
No. 107 Rochester Street	Jan-89	0.45	GL at rear of house	14.12
No. 109 Rochester Street	Feb-90	0.42	Base steps rear house	14.33
	Jan-96	0.24	Base steps rear house	14.15
No. 57 Rochester Street	Jan-96	0.41	Ground level back yard	9.92
No. 28 Broughton Road	Approx. 1992	0.24	North East corner of house	12.88
No. 33-35 Burlington Road	1989	0.3	Garage Floor Level	9.14
No. 38-46 Burlington Road(Hairdresser)	Feb-90	0.48	Ground level at rear shed	9.71
No. 48 Burlington Road	Jan-96	0.1	Ground Floor Level	9.55
No. 29 Burlington Road	Feb-90	-	Stormwater reached this level at rear of factory	9.16
No. 30 The Crescent (Unit No. 2)	Jan-96	0.4	Garage Floor Level	8.7
No. 31 The Crescent	Jan-96	0.2	Garage Floor Level	8.33
No. 79 The Crescent	Feb-90	0.3	Floor level	8.2
	Jan-96	0.28	Base patio at rear	7.75
No. 12 Loftus Crescent	Feb-90	0.15	Ground level backyard	7.87
No. 82 Underwood Road	Feb-90	0.45	Ground level at front house and driveway	4.97
No. 86 Underwood Road	Jan-96	0.3	Base steps front house	4.89
No. 90 Underwood Road	Jan-96	0.16	Base steps front of house	4.74
No. 22 Ismay Avenue	Approx. 1986	0.3	Ground at back fence	2.2
No. 34 Ismay Avenue	Jan-90	0.35	Path at back door	2.57
No. 60 Ismay Avenue	Jan-96	0.1	Ground level at front of house	3.83
No. 55 Ismay Avenue	Feb-90	0.37	Base front steps	4.3
	Jan-96	0.18	Base front steps	4.11
No. 51 Ismay Avenue	Feb-90	0.3	Base front steps	4.19
No. 56 Ismay Avenue	Feb-90	0.2	Base front steps	3.83
No. 49 Ismay Avenue	Jan-96	0.22	Base front steps	4.16
No. 48 Ismay Avenue	Jan-96	0.15	Base front steps	3.43
No. 41 Ismay Avenue	Feb-90	0.14	Base front steps	3.71
	Jan-96	0.07	Base front steps	3.64
No. 17 Pemberton Street	1992	0.4	Ground level backyard	16.95

Address	Date of Flood	Depth (m)	Description	Flood Level (mAHD)
No. 27 Pemberton Street	1992	0.17	Base steps rear house	18.72
No. 10 Mitchell Road	Jan-96	0.28	Ground level low side house	14.75
No. 6 Mitchell Road	Jan-96	0.24	Ground level low side house	14.35
No. 104 Arthur Street	Jan-96	0.27	Ground level front of house	13.87
No.106 Arthur Street	Jan-96	0.34	Ground level at boundary	13.85
No. 105 Arthur Street	Jan-96	0.55	Ground level at house steps side house	13.89
No. 29 Arthur Street	Jan-96	0.16	Base front steps	13.23
	Jan-96	0.4-0.5	Ground level at rear fence	12.98
No. 6 Kessell Avenue	Jan-96	0.44	Ground level at fence	7.76
	Feb-90	-	Water reached floor level	8.42
Airey Park Photos	Jan-96	0.75	Base wall No. 77	7.65

2.11.3. Sydney Water Data

SWC holds records of flooding on Powells Creek and the relevant information is provided in Table 15. These records show no instances of flooding in 1990 and only one record (Feb 1996) since 1988.

Table 15: Sydney Water Records of Flooding in the Powells Creek Catchment

Date Flooded From	Address	Depth (m)	Level Above Floor (m)	Level Above Coping (m)	Property Inundation	Comments
?/07/1952	135 Albert Road, Strathfield				Y	Flooding due to construction activity-water supply. Loss of goods.
6/05/1953	Lot 3, Allen St, Homebush					Flooding occurred where Council's bridge restricts the flow
6/05/1953	4-6 Elva St, Strathfield					Flooding occurred where the channel is deficient in capacity
6/05/1953	36 Minna St, Burwood					Flooding occurred where the channel & Council's subsidiary drainage works are deficient
6/05/1953	Lot 2 Bates St, Homebush (cnr The Crescent)					Flood waters crossed the road where Council's culvert is deficient in capacity
6/05/1953	103 Parramatta Rd, Strathfield					Flooding occurred where the channel is covered at coping level.
9/02/1956	8-10 Elva St, Strathfield			0.45	Y	At the future gauging site
9/03/1958	2A Belgrave St, Burwood	0.37				Flooding of road only?
9/03/1958	4-6 Elva St, Strathfield			0.75		Flooding
9/03/1958	9 Bold St, Burwood (Minna St, Burwood - west of its intersection with Bold St)	0.53			Y	Water banked up to a max. of 0.53m deep against the northern fence of Minna St.
9/03/1958	33 Nicholson St, Burwood	0.1				Flooding of road only?

Date Flooded From	Address	Depth (m)	Level Above Floor (m)	Level Above Coping (m)	Property Inundation	Comments
9/03/1958	20 Woodside Ave, Burwood	0.15				Flooding of road only?
9/03/1958	36A Nicholson St, Burwood	0.05			Y	Water (0.05) deep northern side Nicholson St & sewer surcharge in No. 6A
9/03/1958	24 The Boulevard, Strathfield	0.6			Y	Flood entered the shop and damaged the stock- insufficient inlets
17/02/1959	5 Bold St, Burwood		0.45		Y	Flooding occurred above garage floor level at rear of house, but 0.65m below floor level of house
17/02/1959	7 Bold St, Burwood		0.56		Y	Flooding occurred above garage floor level at rear of house, but .28m below floor level of house
18/02/1959	4-6 Elva St, Strathfield			1.14	Y	1.14m above the coping level of the Stormwater channel at Gauging Station. Floodwater entered the Elva Street and carried some of the timbers away
18/02/1959	2 Elva Street, Strathfield			1.24	Y	
18/02/1959	58 Churchill Avenue		1.5		Y	1.5 m above the kitchen floor. No damage was done and the kitchen floor is considerably lower than the back yard.
18/02/1959	66 Churchill Avenue		0.3		Y	0.3 m above the floor. Water coming from Redmyre Road has swept through the house and damaged carpets and furniture. Many premises had been flooded.
18/02/1959	27 Minna St, Burwood	0.84			Y	Flooding occurred above the yard level at N/W corner of house, but was 0.35m below floor level of house
30/10/1959	7 Bold St, Burwood					Slight flooding only. Flood water rose to 0.30m above footpath level, no houses flooded
17/11/1961	53 Ismay Ave, Homebush				Y	Flooding of homes reported.
19/11/1961	19 Oxford St, Burwood		0.15		Y	Above floor flooding
19/11/1961	21 Morwick St, Strathfield		0.3		Y	Above floor flooding
19/11/1961	26 Morwick St, Strathfield		0.025		Y	New block of home units, water rose to within .025m of floor level & 0.38m above laundry floor.
19/11/1961	41 Woodside Ave, Burwood				Y	Brick fence along the frontage collapsed
19/11/1961	19 Oxford St, Burwood		0.15		Y	Above floor flooding
19/11/1961	62/64 Oxford St, Burwood				Y	Extensive damage to fencing & back gardens
19/11/1961	4-6 Elva St, Strathfield		0.87		Y	Harrisons Timber P/L flooded. Damage to motors & furniture.
19/11/1961	8-10 Elva Street				Y	Flood water was just below the floor level. Garden was ruined. Photos available
19/11/1961	7 Bold St, Burwood.				Y	Severe flooding. Flood water rose to 0.75m above footpath level on North side of Minna St - 19th 4.00 a.m. The water was held back by the side palings of the house No.7 Bold Street but eventually found an

Date Flooded From	Address	Depth (m)	Level Above Floor (m)	Level Above Coping (m)	Property Inundation	Comments
						outlet through No. 27 Minna Street.
19/11/1961	27 Minna Street				Y	Water rose .1m below the floor level of the rear house
19/11/1961	35 Nicholson Street	0.73			Y	Water level was 0.73 m above ground level and .3 m below the floor level.
19/11/1961	11 The Boulevarde(Gumbleys Butchery), Strathfield.		0.3		Y	Water entered several shops & rose to about 0.30m above floor in Gumbleys Butchery at No. 11
19/11/1961	2 Elva St, Strathfield (U/S main Western Railway Line)					Considerable damage done along route of main channel. S/water unable to reach underground drains flowed over ground surface to low lying areas & followed course of original creek downstream.
7/05/1963	2 Elva Street, Strathfield			0.6	Y	Observed at 8.15am. High tide at 7.15 am= 1.4m?
20/12/1963	12, 13, 14, 15, 16 & 17 Brunswick St, Strathfield				Y	Flooding of roadway & front yards, did not enter premises. Date of rain-not clear
20/12/1963	2 Elva St, Strathfield , (Railway viaduct on Main Western Line)			0.75	Y	No apparent damage to properties.
9/06/1964	2 Elva St, Strathfield - Sydney Night Patrol			1.52	Y	Flooding caused by culvert under railway + 2 curves immediately upstream. Property flooding = .9m above ground
11/06/1964	2 Elva St, Strathfield - Sydney Night Patrol			0.46	Y	Flooding caused by culvert under railway + 2 curves immediately upstream.
15/04/1969	177 Parramatta Rd, Homebush				Y	A brick retaining wall collapsed at Saleyards Ck Bch. Poor foundation
29/10/1972	2 Elva St, Strathfield - Sydney Night Patrol				Y	Water rose to 1.22m above brickwork recently added to walls within this property. Vehicles were submerged & a wooden bridge lifted & dumped 9m downstream.
29/10/1972	11 Pilgrim Avenue				Y	Basement of a block oh home units was flooded - approximately 1 metre
29/10/1972	2 Elva St, Strathfield (Railway Culvert under the Main Western Line)					Embankment surcharged - see photo
17/03/1983	167-173 Parramatta Road, Homebush	0.3			Y	Flood level 300 mm above footpath. Above floor flood in one work shop-150mm
8/11/1984	7-9 Underwood Road, Homebush			0.6	Y	Debris mark on the fence
8/11/1984	Lot 2 Bates St, Strathfield (cnr The Crescent, Railway Culvert upstream)			0.6	Y	Debris on the embankment
29/04/1988	53 Ismay St, Homebush				Y	Surface flooding of 5 houses in Ismay Ave & overland flow at Powell St.
29/04/1988	Flemington Markets, Parramatta Rd, Homebush					Channel overflowed near markets.
29/04/1988	Lot 2 Bates St, Homebush (U/S of The Crescent, Homebush)			0.3	Y	Was contained within the banks. Flood debris 800 mm above the ground at upstream railway line culvert

Date Flooded From	Address	Depth (m)	Level Above Floor (m)	Level Above Coping (m)	Property Inundation	Comments
7/05/1988	32 The Crescent, Homebush				Y	Above floor flooding. Damage $10,000
2/02/1996	Lot C Allen St, Nth Strathfield					Debris on adjacent fences indicated water flowed 500mm above upstream headwall. Flooding confined to adjacent park.
2/02/1996	24 Pomeroy St, Strathfield			0.3	Y	

2.12. Flood Photographs

A number of flood photographs taken during floods were provided by SMC and these are shown on Figure 5.

2.13. Community Consultation

Community consultation was undertaken as part of the current study to inform the community about the study and gather information on historical flood events. A one-page newsletter detailing the study's purpose was sent to approximately 300 addresses in the study area. The newsletter, which was sent in July 2015, also described the floodplain management process and the flood study's role in managing the area's flood risk. The mailout also included a questionnaire requesting information on any experience of flooding, including the level of affectation and the date of the event. Accounts of flooding could then be used to add to verify the model behaviour and generally add to the knowledge of the area's flood behaviour.

From the questionnaire, twelve responses were received, constituting a response-rate of around 5%. The results from the questionnaires are as follows:

- All responses were from residential properties, with most having lived there for more than 15 years.
- 7 respondents had experienced flooding, with all instances involving water above floor levels of the house or other buildings.
- Approximately 9 events in the last 20 years were identified as causing flooding, with flooding reported in 1995, 1996, 1998, 2005, 2010, three times in 2014 and 2015. However, most events had only one reported instance of flooding, and apart from 0.3 m depth reported for 1995, all depths were 0.2 m or less. No event was consistently mentioned between responses, which suggests variation in flood behaviour between similar events, for example due to pit or pipe blockage, location of the rainfall burst or localised effects on flow behaviour.

Figure 8 shows the location of the respondents, alongside the previous consultation and the Sydney Water historical data.

3. APPROACH

The approach adopted in flood studies to determine design flood levels largely depends upon the objectives of the study and the quantity and quality of the data (survey, flood, rainfall, flow etc.). Whilst there is a limited flood record from the Elva Street gauge there is no extensive historical flood record elsewhere on Powells Creek or on Saleyards Creek. A flood frequency approach can be undertaken at the Elva Street gauge but reliance must also be made on the use of design rainfalls and establishment of a hydrologic/hydraulic modelling system. A diagrammatic representation of the flood study process undertaken in this manner is shown below.

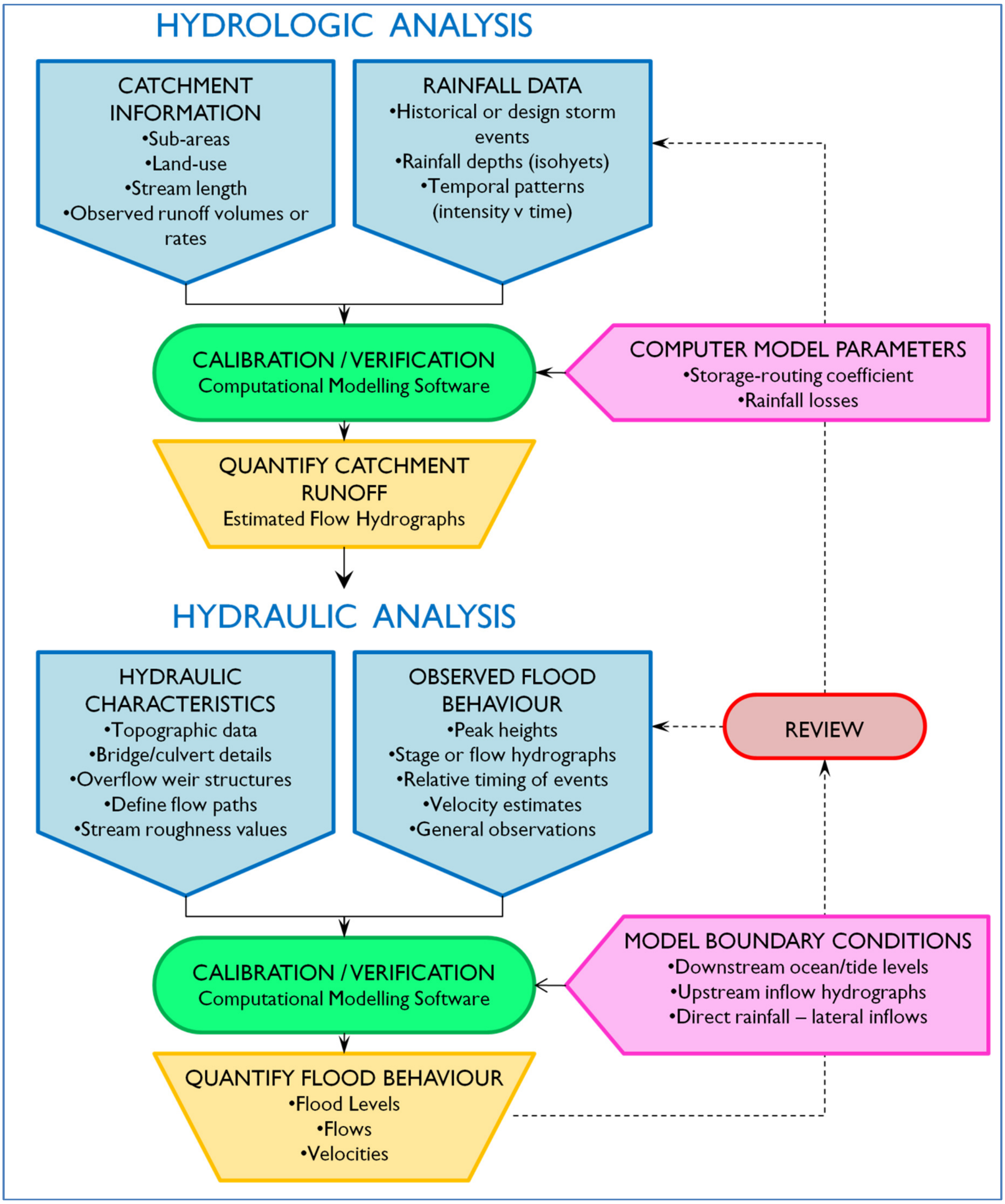


Diagram 2: Flood Study Process

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, 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 to peak level data is the only option and a detailed sensitivity analysis of the different model input parameters constitutes current best practice.

The use of a flood frequency approach for the estimation of design floods and/or independent calibration of the hydrologic model is possible for the Powells Creek catchment using the Elva Street gauge data.

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 discrete models. As such, 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 (Reference 4) 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 such that the overland flow behaviour for the study was generally defined by the hydraulic model. This joint modelling approach was then verified against previous studies and historical data.

3.1. Hydrologic Model

Inflow hydrographs are required as inputs at the boundaries of the hydraulic model. Typically in flood studies a rainfall-runoff hydrologic model (converts rainfall to runoff) is used to provide these inflows. A range of runoff routing hydrologic models is available as described in AR&R 1987 (Reference 4). These models allow the rainfall depth to vary both spatially and temporarily over the catchment and readily lend themselves to calibration against recorded data.

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 same 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 in AR&R 1987 (Reference 4) for the design of pipe systems. 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 (Reference 8) was adopted as it is widely used in Australia and WMAwater has 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 Powells Creek 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 models 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 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. Assessment of Data from UNSW Elva Street Gauge

3.3.1. Overview

It is important that the best possible use is made of the available data as this is the only urban catchment in Sydney where there is a long term record for use in flood frequency analysis and which can be used to calibrate hydrologic (flows) and hydraulic (water level) models. However, there are a number of issues with the data and these are discussed below.

3.3.2. Gaugings and Rating Curve

The cross-sectional area of the channel has not changed (lined 'U' shaped channel) since 1958 although the coping has been raised. The gauge zero is at RL 5.25 mAHD and over 29 stream gaugings have been taken. The channel is well gauged below 1 m (RL 6.25 mAHD); there are 14 gaugings below 0.5 m (RL 5.75 mAHD); 14 gaugings between 0.5 m and 1.0 m; and the highest gauging is at 1.35 m (RL 6.6 mAHD). The gaugings show very little scatter and fit as a smooth line on log-log paper. Above 0.2 m the flow tends to be supercritical and velocities are very high (above 4 m/s). This is the greatest source of uncertainty in the gauging as the velocity is above the normal range of the current meter used to take velocity measurements.

There are three known rating curves (Figure 7) as indicated below but the Reference 2 and digital record curves are practically identical and shown as the same:

- used in Reference 6 and taken from UNSW records at the time;
- used in the 1998 Powells Creek Flood Study (Reference 2);
- used in the digital records.

As part of the present study a rating curve is produced from the TUFLOW model (Figure 7). All the prior curves, whilst based on various velocity gaugings aimed to extend the rating curve beyond the highest flow gauging height of 1.35 m (RL 6.6 mAHD).

It is interesting to note that the Reference 2 rating curve and the TUFLOW model rating curves are relatively similar in magnitude at a given height. The TUFLOW model rating produces a smaller flow up to approximately 1.8 m before transitioning to produce larger flows than the Reference 2 rating above this level.

Uncertainty between the prior rating curves listed above increases once the flow breaks out of the channel (approximately at 2.5 m or RL 7.75 mAHD). The channel may also choke downstream at very high depths. This is not the case with the TUFLOW model rating which performs equally well for both in and out of bank floods. Since approximately 2000 there have been significant changes in the number and size of the bridges across the channel in the immediate reach upstream from the railway line. There is no complete record of the dates when bridges have been removed or installed. The presence of bridges will influence the high flow rating but for the majority of the record the events were not above the coping and thus not influenced by these changes.

3.3.3. For Use in Flood Frequency Analysis

Flood frequency analysis is the fitting of statistical distribution to either the annual maxima peaks or a partial series which are events above a threshold. Partial series analysis is not possible for this study as there are too many gaps in the record. Whilst the gaps in the record also affect the annual maxima series it is expected that this approach will still provide a robust result.

Derivation of the annual maxima needs to address whether the record should be based on just the digital record or whether it should be extended to include the data shown in Table 11, and whether the record should be extended from the end of the digital record (1997) to date. It is known that there have been no large events since 1997.

A tabulation of the annual maxima from the various sources is provided on Table 16.

Table 16: Annual Maxima Peaks

Year	Peak Stage (m) from Reference 6	Peak Stage (m) from Digital Records	Difference in Peak Stage (m)	Peak Flows from Reference 6 (m³/s)	Peak Flows from 1998 Flood Study Reference 2 (m³/s)	Peak Flows from Digital Record (m³/s)
1958		1.48			16.0	16.1
1959	3.29	3.26	0.03	29.9	48.2	49.1
1960	1.30	1.12	0.18	11.1	10.8	10.6
1961	4.18	0.79	3.39	38.3	7.0	5.9
1962	1.69	1.74	-0.05	14.8	20.0	20.3
1963	2.40	2.47	-0.07	22.0	33.0	32.1
1964	3.52	1.88	1.64	32.1	25.3	22.5
1965	1.02	0.88	0.14	8.0	8.8	7.2
1966	1.28	1.23	0.05	10.9	12.6	12.3
1967	1.52	1.40	0.12	13.2	17.2	14.9
1968	0.84	0.70	0.14	5.9	5.3	4.7
1969	1.71	1.62	0.09	15.1	18.3	18.4
1970	3.09	1.43	1.66	28.0	17.4	15.4
1971	1.93	1.10	0.83	17.8	12.1	10.3
1972	3.20	2.76	0.44	29.1	38.0	37.3
1973	2.35	2.17	0.18	21.5	33.5	27.1
1974	2.34	2.23	0.11	21.4	28.9	28.0
1975	1.58	1.52	0.06	13.8	17.0	16.7
1976	1.70	1.25	0.45	14.9	14.9	12.6
1977	1.15	1.49	-0.34	9.6	16.5	16.3
1978	1.47	1.38	0.09	12.7	15.1	14.6
1979	1.27	1.22	0.05	10.8	12.6	12.1
1980	1.26	1.27	0.00	10.7	12.7	12.8
1981	1.41	1.38	0.03	12.1	14.6	14.6
1982	1.71	1.67	0.04	15.1	19.3	19.1
1983	1.83	1.80	0.03	16.8	21.3	21.2
1984	1.84	1.81	0.03	16.9	21.3	21.4
1985	1.30	1.21	0.09	11.1	13.1	11.9
1986	1.93	1.73	0.20	17.8	20.2	20.1
1987		1.18			11.8	11.4
1988		1.92			23.1	23.1
1989		1.28			13.9	13.0
1990		1.92			23.3	23.1
1991		1.68			19.2	19.2
1992		1.53			17.1	16.9
1993		1.88				22.4
1994		1.44			6.9	15.4
1995		1.31			13.3	13.4
1996		0.90			7.8	7.4
1997		0.86			7.6	6.9

3.4. Calibration and Verification of the Modelling Process

3.4.1. Approach

As flow data is available from the Elva Street gauge this means that the catchment hydrology (flows) can be calibrated and verified at this location. This is a significant advantage for this catchment as this is possible for only approximately 10 urban catchments in Australia and less than 5 in NSW. TUFLOW model peak levels and the shape of the hydrograph can also be calibrated to water level data from the Elva Street gauge.

In addition, peak levels from TUFLOW can be calibrated to observed water level data provided by Council and Sydney Water (Section 2.11 and Figure 8).

The stages in the model calibration approach were as follows:

1. collect available historical rainfall and water level data;
2. select events for calibration and verification based on the quality and quantity of available data;
3. input historical rainfall data for calibration event to DRAINS;
4. input output of above DRAINS model to TUFLOW;
5. run TUFLOW for historical event;
6. compare output from TUFLOW for calibration event at the Elva Street gauge and other locations where historical flood height data are available;
7. re run steps 3 to 6 and adjust model parameters until a suitable match is obtained;
8. re run steps 3 to 6 for verification events without adjustment of model parameters;
9. compare output from TUFLOW from verification events at the Elva Street gauge and other locations where historical flood height data are available;
10. re-run steps 3 to 9 until a satisfactory calibration/verification is achieved.

3.4.2. Calibration Events

The choice of floods used in calibration depends upon a number of factors including the:

- *time since the flood occurred.* The longer the time since a flood occurred, the greater the likelihood of subsequent changes to the catchment. The major changes in recent times have been construction/alterations to buildings and fences in the floodplain and to the piped drainage system. The most significant change in recent times at the Elva Street gauge is construction of several bridges across the channel. However, as all the recent events suitable for calibration did not overtop the coping the impact of new bridges is not relevant;
- *quantity and quality of rainfall and streamflow data which are available*. This should have been of lesser importance in this study as data are available from two well placed pluviometers and the Elva Street water level gauge. However, problems with the UNSW rainfall and water level data meant that this became the most important factor in determining the choice of events;
- *quantity, quality and location of recorded levels along the creeks*. It may be preferable to use a small flood with several levels which define a profile rather than a large flood with only one level. This issue is of little significance as there are few events with suitable recorded levels, apart from at the gauge;
- *magnitude of the flood levels*. The larger the flood the more suitable it is for calibration as it is closer to the larger design flood events.

The following is a summary of the available data considered suitable for calibration.

2 January 1996

- Elva Street water level gauge malfunctioned and the Elva Street pluviometer had no digital record. The St Sabina pluviometer recorded 62 mm in 45 minutes;
- only record available for Sydney Water gauge under the M4;
- 39 flood levels are available (Table 14);
- at Enfield this event approached a 1% AEP (20 min to 60 min duration) but was approximately only a 5% AEP (or less) at the other gauges.

8 or 9 February 1992

- the Elva Street gauge recorded a peak of 1.5 m and it would appear from the available pluviometer records that this was not a large event. For this reason it is not suitable for calibration purposes.

11 March 1991

- the Elva gauge recorded a peak of 1.7 m and the rainfall intensity approached a 10% AEP (30 minute duration) at Enfield but the lack of other flood height data and failure of both the UNSW pluviometers meant this flood was not suitable for calibration purposes.

18 March 1990

- the flood was approximately a 30% AEP event at the St Sabina pluviometer and a 5% AEP (30 minute duration) at the Elva Street pluviometer. The peak levels and flows at the Elva Street gauge are 1.92 m and approximately 23 m^3/s (based on the UNSW rating curve),
- the availability of water level and pluviometer records from the UNSW gauges meant that this event could be used for calibration at the Elva Street gauge. However, no flood height data were available for calibration of the TUFLOW model elsewhere.

February 1990

- four peaks occurred during February 1990 (3rd, 7th, 10th and 17th). The water level and pluviometer data (UNSW gauges) are shown on Figure 9. The peak levels and flows (based on the UNSW rating curve) at the Elva Street gauge are:
  - 3rd Feb 1990 - 1.4 m - 14 m^3/s,
  - 7th Feb 1990 - 1.4 m - 15 m^3/s,
  - 10th Feb 1990 - 1.8 m - 21 m^3/s,
  - 17th Feb 1990 - 1.1 m - 11 m^3/s,
- several flood levels (assumed to be for 10th February 1990) are available (Table 14),
- the 10th February event was slightly less than a 20% AEP rainfall event (30 minute and 60 minute durations);
- the water level records indicates a peak on the morning of 8th February 1990. This is not compatible with the rainfall record which indicates that the peak was approximately 24 hours earlier. It has been assumed that the timing on the water level gauge malfunctioned;
- the availability of pluviometer and water level data from the UNSW gauges meant that all four events could be used for calibration at the Elva Street gauge. The largest event

(10th February) was used for calibration of the TUFLOW model as it is presumed the recorded flood levels relate to this event.

4-6 August 1986:

- digital records from the Elva Street gauge shown no record for this event. However Reference 2 indicates a peak of 1.95 m obtained from data collected as part of Reference 6,
- the St Sabina pluviometer malfunctioned and the Elva Street pluviometer recorded a maximum of 21 mm in 30 minutes which is only modest rainfall. For this reason this event could not be used for calibration.

Summary

Five events (3rd, 7th, 10th and 17th February 1990 and 18th March 1990) were available for calibration of the Elva Street gauge and two events (10th February 1990 and 2nd January 1996) for calibration of the TUFLOW model.

3.5. Design Flood Modelling

Following model establishment and calibration the following steps were undertaken:

- design tributary inflows were obtained from the DRAINS hydrologic model and included in the TUFLOW model;
- flood frequency of the Elva Street gauge records;
- assessment of the design event causing the maximum water levels which is termed the critical storm duration;
- sensitivity analyses to assess the effect of changing model parameters and the assumed water level in the Parramatta River;
- assessment of possible effects of climate change on design flood levels.

4. HYDROLOGIC MODELLING

4.1. Sub-catchment Definition

The total catchment represented by the current DRAINS model is 8.45 km^2. This area has been represented by 749 sub-catchments (Figure 10) giving an average sub-catchment size of approximately 1.13 hectares. 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 pit and pipe network is shown on Figure 11. The drainage system defined in the model comprises:

- 2,156 pipes;
- 1,847 inlet pits;
- 96 upstream inlet pits;
- 317 junction pits.

4.2. Impervious Surface Area

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete surfaces occurs 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 impervious 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 Powells Creek 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 17.

Table 17: Impervious Percentage per Land-use

Land-use Category	Impervious Percentage
Residential/Commercial property	60% Impervious
Non-bitumen road reserve	60% Impervious
Vacant land	0% Impervious
Green space (such as public parks)	0% Impervious
Roadway/Car parks	100% Impervious
Waterways	0% Impervious

4.3. Rainfall Losses

Methods for modelling the proportion of rainfall that is “lost” to infiltration are outlined in AR&R (Reference 4). 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 18. These are consistent with the parameters adopted in previous studies undertaken by WMAwater.

Table 18: 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

4.4. Design Rainfall Data

Rainfall intensities were derived from the BoM website using AR&R (Reference 4) data. Calculation of the Probable Maximum Precipitation (PMP) was undertaken using the Generalised Short Duration Method (GSDM) according to Reference 7.

For the PMP estimate the following criteria applied:

- as the catchment area is less than 1000 km^2 and located in the coastal transitional area the Generalised Short Duration Method (GSDM) was adopted;
- zero adjustment for elevation was assumed as the catchment topography is less than 1500 mAHD;
- a moisture adjustment factor of 0.7 was adopted;
- the catchment is considered to be 100% 'smooth'.

5. HYDRAULIC MODELLING

5.1. TUFLOW

The TUFLOW modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water equations in two dimensions. The TUFLOW software has been widely used for a range of similar floodplain projects both internationally and within Australia and is capable of dynamically simulating complex overland flow regimes. The TUFLOW model build used in this study is 2013-12-AC-w64 and further details regarding TUFLOW software can be found in the User Manual (Reference 8).

The model uses a regularly spaced computational grid, with a cell size of 2 m by 2 m. This resolution was adopted as it provides an appropriate balance between providing sufficient detail for roads and overland flow paths, while still resulting in workable computational run-times. The model grid was established by sampling from a DEM generated from a triangulation of filtered ground points from the ALS dataset, discussed in Section 2.6 and shown in Figure 3.

The TUFLOW hydraulic model includes the Powells Creek catchment to Homebush Bay with the open channel in 1D and the overland areas in 2D. The total area included in the 2D model is approximately 10 km^2. The extents of the TUFLOW model are shown in Figure 12.

5.2. Boundary Locations

5.2.1. Inflows and Downstream Boundary

Local runoff hydrographs were extracted from the DRAINS model for inclusion within the TUFLOW model domain. 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.

The downstream boundary was located at the Parramatta River, as shown in Figure 12.

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 Manning's "n" values adopted, including flowpaths (overland, pipe and in-channel), are shown in Table 19 and were based on site inspection and past experience in similar floodplain environments.

Table 19: Manning's "n" values adopted in TUFLOW

Material	Manning's n Value
Bitumen road reserve and some car parks	0.02
Green Space - Golf Course, Parks, Vacant Lots	0.04
Residential/urban area	0.03
Non-bitumen road reserve	0.032
Waterways	0.015
Pipes	0.012

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 12, bridges were modelled as 1D features within the 1D channels, with the purpose of maintaining continuity within the model.

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.

5.5. 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 occurred in the Newcastle area in June 2007. 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 AEP of the event. Storm duration is another influencing factor, with the mobilisation of blockage materials generally increasing with increasing storm duration.

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 20.

Table 20: Suggested 'Design' and 'Severe' Blockage Conditions for Various Structures (AR&R Revision Project 11, 2013)

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

Current modelling has been undertaken assuming no blockage of pipes, culverts and bridges greater than 300 mm in diameter. Pipes less than or equal to 300 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 9. These scenarios included blockage of all pipes, blockage of bridges/culverts over the open channel, and blockage of the drainage infrastructure. Blockage was assumed to occur laterally across the cross-section. 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 Powells Creek catchment.

No historical evidence of blocking in the catchment is available; however, it is possible that changed activities on the floodplain may mean that there may be a higher chance of blockage today than in the past. For example, colourbond fencing is much less permeable and less likely to collapse than the more traditional paling fencing. Individual palings becoming mobile in a flood are also less likely to cause blockage than a panel of colourbond fencing. In some council areas garbage bins are known to become mobile during floods and can cause blockage. In summary, it is impossible to accurately determine whether blockage will or will not be an issue in the next flood.

5.6. Ground Truthing

Inspection of the above-ground features along the catchment's overland flowpaths was undertaken following calibration and verification of the hydraulic model. This entailed producing design flood results and mapping the peak flood depth in detail across the catchment. This allowed identification of features (largely buildings) that blocked or partially blocked overland flow. Model schematisation of these features was then compared to the actual features on a site visit, and the model was updated where any discrepancy was identified. Changes were minor and only impacted results in the vicinity of the modification. The most common change was to areas where two houses had been represented as a single impermeable barrier in the model grid, which was amended to allow flow between the buildings.

6. MODEL CALIBRATION AND VERIFICATION

6.1. Introduction

It is important that the performance of the overall modelling system be substantiated prior to defining design flood behaviour.

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

6.2. Results

The results of the calibration and verification process using the six historical events are shown on Figure 13 (Elva Street Gauge) and Figure 14 (across catchment) and on Table 21 (Elva Street Gauge) and Table 22 (across catchment).

Table 21: Calibration Results - Elva Street Gauge

Date	Recorded Level (m AHD)	Modelled Level St Sabina Pluviometer (m AHD)	Difference (m)	Modelled Level Elva St Pluviometer (m AHD)	Difference (m)
3-Feb-90	6.67	6.59	-0.08	6.61	-0.06
7-Feb-90	6.68	6.68	0.00	6.77	0.09
10-Feb-90	7.00	6.80	-0.20	7.01	0.01
17-Feb-90	6.41	6.46	0.05		
18-Mar-90	7.13	6.75	-0.38		
2-Jan-96		8.06			

Table 22: Calibration Results - Peak Heights

Address	Location	Surveyed Level 1990 February 10 (mAHD)	Surveyed Level 1996 January 2 (mAHD)	Modelled Level 1990 February 10 (mAHD)	Modelled Level 1996 January 2 (mAHD)	Difference- 1990 February 10 (mAHD)	Difference- 1996 January 2 (mAHD)
21 Llandilo Avenue	Garage Floor Level	29.9	N/A	30.03	N/A	0.13	N/A
21 Llandilo Avenue	North-West Corner	28.8	N/A	28.69	N/A	-0.11	N/A
8 Agnes Street	Driveway and Front Boundary	N/A	26.71	N/A	26.66	N/A	-0.05
41 Albyn Road	Crest of Driveway	N/A	22.54	N/A	22.48	N/A	-0.06
41 Albyn Road	Low Point along West. Boundary	N/A	21.64	N/A	21.58	N/A	-0.06
47 Albyn Road	Garage Floor Level	N/A	21.18	N/A	21.16	N/A	-0.02
37 Redmyre Road	Crest of Driveway	N/A	13.27	N/A	13.10	N/A	-0.17
37 Redmyre Road	Ground Level at Garage	N/A	12.21	N/A	12.44	N/A	0.23
35 Redymre Road	Crest of Driveway	N/A	13.26	N/A	13.12	N/A	-0.14
35 Redymre Road	Ground Level at Back Fence	N/A	12.13	N/A	12.07	N/A	-0.06
45 Churchill Avenue	Base Steps at Front House	N/A	10.74	N/A	11.02	N/A	0.28
60 Churchill Avenue	Ground Level at Path Granny Flat	N/A	11.49	N/A	11.42	N/A	-0.07
Pharmacy adjoining Plaza Entrance, The Boulevarde		N/A	12.29	N/A	12.70	N/A	0.41
65 Oxford Street	Carport Slab	N/A	24.16	N/A	23.98	N/A	-0.18
63 Oxford Street	South-West corner of house	N/A	23.75	N/A	23.60	N/A	-0.15
61 Oxford Street	Garage Floor Level	N/A	23.24	N/A	22.99	N/A	-0.25
59 Oxford Street	Patio Level	N/A	23.14	N/A	23.06	N/A	-0.08
141Albert Street	Ground level along eastern fence	19.51	N/A	19.31	N/A	-0.20	N/A
135 Albert Street	Bottom steps rear of house	18.49	N/A	18.26	N/A	-0.23	N/A
137 Albert Street	Crest of driveway	19.24	N/A	19.06	N/A	-0.18	N/A
137 Albert Street	Water reached floor level	19.01	N/A	18.93	N/A	-0.08	N/A
100 Beresford Road	Driveway at entrance to house	15.91	N/A	15.80	N/A	-0.11	N/A
102 Beresford Road	Ground level at back door	16.43	N/A	16.46	N/A	0.03	N/A
104 Beresford Road	Ground level rear house	17	N/A	16.81	N/A	-0.19	N/A
110 Beresford Road	Midway along eastern fence	17.5	N/A	17.63	N/A	0.13	N/A
108 Beresford Road	Base steps rear house	17.49	N/A	17.33	N/A	-0.16	N/A

Address	Location	Surveyed Level 1990 February 10 (mAHD)	Surveyed Level 1996 January 2 (mAHD)	Modelled Level 1990 February 10 (mAHD)	Modelled Level 1996 January 2 (mAHD)	Difference-1990 February 10 (mAHD)	Difference-1996 January 2 (mAHD)
53 Beresford Road	Garage floor level	15.29	N/A	15.17	N/A	-0.12	N/A
89 Rochester Street	Floor level shop	12.84	N/A	12.70	N/A	-0.14	N/A
109 Rochester Street	Base steps rear house	14.33	N/A	14.23	N/A	-0.10	N/A
109 Rochester Street	Base steps rear house	N/A	14.15	N/A	14.33	N/A	0.18
57 Rochester Street	Ground level back yard	N/A	9.92	N/A	10.45	N/A	0.53
38-46 Burlington Road	Ground level at rear shed	9.71	N/A	9.93	N/A	0.22	N/A
48 Burlington Road	Ground Floor Level	N/A	9.55	N/A	9.49	N/A	-0.06
29 Burlington Road	Stormwater reached this level at rear of factory	9.16	N/A	8.97	N/A	-0.19	N/A
30 The Crescent	Garage Floor Level	N/A	8.7	N/A	8.58	N/A	-0.12
31 The Crescent	Garage Floor Level	N/A	8.33	N/A	8.23	N/A	-0.10
79 The Crescent	Floor level	8.2	N/A	7.13	N/A	-1.07	N/A
79 The Crescent	Base patio at rear	N/A	7.75	N/A	7.8	N/A	0.05
12 Loftus Crescent	Ground level backyard	7.87	N/A	Local runoff	N/A	Local runoff	N/A
86 Underwood Road	Base steps front house	N/A	4.89	N/A	5.02	N/A	0.13
82 Underwood Road	Ground level at front house and driveway	4.97	N/A	4.50	N/A	-0.47	N/A
90 Underwood Road	Base steps front of house	N/A	4.74	N/A	4.95	N/A	0.21
60 Ismay Avenue	Ground level at front of house	N/A	3.83	N/A	3.80	N/A	-0.03
55 Ismay Avenue	Base front steps	4.3	4.11	4.02	4.27	-0.28	0.16
51 Ismay Avenue	Base front steps	4.19	N/A	3.94	N/A	-0.25	N/A
56 Ismay Avenue	Base front steps	3.83	N/A	3.78	N/A	-0.05	N/A
49 Ismay Avenue	Base front steps	N/A	4.16	N/A	3.97	N/A	-0.19
48 Ismay Avenue	Base front steps	N/A	3.43	N/A	3.43	N/A	0.00
41 Ismay Avenue	Base front steps	3.71	N/A	Local runoff	N/A	Local runoff	N/A
10 Mitchell Road	Ground level low side house	N/A	14.75	N/A	14.7	N/A	-0.05
6 Mitchell Road	Ground level low side house	N/A	14.35	N/A	14.33	N/A	-0.02
104 Arthur Street	Ground level front of house	N/A	13.87	N/A	13.72	N/A	-0.15
106 Arthur Street	Ground level at boundary	N/A	13.85	N/A	13.78	N/A	-0.07
105 Arthur Street	Ground level at house steps side house	N/A	13.89	N/A	13.94	N/A	0.05
29 Arthur Street	Base front steps	N/A	13.23	N/A	13.36	N/A	0.13
29 Arthur Street	Ground level at rear fence	N/A	12.98	N/A	12.88	N/A	-0.10

Address	Location	Surveyed Level 1990 February 10 (mAHD)	Surveyed Level 1996 January 2 (mAHD)	Modelled Level 1990 February 10 (mAHD)	Modelled Level 1996 January 2 (mAHD)	Difference-1990 February 10 (mAHD)	Difference-1996 January 2 (mAHD)
6 Kessell Avenue	Ground level at fence	N/A	7.76	N/A	7.80	N/A	0.04
6 Kessell Avenue	Water reached floor level	8.42	N/A	8.15	N/A	-0.27	N/A

Note: Local runoff denotes when the flooding is very localised and is therefore not identified in the TUFLOW model.
Highlighted values are referred to in discussion of results across the catchment.

6.3. Discussion of Results

6.3.1. Elva Street Gauge

Apart from 18th March 1990 and to a lesser extent 10th February 1990, there is a good match to the peak at the Elva Street gauge using the St Sabina pluviometer. The use of the Elva Street pluviometer significantly improves the match for the 10th February 1990 event compared to using the St Sabina pluviometer.

For all events the relative timings of the water level gauge and the pluviometer are incorrect due to timing errors with the instruments. This was recognised in Reference 6 and an attempt was made to correct this by assuming that the "clocks" decrease or increase in speed linearly (this can be calculated as the on and off times are recorded and the elapsed real time can be compared to the chart time).

In general the gauge shows a much more rapid rise and fall than the model results, particularly on the falling limb. Thus the model assumes a greater volume of runoff than actually occurs.

Where comparisons can be made, the results from the St Sabina and Elva Street pluviometer show similar shapes of hydrographs. The timing of the two pluviometers are also similar suggesting that the error in timing is the water level gauge. The two pluviometers are only 800 m apart but timing differences may reflect the passage of a storm across the area.

6.3.2. Across the Catchment

For the historical event of 10th February 1990, most of the differences between surveyed and modelled levels were within 0.2 m. However, the modelled flood level at 79 The Crescent was 1.07 m below the level recorded at the floor. The ALS at this location was 7.05 mAHD which was far lower than the recorded flood level of 8.2 mAHD.

The differences were also generally within 0.2 m for the historical event of 2nd January 1996. The recorded flood level at the ground level at 57 Rochester Street was 0.53 m lower than the modelled level. This cannot be explained as the location is a major flow path with depths up to 0.8m deep. The ALS at the Pharmacy adjoining Plaza Entrance indicates ground levels of 12.49 mAHD, which is higher than the recorded level and the reason that the modelled level was 0.41 m higher.

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. Powells Creek is one of the two catchments in the Sydney basin that has a reasonably reliable water level record over a long period and has had velocity gaugings undertaken (required to derive a rating curve). Thus flood frequency analysis can be undertaken. However this approach only provides results at the gauge location and a *rainfall and runoff routing* approach, using DRAINS model results, is also required to derive inflow hydrographs to the TUFLOW hydraulic model, which determines design flood levels, flows and velocities in areas beyond the actual gauge location. This approach reflects current engineering best practice and is consistent with the quality and quantity of available data.

7.2. Critical Duration for Rainfall Runoff Approach

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 4.5 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 produce the highest flood levels across the entire catchment for the 1% AEP event. However, having a combination of storm durations is difficult to manage (for example which event produces the peak velocity or peak hazard). It is therefore preferable to adopt a single storm duration for design flood estimation.

The 25 minute design storm duration was mostly higher in areas of shallow overland flow (92% of the area having a peak flood depth no greater than 0.3 m). As such, the 25 minute storm does not reflect the areas of deeper flow which are considered more hazardous. The 2 hour storm duration was selected as it was the critical storm over a greater area than the 1 hour storm duration. However, the height difference between the two durations was within ± 0.025 m across 90% of the area affected by these two durations.

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 the 1 hour storm duration was critical in the

PMF event.

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 the Parramatta River and Powells Creek. Consideration must therefore also be given to accounting for the joint probability of 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. However, the 1986 Parramatta River Flood Study (Reference 9) indicates that in this reach of the river the design water level is determined by the tide level and no design flood levels are provided. For the present study a constant water level of 1m AHD was applied to the downstream boundary for each design rainfall event. As the typical tidal in Homebush Bay is +0.6 m AHD to -0.4 m AHD a tailwater level of 1m AHD is relatively high. The maximum ocean tide in a year is 1.1 mAHD.

7.4. Design Results

The results from this study are presented as:

- Peak flood level profiles in Figure 16;
- Peak flood depths and level contours in Figure 17;
- Peak flood velocities in Figure 18;
- Provisional hydraulic hazard in Figure 19; and
- Provisional hydraulic categorisation in Figure 20.

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

7.4.1. Summary of Results

Peak flood levels, depths and flows at key locations within the catchment are summarised in Table 23, Table 24 and Table 25 for design events. These key locations coincide with the key locations used for the sensitivity analysis discussed in Section 9 and are shown on Figure 12.

Table 23: Peak Flood Levels (m AHD) at Key Locations – Design Events

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H01	Open Channel Upstream of Underwood Road	1.49	1.62	1.73	1.87	2.02	2.13	2.21	2.31	3.72
H02	Park Road	3.45	3.47	3.50	3.53	3.55	3.57	3.61	3.65	5.01
H03	Parramatta Road	3.32	3.55	3.65	3.76	3.87	3.96	4.04	4.15	5.25
H04	The Crescent	6.97	7.31	7.46	7.66	7.83	7.99	8.13	8.32	11.25
H05	Allan Davidson	9.38	9.39	9.40	9.41	9.43	9.53	9.62	9.69	11.26
H06	Arthur Street	13.10	13.15	13.17	13.21	13.24	13.27	13.30	13.34	13.88
H07	Open Channel Upstream of Pomeroy Street	1.80	2.07	2.21	2.44	2.53	2.61	2.67	2.78	4.02
H08	Beresford Road	15.25	15.27	15.29	15.32	15.34	15.36	15.39	15.43	15.79
H09	Pilgrim Avenue	9.18	9.28	9.33	9.38	9.43	9.47	9.51	9.56	12.11
H10	Brunswick Avenue	16.17	16.28	16.33	16.40	16.44	16.49	16.54	16.59	17.10
H11	Redmyre Road	12.36	12.47	12.53	12.61	12.70	12.84	12.94	13.08	14.36
H12	Torrington Road	27.48	27.48	27.48	27.48	27.49	27.49	27.49	27.49	27.51
H13	Morwick Street	13.53	13.70	13.77	13.86	13.91	13.98	14.03	14.10	15.08
H14	Russell Street	14.89	15.05	15.13	15.22	15.30	15.37	15.42	15.49	16.26
H15	Wentworth Road	16.22	16.43	16.51	16.59	16.65	16.71	16.76	16.82	17.53
H16	Norwood Street	17.35	17.47	17.53	17.61	17.66	17.72	17.78	17.85	18.49
H17	Woodside Avenue	19.37	19.43	19.50	19.59	19.65	19.71	19.77	19.85	20.38
H18	Nicholson Street	21.17	21.23	21.25	21.29	21.35	21.40	21.45	21.51	22.05
H19	Belgrave Street	22.35	22.42	22.45	22.50	22.54	22.58	22.61	22.66	23.19
H20	Minna Street	23.49	23.57	23.63	23.68	23.73	23.77	23.80	23.84	24.22

Table 24: Peak Flood Depths (m) at Key Locations – Design Events

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H01	Open Channel Upstream of Underwood Road	1.16	1.29	1.40	1.53	1.68	1.79	1.88	1.98	3.38
H02	Park Road	0.15	0.17	0.20	0.23	0.25	0.28	0.31	0.35	1.71
H03	Parramatta Road	0.74	0.93	1.01	1.12	1.23	1.32	1.40	1.51	2.61
H04	The Crescent	0.24	0.58	0.72	0.92	1.09	1.25	1.40	1.58	4.52
H05	Allan Davidson	0.07	0.09	0.09	0.11	0.13	0.23	0.32	0.38	1.96
H06	Arthur Street	0.06	0.11	0.13	0.18	0.21	0.23	0.26	0.30	0.84
H07	Open Channel Upstream of Pomeroy Street	1.35	1.62	1.76	1.99	2.08	2.16	2.22	2.33	3.57
H08	Beresford Road	0.05	0.06	0.08	0.11	0.13	0.15	0.18	0.22	0.58
H09	Pilgrim Avenue	0.17	0.27	0.31	0.36	0.41	0.46	0.49	0.54	3.09
H10	Brunswick Avenue	0.46	0.57	0.62	0.68	0.73	0.77	0.83	0.88	1.39
H11	Redmyre Road	0.02	0.13	0.19	0.27	0.36	0.50	0.60	0.74	2.02
H12	Torrington Road	0.02	0.02	0.02	0.03	0.03	0.03	0.03	0.03	0.06
H13	Morwick Street	0.19	0.36	0.43	0.51	0.57	0.64	0.69	0.76	1.74
H14	Russell Street	0.11	0.27	0.35	0.45	0.52	0.59	0.64	0.71	1.48
H15	Wentworth Road	0.49	0.71	0.79	0.87	0.93	0.99	1.03	1.10	1.81
H16	Norwood Street	0.10	0.23	0.28	0.36	0.42	0.47	0.53	0.61	1.24
H17	Woodside Avenue	0.03	0.09	0.16	0.25	0.31	0.37	0.43	0.51	1.04
H18	Nicholson Street	0.06	0.12	0.14	0.18	0.24	0.29	0.34	0.40	0.94
H19	Belgrave Street	0.19	0.26	0.29	0.34	0.38	0.42	0.45	0.50	1.03
H20	Minna Street	0.19	0.27	0.32	0.38	0.42	0.46	0.50	0.54	0.91

Table 25: Peak Flows (m3/s) at Key Locations – Design Events

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Q01	Underwood Road	21.00	29.01	33.61	39.12	44.56	50.84	56.08	63.00	165.62
Q02	Park Road	17.47	24.39	27.95	32.33	36.55	40.26	43.97	48.93	117.70
Q03	Parramatta Road	17.59	26.46	31.73	39.08	46.73	53.51	59.55	71.79	250.72
Q04	The Crescent	13.45	19.80	21.32	23.52	25.88	27.66	29.68	32.18	59.92
Q05	Allan Davidson	10.29	14.88	17.85	21.78	23.66	26.57	30.69	37.86	147.25
Q06	Arthur Street	2.67	4.37	5.42	6.99	8.42	9.93	11.39	13.53	48.88
Q07	Pomeroy Street	29.48	42.52	49.63	64.59	73.99	84.32	93.76	109.61	396.62
Q08	Beresford Road	2.54	3.64	4.51	5.65	6.63	7.52	8.68	10.41	31.42
Q09	Pilgrim Avenue	2.67	8.14	12.37	18.21	24.04	29.62	35.01	43.30	200.52
Q10	Brunswick Avenue	2.67	6.82	8.09	10.48	12.37	14.54	17.25	20.07	58.44
Q11	Redmyre Road	1.88	5.10	7.32	10.36	13.35	16.29	19.50	23.64	110.73
Q12	Torrington Road	0.17	0.23	0.27	0.31	0.35	0.39	0.44	0.49	1.10
Q13	Morwick Street	1.51	5.75	8.23	11.82	14.93	18.82	22.13	27.24	113.79
Q14	Russell Street	1.36	4.98	7.64	10.91	14.00	17.29	20.61	25.52	102.96
Q15	Wentworth Road	2.80	6.18	8.47	11.51	14.33	17.41	20.27	24.52	89.37
Q16	Norwood Street	3.45	8.21	9.49	11.56	14.09	17.14	20.04	24.19	82.18
Q17	Woodside Avenue	2.21	3.30	4.57	6.91	8.83	11.44	13.58	17.40	64.57
Q18	Nicholson Street	0.43	1.92	2.86	4.19	5.75	7.38	9.10	11.44	39.63
Q19	Belgrave Street	1.42	2.87	4.27	5.53	6.77	8.30	9.47	11.13	34.83
Q20	Minna Street	1.79	3.71	4.32	5.33	6.26	7.43	8.52	9.81	30.02

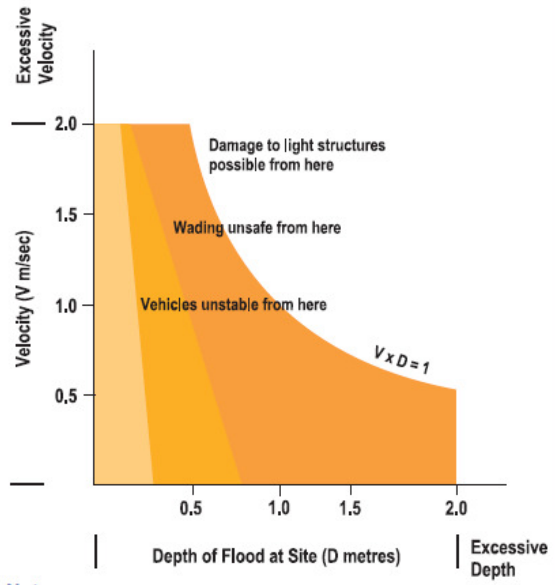
7.4.2. Duration of Inundation

Duration of flooding has also been mapped on Figure 22, which shows the duration for which different locations have greater than 0.3 m of depth in the 1% AEP 2 hour event. The figure shows that for this duration, the majority of the inundation is drained quickly, typically in less than 30 minutes. Although the mapped data is for a design event with an idealised temporal pattern and duration, the results are useful as giving indicative values of duration and for showing areas where flooding is more prolonged relative to the wider catchment.

7.4.3. Provisional Flood Hazard Categorisation

Hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual (Reference 1), the relevant section of which is shown in Diagram 3. For the purposes of this report, the transition zone presented in Diagram 3 (L2) was considered to be high hazard.

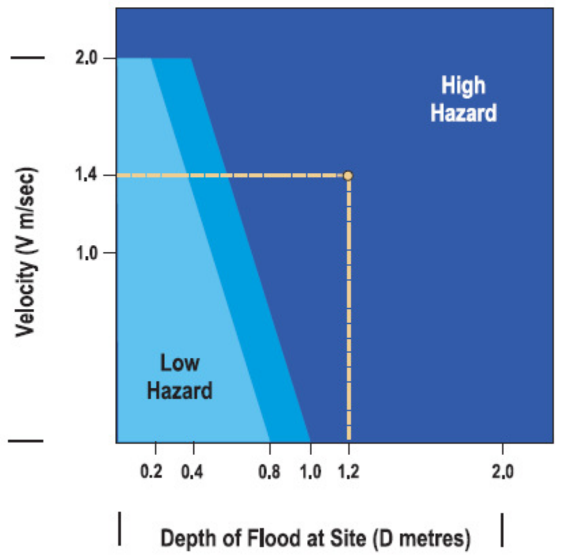
Diagram 3: (L1) Velocity and Depth Relationship; (L2) Provisional Hydraulic Hazard Categories (NSW State Government, 2005)



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**

7.4.4. Provisional Hydraulic Categorisation

The hydraulic categories, namely floodway, flood storage and flood fringe, are described in the Floodplain Development Manual (Reference 1). 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 has been adopted by consultants in a number of flood studies in NSW:

- 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 > 1 m; and
- Flood Fringe comprises areas outside the Floodway where peak depth < 1 m.

7.4.5. 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 26 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 26: 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 Powells Creek 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 study area is shown in Figure 23. The study area has limited exposure to embankments resulting in no Low/High Trapped Perimeter (LTP/HTP) areas. There is a small area classified as a High Flood Island (HFI) along The Boulevarde, where some properties are surrounded by flood water but are not inundated in the PMF event. Areas of Rising Road Access (RRA) are present on the fringes of the flood extent, particularly within the southern portion of the study area. In areas where a main flow path is present, the majority of the properties were classified as a Low Flood Island (LFI).

8. FLOOD FREQUENCY ANALYSIS

8.1. Overview

Flood frequency analysis (FFA) enables the magnitudes of floods (5%, 1% AEP etc.) to be estimated based on statistical analysis of recorded flows. It can be undertaken graphically or using a mathematical distribution.

The reliability of the flood frequency approach depends largely upon the length and quality of the observed record and accuracy of the rating curve. In addition, flood frequency inherently accounts for many assumptions which are required in rainfall-runoff routing for determining the magnitude of floods for annual exceedance probabilities.

This approach has the following advantages in design flood estimation:

- no assumptions are required regarding the relationship between probabilities of rainfall and runoff;
- all factors affecting flood magnitude are already integrated into the data;
- estimation of rainfall losses is not required;
- confidence limits can be estimated;
- historic rainfall data is not required.

The flood frequency approach does, however, have some limitations. These are:

- there is no "perfect" distribution", thus different distributions will provide different answers;
- as most flood records are relatively short (compared to the design event for which a magnitude is required) there is considerable uncertainty. Whilst rainfall records at a particular location are also short, data can be used by the BoM from other gauges to accurately estimate design intensities much greater than the period of record at a single gauge;
- changes to the local topography such as levee banks, hydraulic controls and the construction of retarding basins or bridges can affect the homogeneity of the data set;
- short to medium term climatic changes may influence the flood record; and
- there are many issues with the accuracy of rating curves, especially at high flows. However, this is less of an issue with the use of hydraulic models based on high quality survey (ALS) to obtain site rating curves.

While some of these factors can affect the quality of the flood frequency analysis, for the purpose of providing confirmation for the runoff routing results they are considered reasonable.

8.2. Examined Annual Series

Utilising the data presented in Table 16, various data sets of annual maximum levels are available for converting to flows for the purpose of FFA. These levels can be converted into flows using one of the rating curves described in Section 3.3.2 and presented in Figure 7. Eight potential scenarios have been evaluated for FFA with the tested combinations presented in Table 27 and described below.

Table 27: Flow (m^3/s) Data Sets Used in FFA

Year	Data Set #1*	Data Set #3**	Data Set #5	Data Set #6	Data Set #7	Data Set #8
1958	16.09	12.66	12.66	12.66	12.66	12.66
1959	49.07	54.56	54.56	54.56	54.56	53.9
1960	10.57	10.06	10.06	10.06	10.06	7.34
1961	5.87	83.07	3.47	83.07	3.47	3.47
1962	20.3	19.04	19.04	19.04	19.04	21.24
1963	32.06	35.74	35.74	35.74	35.74	37.06
1964	22.48	61.01	61.01	26.39	26.39	26.39
1965	7.16	6.01	6.01	6.01	6.01	4.38
1966	12.3	9.74	9.74	9.74	9.74	8.95
1967	14.91	13.52	13.52	13.52	13.52	11.45
1968	4.75	3.96	3.96	3.96	3.96	2.66
1969	18.39	19.89	19.89	19.89	19.89	16.4
1970	15.35	50.46	50.46	50.46	50.46	11.82
1971	10.33	27.57	27.57	27.57	27.57	7.06
1972	37.3	52.65	52.65	52.65	52.65	42.85
1973	27.12	34.82	34.82	34.82	34.82	31.63
1974	28.03	34.64	34.64	34.64	34.64	32.67
1975	16.72	15.12	15.12	15.12	15.12	13.52
1976	12.56	19.46	19.46	19.46	19.46	9.26
1977	16.31	7.76	7.76	7.76	7.76	12.86
1978	14.6	12.47	12.47	12.47	12.47	11.25
1979	12.15	9.58	9.58	9.58	9.58	8.8
1980	12.78	9.42	9.42	9.42	9.42	9.58
1981	14.58	11.56	11.56	11.56	11.56	11.25
1982	19.11	19.89	19.89	19.89	19.89	18.24
1983	21.21	24.98	24.98	24.98	24.98	24.03
1984	21.39	25.28	25.28	25.28	25.28	24.36
1985	11.92	10.06	10.06	10.06	10.06	8.65
1986	20.14	27.57	27.57	27.57	27.57	20.78
1987	11.4	8.2	8.2	8.2	8.2	8.2
1988	23.08	27.35	27.35	27.35	27.35	27.35
1989	13	9.74	9.74	9.74	9.74	9.74
1990	23.11	27.35	27.35	27.35	27.35	27.35
1991	19.21	18.63	18.63	18.63	18.63	18.63
1992	16.89	13.76	13.76	13.76	13.76	13.76
1993	22.38	26.39	26.39	26.39	26.39	26.39
1994	15.45	11.97	11.97	11.97	11.97	11.97
1995	13.41	10.23	10.23	10.23	10.23	10.23
1996	7.35	4.6	4.6	4.6	4.6	4.6
1997	6.87	4.17	4.17	4.17	4.17	4.17
Average	**17.4**	**22.1**	**20.1**	**21.3**	**19.3**	**16.7**

* Data Set #2 uses the same data as Data Set #1 however incorporates 17 additional years of data as mentioned in Section 8.2.1.
** Data Set #4 uses the same data as Data Set #3 however incorporates 17 additional years of data as mentioned in Section 8.2.1.

Digitally Record Flows

1. Entire Period of Record (EPR);
2. EPR with data from 1998 incorporated using Bayesian methods (see Section 8.2.1).

Reference 6 **Levels Converted with TUFLOW Rating**

3. EPR with missing years 1958 and 1987 - 1997 completed using the Digital Record;
4. EPR with missing years 1958 and 1987 - 1997 completed using the Digital Record with data from 1998 incorporated using Bayesian methods (see Section 8.2.1);
5. As 4. but with 1961 event stage replaced by digital record stage;
6. As 4. but with 1964 event stage replaced by digital record stage;
7. As 4. but with 1961 and 1964 events stage replaced by digital record stage.

Digital Record Stage

8. Digital record stage converted to flow using TUFLOW rating with additional data from 1998 incorporated using Bayesian methods (see Section 8.2.1).

8.2.1. Inclusion of Incomplete Data from 1998 to 2014

As mentioned in Section 2.10.1, the Elva Street gauge's digital records after November 1997 are incomplete. Accordingly data from this period cannot be used as part of an annual series for FFA purposes. However, use of Bayesian methods (an interpretation of the concept of probablity) method now allow historic events of unknown magnitude to be included into the FFA. Essentially, Bayesian methods allow events to be added above and below a threshold value for use in the analysis.

Anecdotal evidence suggests that there have been no significant flood events in the period of 1998 to 2014. As the exact magnitude of these events is unknown, an assumption has been made as to the likely maximum flow achieved in at least one of these events. This has been determined to be the average of the digital data set flows for each data set (see the last row of Table 27) which varies depending on the data set being analysed. Using Bayesian methods, data from the period of 1998 to 2014 (17 years of additional data), have been incorporated into the FFA by assuming that they have not exceeded this threshold value.

8.2.2. Adopted Data Set

FFA of the eight data sets presented in Table 27 has been performed with the results presented in Appendix B. Further examination of the data sets was undertaken to determine which provides the most reasonable representation of annual maximum flows for FFA. This analysis is presented below.

Data Sets #1 and #3

With new computational technology and Bayesian statistical methods it seems appropriate that the incomplete data from 1998 to 2014 (see Section 8.2.1) be incorporated into the FFA. Accordingly, Data Sets #1 and #3 have been discounted from further consideration as Data Sets #2 and #4 use the same annual series but incorporate this additional data.

It is interesting to compare design flow results from Data Set #1 to the Reference 2 FFA results as FFA has been performed on the same data set and should therefore produce similar results. Table 28 presents this flow comparison and shows that as expected the results are similar. Minor differences in the larger design flows are due to differences in the applied statistical distribution fitting method.

Table 28: Comparison of Reference 2 and Data Set #1 FFA Design Flows

AEP (%)	Reference 2 Design Flows (m^3/s)	Data Set #1 Design Flows (m^3/s)
20	23.8	24.0
10	29.3	29.7
5	34.6	35.2
2	41.7	42.6
1	47.2	48.2

Data Set #2

Due to the uncertainties associated with the digital record rating curve described in Section 3.3.2, the TUFLOW model rating is preferred for converting levels to flows. As Data Set #2 uses the digital record rating curve this data set has been discounted.

Data Sets #4, #5 and #6

As presented in Table 11, a number of events used debris marks to estimate peak flood level. In particular, the 1961 and 1964 events. Reference 6 levels were obtained from flood marks as no digital gauge data was available. Peak flood marks obtained from reported debris are considered less reliable than those recorded by the gauge. To test the veracity of the magnitude of the 1961 and 1964 events, daily and pluviometer rainfall data for these events were examined for proximate gauges. It should be noted that at the time of these events rainfall data and particularly sub-daily rainfall data (pluviometer) was sparse.

This analysis could not confirm the estimated magnitude of these events as the estimated rainfall AEP was generally much less than the estimated flow AEP for these events. This led to the 1961 and 1964 events being discounted for FFA purposes. The 1959 event peak flood level was also estimated from debris however the reported Reference 2 and Reference 6 peak gauge heights are effectively the same giving credit to the true magnitude of this event.

In light of these findings, Data Sets #4, #5 and #6 have been excluded from the design flow estimates.

Data Set #7 and #8

FFA results of Data Sets #7 and #8 were analysed to determine which of these two data sets should be used to produce design flows. Two variables were examined, namely, the goodness of fit of the:

- Annual series data to the distribution; and
- TUFLOW model design flows to the distribution.

Appendix B, Figures B7 and B8 presents the annual series, distributions and TUFLOW model design flows for both Data Sets. For both examined variables, Data Set #8 displayed better correlation than Data Set #7 and was thus selected in preference.

8.3. Probability Distribution

AR&R (Reference 4) recommends that FFA should be applied to peak flows rather than heights. In frequency analysis of flows, the fitting of a particular distribution may be carried out analytically or by fitting a probability distribution. The data may consist of an annual series, where the largest peak in each year is used, or a partial series, where all flows above a selected base value are used. The relative merits of each method are discussed in detail in AR&R. In general, an annual series is preferable as there are more methods and experience available.

Many probability distributions have been applied to FFA and this is a very active field of research. However, it is not possible to determine the "correct" form of the distribution as there is no robust evidence that any particular distribution is more appropriate than another. AR&R provides further discussion on this issue.

Since publication of AR&R (Reference 4) in 1987 there have been significant developments in the field of FFA both in Australia and overseas. The approach adopted in this study reflects these developments. Recent research has suggested that the fitting method is as important as the adopted distribution. The traditional fitting method has generally been based on moments and this makes the fit very sensitive to the highest and lowest values. Recent research has shown that L-moment and Bayesian likelihood approaches are much more robust than traditional moment fitting and are now the recommended methods.

For this analysis a Bayesian maximum likelihood approach has been adopted in preference to L-moments because the method readily lends itself to include limited information about events outside the continuous period of record. The Flike flood frequency analysis software developed by Kuczera (Reference 10) uses the Bayesian approach and was utilised in this study.

The rating curve (height-discharge relationship) adopted for the estimation of streamflows from the recorded gauge heights is critical to the success of FFA. The FFA was conducted using the rating curve derived from the calibrated hydraulic model (refer subsequent sections) as well as that obtained from the digital records (see Section 3.3.2).

Two probability distributions were tested, Log Pearson III (LP3) and Generalised Extreme Value (GEV) distributions and it was found that the LP3 distribution produced a better curve fit to the data.

8.4. Design Flow Results

The results of the FFA are provided in Table 29 and shown on Figure 21 for the LP3 distribution. The choice of distribution was found to have some influence on design flow estimates. It was found that the LP3 distribution fit the annual series data better than the GEV distribution and was therefore selected in preference for determining design flows.

Table 29: Flood Frequency Analysis – Powells Creek Elva Street gauge

Design Flood Event	Peak Flow FFA (m^3/s)	
	LP3 Distribution	GEV Distribution
0.5 (1 in 2 year) EY	11.4	11.6
0.2 (1 in 5 year) EY	20.9	20.5
10% (1 in 10 year) AEP	28.5	28.1
5% (1 in 20 year) AEP	36.9	37.1
2% (1 in 50 year) AEP	49.1	51.7
1% (1 in 100 year) AEP	59.5	65.4

8.5. Reconciling Flood Frequency and Rainfall Runoff Results

When compared to FFA design flow estimates those from TUFLOW (Figure 21) overestimate flows for more frequent events and underestimate flow in the 2% AEP event or greater.

There are many explanations as to why the flood frequency and rainfall runoff modelling do not reconcile. These are primarily due to data limitations as well as the adequacy of the hydrologic model in representing the runoff routing behaviour of the catchment. Some of the main limitations of the FFA are the limited period of record as well as rating curve errors. Due to the nature of the rating curve, high flow estimates at the Elva Street gauge are very sensitive to small changes in the water level.

In addition to potential uncertainty of the analysis it is important to realise that the flood frequency relationship may not be representative of the greater Powells Creek catchment given that the Elva Street catchment only covers a proportion of the catchment.

As FFA estimates become more uncertain for less frequent flooding such as the 1% AEP which is generally adopted for development control purposes, flow estimates from TUFLOW modelling were adopted for the current study.

9. SENSITIVITY ANALYSIS

9.1. 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:

- 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;
- 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 (elevated levels in the Parramatta River) of 0.4 m and 0.9 m were assessed.

These sensitivity scenarios were undertaken for the 1% AEP rainfall event with a tailwater level of 1 mAHD in the Parramatta River.

9.2. 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.

9.2.1. Rainfall Increase

The BoM has indicated that there is no intention at present to revise design rainfalls to take account of the potential climate change, as the implications of temperature changes on extreme rainfall intensities are presently unclear, and there is no certainty that the changes would in fact increase design rainfalls for major flood producing storms. There is some recent literature by CSIRO that suggests extreme rainfalls may increase by up to 30% in parts of NSW (in other places the projected increases are much less or even decrease); however, this information is not of sufficient accuracy for use as yet (Reference 11).

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 index. Although mean daily rainfall intensity is not observed to differ significantly between Indian Pacific Oscillation 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 Powells Creek catchment under warmer climate scenarios.

In light of this uncertainty, the NSW State Government (Reference 11) 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.

9.2.2. Sea Level Rise

The *NSW Sea Level Rise Policy Statement* was released by the NSW Government in October 2009 (Reference 12). This Policy Statement was accompanied by the *Derivation of the NSW Government's sea level rise planning benchmarks* (Reference 13) 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* (Reference 14).

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"* (Reference 13).

In light of this uncertainty, the NSW State Government's advice is subject to periodical review. As of October 2012 the NSW State Government withdrew endorsement of sea level rise predictions but still require sea level rise to be considered. This was taken as a 0.4 m rise by the year 2050 and a 0.9 m rise by the year 2100.

9.3. Results

The sensitivity scenario results were compared to the 1% AEP rainfall event and a summary of peak flood level and peak flow differences at various locations are provided in the sections below.

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.

9.3.1. Roughness Variations

Overall peak flood level results were shown to be relatively insensitive to variations in the roughness parameter. Generally, these results were found to be within ± 0.1 m.

Table 30: Results of Roughness Variation – Change in Level

ID	Location	Peak Flood Depth 1% AEP	Difference with 1% AEP (m)	
			Decrease roughness by 20%	Increase roughness by 20%
H01	Open Channel Upstream of Underwood Road	1.16	-0.11	0.07
H02	Park Road	0.15	0.01	0.01
H03	Parramatta Road	0.74	0.04	-0.07
H04	The Crescent	0.24	0.02	0.11
H05	Allan Davidson	0.07	0.05	0.07
H06	Arthur Street	0.06	0.01	0.02
H07	Open Channel Upstream of Pomeroy Street	1.35	-0.05	0.04
H08	Beresford Road	0.05	-0.01	0.02
H09	Pilgrim Avenue	0.17	-0.02	0.02
H10	Brunswick Avenue	0.46	-0.02	0.03
H11	Redmyre Road	0.02	0.03	-0.02
H12	Torrington Road	0.02	0.00	0.00
H13	Morwick Street	0.19	0.00	0.01
H14	Russell Street	0.11	0.01	-0.01
H15	Wentworth Road	0.49	0.00	0.00
H16	Norwood Street	0.10	-0.01	0.00
H17	Woodside Avenue	0.03	0.01	-0.01
H18	Nicholson Street	0.06	-0.02	0.00
H19	Belgrave Street	0.19	0.00	0.00
H20	Minna Street	0.19	0.00	0.00

Table 31: Results of Roughness Variation – Change in Flow

ID	Location	Peak Flow 1% AEP	Difference with 1% AEP (m^3/s)	
			Decrease roughness by 20%	Increase roughness by 20%
Q01	Underwood Road	50.84	1.32	-5.02
Q02	Park Road	40.26	1.32	-3.93
Q03	Parramatta Road	53.51	1.81	-0.82
Q04	The Crescent	27.66	0.66	-2.02
Q05	Allan Davidson	26.57	0.98	-0.08
Q06	Arthur Street	9.93	0.53	-0.37
Q07	Pomeroy Street	84.32	-1.12	-2.14
Q08	Beresford Road	7.52	0.81	-0.46
Q09	Pilgrim Avenue	29.62	1.13	-1.03
Q10	Brunswick Avenue	14.54	0.56	-0.03
Q11	Redmyre Road	16.29	1.15	-0.82
Q12	Torrington Road	0.39	0.09	0.00
Q13	Morwick Street	18.82	1.41	-0.62
Q14	Russell Street	17.29	1.11	-0.30
Q15	Wentworth Road	17.41	1.01	-0.74
Q16	Norwood Street	17.14	0.90	-0.56
Q17	Woodside Avenue	11.44	0.85	-0.39
Q18	Nicholson Street	7.38	0.11	-0.24
Q19	Belgrave Street	8.30	0.39	-0.31
Q20	Minna Street	7.43	0.33	-0.48

9.3.2. Blockage Variations

Peak flood level results were found to be relatively insensitive to blockage of pipes; although generally peak flood levels increased in the upstream areas and decreased in the downstream areas (due to the retarding effect in the upstream areas). The two locations where peak flood level increases were recorded were Redmyre Road and Woodside Avenue.

Woodside Avenue is located at the confluence of two flow paths from the south-east and south-west; with both inflows and outflows serviced by SWC major drainage lines. The buildings crossing the overland flow path downstream of the roadway constrict the overland flow path exiting Woodside Avenue, resulting in accumulation of flood waters and increased flood levels. The flow accumulation that occurred at Woodside Avenue resulted in less pronounced increased peak flood levels at downstream locations such as Norwood Street and Wentworth Road.

Redmyre Road is highly dependent on the pipe network as the buildings downstream (Strathfield Plaza and other commercial premises) are highly constrictive to overland flow. The location is subject to a complex collection of pipes operated by SWC, Burwood Council and Strathfield Municipal Council. With a large collection of pipes, the Redmyre Road location was more sensitive to blockage of the pipe network.

Table 32: Results of Blockage Variation – Change in Level

ID	Location	Peak Flood Depth 1% AEP	Difference with 1% AEP (m)	
			Pipe Blockage of 20%	Pipe Blockage of 50%
H01	Open Channel Upstream of Underwood Road	1.16	-0.02	-0.05
H02	Park Road	0.15	0.01	0.01
H03	Parramatta Road	0.74	-0.01	-0.02
H04	The Crescent	0.24	-0.02	-0.06
H05	Allan Davidson	0.07	-0.01	-0.03
H06	Arthur Street	0.06	0.01	0.04
H07	Open Channel Upstream of Pomeroy Street	1.35	-0.02	-0.05
H08	Beresford Road	0.05	0.01	0.03
H09	Pilgrim Avenue	0.17	0.01	0.03
H10	Brunswick Avenue	0.46	0.02	0.03
H11	Redmyre Road	0.02	0.06	0.17
H12	Torrington Road	0.02	0.00	0.00
H13	Morwick Street	0.19	0.02	0.06
H14	Russell Street	0.11	0.02	0.08
H15	Wentworth Road	0.49	0.01	0.05
H16	Norwood Street	0.10	-0.01	0.00
H17	Woodside Avenue	0.03	0.05	0.13
H18	Nicholson Street	0.06	0.03	0.09
H19	Belgrave Street	0.19	0.03	0.07
H20	Minna Street	0.19	0.02	0.05

Table 33: Results of Blockage Variation – Change in Flow

ID	Location	Peak Flow 1% AEP	Difference with 1% AEP (m^3/s)	
			Pipe Blockage of 20%	Pipe Blockage of 50%
Q01	Underwood Road	50.84	-0.47	-0.76
Q02	Park Road	40.26	0.13	-0.34
Q03	Parramatta Road	53.51	-1.16	-3.99
Q04	The Crescent	27.66	0.00	-0.69
Q05	Allan Davidson	26.57	0.22	0.95
Q06	Arthur Street	9.93	0.27	0.78
Q07	Pomeroy Street	84.32	-2.17	-6.63
Q08	Beresford Road	7.52	0.36	1.01
Q09	Pilgrim Avenue	29.62	2.57	4.89
Q10	Brunswick Avenue	14.54	0.55	1.13
Q11	Redmyre Road	16.29	1.51	4.10
Q12	Torrington Road	0.39	0.00	0.00
Q13	Morwick Street	18.82	1.10	3.54
Q14	Russell Street	17.29	0.99	5.29
Q15	Wentworth Road	17.41	0.69	2.97
Q16	Norwood Street	17.14	0.56	2.20
Q17	Woodside Avenue	11.44	1.43	3.86
Q18	Nicholson Street	7.38	1.11	2.36
Q19	Belgrave Street	8.30	0.35	0.97
Q20	Minna Street	7.43	0.42	0.87

9.3.3. Sea Level Rise Variations

The sea level rise scenarios were found to have an insignificant effect on peak flood levels, except in the most downstream reaches of the catchment. The open channel upstream of Underwood Road and Pomeroy Street had channel inverts of 0.35 m AHD and 0.45 m AHD (respectively) and were therefore tidally affected under current tidal conditions. Under sea level rise conditions, these locations were found to have increased peak flood levels; although the increase in peak flood level was found to be diffused, such that a 0.9 m increase in sea levels resulted in a lesser flood level increase of 0.25 m. The attenuation of sea level rise impacts was found to be the result of the retarding effect of the downstream mangroves and the restrictive effect of bridge structures crossing the open channel.

Table 34: Results of Sea Level Rise – Change in Level

ID	Location	Peak Flood Depth 1% AEP	Difference with 1% AEP (m)	
			Tailwater increase to 1.4 m AHD	Tailwater increase to 1.9 m AHD
H01	Open Channel Upstream of Underwood Road	1.16	0.08	0.25
H02	Park Road	0.15	0.00	0.00
H03	Parramatta Road	0.74	0.00	0.00
H04	The Crescent	0.24	0.00	0.00
H05	Allan Davidson	0.07	0.00	0.00
H06	Arthur Street	0.06	0.00	0.00
H07	Open Channel Upstream of Pomeroy Street	1.35	0.03	0.10
H08	Beresford Road	0.05	0.00	0.00
H09	Pilgrim Avenue	0.17	0.00	0.00
H10	Brunswick Avenue	0.46	0.00	0.00
H11	Redmyre Road	0.02	0.00	0.00
H12	Torrington Road	0.02	0.00	0.00
H13	Morwick Street	0.19	0.00	0.00
H14	Russell Street	0.11	0.00	0.00
H15	Wentworth Road	0.49	0.00	0.00
H16	Norwood Street	0.10	0.00	0.00
H17	Woodside Avenue	0.03	0.00	0.00
H18	Nicholson Street	0.06	0.00	0.00
H19	Belgrave Street	0.19	0.00	0.00
H20	Minna Street	0.19	0.00	0.00

Table 35: Results of Sea Level Rise – Change in Flow

ID	Location	Peak Flow 1% AEP	Difference with 1% AEP (m^3/s)	
			Tailwater increase to 1.4 m AHD	Tailwater increase to 1.9 m AHD
Q01	Underwood Road	50.84	-0.44	-0.98
Q02	Park Road	40.26	-0.04	0.22
Q03	Parramatta Road	53.51	0.09	0.11
Q04	The Crescent	27.66	0.10	0.00
Q05	Allan Davidson	26.57	0.00	0.00
Q06	Arthur Street	9.93	0.00	0.00
Q07	Pomeroy Street	84.32	0.54	0.90
Q08	Beresford Road	7.52	0.00	0.00
Q09	Pilgrim Avenue	29.62	0.04	0.04
Q10	Brunswick Avenue	14.54	0.00	0.08
Q11	Redmyre Road	16.29	0.09	0.10
Q12	Torrington Road	0.39	0.00	0.00
Q13	Morwick Street	18.82	0.09	0.08
Q14	Russell Street	17.29	0.08	0.04
Q15	Wentworth Road	17.41	0.07	0.11
Q16	Norwood Street	17.14	0.08	0.12
Q17	Woodside Avenue	11.44	0.16	0.18
Q18	Nicholson Street	7.38	0.00	0.00
Q19	Belgrave Street	8.30	0.00	0.00
Q20	Minna Street	7.43	-0.06	-0.06

9.3.4. Rainfall Variations

The effect of increasing the design rainfalls by 10%, 20% and 30% have been evaluated for the 1% AEP rainfall event with impacts on peak flood levels observed throughout the study area (shown in Table 36). Generally speaking, each incremental 10% increase in rainfall results in an approximately 0.05 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 rainfall conditions and a significant impact on flood levels is not unexpected.

Table 36: Results of Rainfall Increase – Change in Level

ID	Location	Peak Flood Depth 1% AEP	Difference with 1% AEP (m)		
			Increase in rainfall by 10%	Increase in rainfall by 20%	Increase in rainfall by 30%
H01	Open Channel Upstream of Underwood Road	1.16	0.08	0.16	0.23
H02	Park Road	0.15	0.02	0.04	0.07
H03	Parramatta Road	0.74	0.06	0.12	0.20
H04	The Crescent	0.24	0.09	0.23	0.37
H05	Allan Davidson	0.07	0.02	0.10	0.14
H06	Arthur Street	0.06	-0.01	0.01	0.05
H07	Open Channel Upstream of Pomeroy Street	1.35	0.06	0.14	0.21
H08	Beresford Road	0.05	0.00	0.03	0.06
H09	Pilgrim Avenue	0.17	0.03	0.07	0.11
H10	Brunswick Avenue	0.46	0.03	0.07	0.11
H11	Redmyre Road	0.02	0.11	0.20	0.29
H12	Torrington Road	0.02	0.00	0.00	0.00
H13	Morwick Street	0.19	0.05	0.10	0.14
H14	Russell Street	0.11	0.05	0.10	0.15
H15	Wentworth Road	0.49	0.04	0.08	0.13
H16	Norwood Street	0.10	0.04	0.09	0.14
H17	Woodside Avenue	0.03	0.04	0.09	0.15
H18	Nicholson Street	0.06	0.01	0.06	0.10
H19	Belgrave Street	0.19	0.00	0.04	0.08
H20	Minna Street	0.19	0.01	0.04	0.07

Table 37: Results of Rainfall Increase – Change in Flow

ID	Location	Peak Flow 1% AEP	Difference with 1% AEP (m^3/s)		
			Increase in rainfall by 10%	Increase in rainfall by 20%	Increase in rainfall by 30%
Q01	Underwood Road	50.84	1.62	6.46	10.99
Q02	Park Road	40.26	2.43	5.68	8.94
Q03	Parramatta Road	53.51	5.91	13.56	21.50
Q04	The Crescent	27.66	1.19	3.34	5.26
Q05	Allan Davidson	26.57	0.57	5.63	9.78
Q06	Arthur Street	9.93	-0.55	0.89	2.28
Q07	Pomeroy Street	84.32	9.09	19.63	32.70
Q08	Beresford Road	7.52	0.29	1.19	2.39
Q09	Pilgrim Avenue	29.62	5.27	11.11	17.49
Q10	Brunswick Avenue	14.54	1.23	3.44	5.44
Q11	Redmyre Road	16.29	3.26	5.90	9.34
Q12	Torrington Road	0.39	-0.02	0.02	0.06
Q13	Morwick Street	18.82	3.35	6.73	10.50
Q14	Russell Street	17.29	3.03	6.00	10.20
Q15	Wentworth Road	17.41	2.21	4.93	8.08
Q16	Norwood Street	17.14	2.06	4.41	7.76
Q17	Woodside Avenue	11.44	1.43	3.38	6.03
Q18	Nicholson Street	7.38	0.40	2.19	3.78
Q19	Belgrave Street	8.30	0.27	1.26	2.47
Q20	Minna Street	7.43	0.33	1.07	2.14

10. PRELIMINARY FLOOD PLANNING AREAS

10.1. 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 via Section 149(2) notifications under the 1979 EP&A Act. 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.

The FPA is classified as 'provisional' as it is based on results from the current study, and may be re-assessed as part of a floodplain risk management study for the catchment. Such a study would review the area's existing planning policies with respect to floodplain management, and then make recommendations (including adoption of a Flood Planning Area and Flood Planning Level) via a floodplain risk management plan. It may also be that the same assessment for the LGA's other catchments be undertaken so that a single LGA-wide FPA/FPL can be adopted.

10.2. 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 Powells Creek 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.

Furthermore, a “ground truthing” exercise was undertaken to ensure that the properties identified as subject to flood related development controls were located within a continuous flow path area.

10.3. Results

The provisional FPA is shown in Figure 24. The mainstream flood affectation was limited to the Strathfield LGA (not reported herein); with only overland flood affectation within the Burwood LGA portion of the Powells Creek Catchment.

A total of 212 properties were identified for flood related development controls in Burwood. This results in total averages of 1.6 properties per hectare for the Burwood LGA portion of the Powells Creek Catchment.

Properties that are not identified as part of this process may not be excluded from development controls. It is advisable that new developments (regardless of whether they are identified 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.

11. HOTSPOT DISCUSSION

Hotspots in the area are defined as those locations where there is a known flood issue. They are identified by considering accounts of previous floods, and by examining the flood behaviour. The latter involves identifying areas of high hazard flow where flooding of property occurs, where inundation of main roads occurs and through consideration of subsurface drainage capacity. As described in Section 2, the catchment has a history of flooding and is well understood through both the community's experience and the hydraulic model results.

11.1. Minna Street to Norwood Street

From the Minna Street – Bold Street intersection to the Norwood Street – Oxford Street intersection, there is a natural depression that results in flow occurring in a north-westerly direction. This flow often occurs perpendicular to the roadway alignment and through private property. Flood risk arises from over-floor flooding and risk to pedestrians and vehicles on road crossings.

The peak flood depths and levels across this location are shown in Table 38 and Table 39. The 5% AEP and 1% AEP peak flood depths and level contours are shown on Figure C 1 and Figure C 2.

Table 38: Minna Street – Peak Flood Levels (m AHD)

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H16	Norwood Street	17.35	17.47	17.53	17.61	17.66	17.72	17.78	17.85	18.49
H17	Woodside Avenue	19.37	19.43	19.50	19.59	19.65	19.71	19.77	19.85	20.38
H18	Nicholson Street	21.17	21.23	21.25	21.29	21.35	21.40	21.45	21.51	22.05
H19	Belgrave Street	22.35	22.42	22.45	22.50	22.54	22.58	22.61	22.66	23.19
H20	Minna Street	23.49	23.57	23.63	23.68	23.73	23.77	23.80	23.84	24.22

Table 39: Minna Street – Peak Flood Depths (m)

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H16	Norwood Street	0.10	0.23	0.28	0.36	0.42	0.47	0.53	0.61	1.24
H17	Woodside Avenue	0.03	0.09	0.16	0.25	0.31	0.37	0.43	0.51	1.04
H18	Nicholson Street	0.06	0.12	0.14	0.18	0.24	0.29	0.34	0.40	0.94
H19	Belgrave Street	0.19	0.26	0.29	0.34	0.38	0.42	0.45	0.50	1.03
H20	Minna Street	0.19	0.27	0.32	0.38	0.42	0.46	0.50	0.54	0.91

Flooding is likely to be short duration (less than one hour) but occur with little to no warning. This is shown in the flood level hydrographs on Figure C 3.

The majority of the hotspot has low hazard flow, with high hazard limited to the centre of the flow path in the hotspot and likely to occur where flow is forced through gaps between buildings.

The peak flood flows across this location are shown in Table 40.

Table 40: Minna Street – Peak Flows (m^3/s)

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Q16	Norwood Street	3.45	8.21	9.49	11.56	14.09	17.14	20.04	24.19	82.18
Q17	Woodside Avenue	2.21	3.30	4.57	6.91	8.83	11.44	13.58	17.40	64.57
Q18	Nicholson Street	0.43	1.92	2.86	4.19	5.75	7.38	9.10	11.44	39.63
Q19	Belgrave Street	1.42	2.87	4.27	5.53	6.77	8.30	9.47	11.13	34.83
Q20	Minna Street	1.79	3.71	4.32	5.33	6.26	7.43	8.52	9.81	30.02

11.2. Wentworth Road

The intersection of Wentworth Road and Hornsey Street is a topographical low point. The upstream flow path originates from the south-east and has a contributing catchment area of approximately 84 ha.

Located to the west of Wentworth Road, the sporting field for Santa Sabina College is bounded by a ridge along the Wentworth Road and northern boundaries that in some locations is 1.5 m higher than the road elevation. The only two means for flow to enter the sporting field from Wentworth Road is for flooding to backwater up to the south-east corner of the sporting field (where the roadway elevation and sporting field elevation is approximately equal and no ridge is present) or for the ridge to be overtopped. As such, the sporting field is not inundated in events up to and including the 0.2% AEP event, although it is inundated in the PMF event.

In events up to and including the 0.2% AEP event, flow is conveyed downstream to the north-west of Wentworth Road via stormwater pipes and overland between Hornsey Street and Russell Street. The egress overland flow path is constricted by buildings and therefore accumulates and backwaters along Wentworth Road.

The peak flood depths and levels at this location are shown in Table 41. The 5% AEP and 1% AEP peak flood depths and level contours are shown on Figure C 4 and Figure C 5; and the flood level hydrographs are shown on Figure C 6.

Table 41: Wentworth Road – Peak Flood Levels (m AHD) and Depths (m)

ID	Location	Type	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H15	Wentworth Road	Level (mAHD)	16.22	16.43	16.51	16.59	16.65	16.71	16.76	16.82	17.53
		Depth (m)	0.49	0.71	0.79	0.87	0.93	0.99	1.03	1.10	1.81

The peak flood flows at this location are shown in Table 42.

Table 42: Wentworth Road – Peak Flows (m^3/s)

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Q15	Wentworth Road	2.80	6.18	8.47	11.51	14.33	17.41	20.27	24.52	89.37

11.3. Russell Street and Russell Lane

Russell Lane receives flow from Wentworth Road to the south-east and discharges flow onto Russell Street to the north-west. Russell Street is at the confluence of two flow paths; from the south-east via Russell Lane and from the south-west via The Boulevarde.

Russell Lane is in a slight depression comparative to the downstream Russell Street, resulting in accumulation on Russell Lane. The lowest topographical point along Russell Street is between Wentworth Road and The Boulevarde; with ground elevation differences of 2.7 m and 4.1 m respectively. Flow discharging from Russell Street to the north is perpendicular to the roadway alignment and is constricted by the buildings.

The peak flood depths and levels at this location are shown in Table 43. The 5% AEP and 1% AEP peak flood depths and level contours are shown on Figure C 7 and Figure C 8; and the flood level hydrographs are shown on Figure C 9.

Table 43: Russell Street – Peak Flood Levels (m AHD) and Depths (m)

ID	Location	Type	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H14	Russel Street	Level (mAHD)	14.89	15.05	15.13	15.22	15.30	15.37	15.42	15.49	16.26
		Depth (m)	0.11	0.27	0.35	0.45	0.52	0.59	0.64	0.71	1.48

The peak flood flows at this location are shown in Table 44.

Table 44: Russell Street – Peak Flows (m^3/s)

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Q14	Russell Street	1.36	4.98	7.64	10.91	14.00	17.29	20.61	25.52	102.96

11.4. Morwick Street to Lyons Street

Morwick Street receives flow from Russell Street to the south-east and conveys flow through a natural depression towards the intersection of Lyons Street and The Boulevarde. This flow intersects a number of properties within this block, with notable accumulation of flow upstream of contiguous buildings between The Boulevarde and Bells Lane.

The peak flood depths and levels at this location are shown in Table 45. The 5% AEP and 1% AEP peak flood depths and level contours are shown on Figure C 10 and Figure C 11; and the flood level hydrographs are shown on Figure C 12.

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

ID	Location	Type	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H13	Morwick Street	Level (mAHD)	13.53	13.70	13.77	13.86	13.91	13.98	14.03	14.10	15.08
		Depth (m)	0.19	0.36	0.43	0.51	0.57	0.64	0.69	0.76	1.74

The peak flood flows at this location are shown in Table 46.

Table 46: Morwick Street – Peak Flows (m^3/s)

ID	Location	0.5 EY	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Q13	Morwick Street	1.51	5.75	8.23	11.82	14.93	18.82	22.13	27.24	113.79

12. ACKNOWLEDGEMENTS

Burwood Council funded this document, with most analysis being undertaken as part of a Sydney Water-funded study of the entire catchment. The assistance of the following in providing data and guidance to the study is gratefully acknowledged:

- Sydney Water;
- Strathfield Municipal Council;
- City of Canada Bay Council;
- Burwood Council;
- Bureau of Meteorology;
- Residents of the Powells Creek catchment.

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Figures

FIGURE 1
POWELLS CREEK (BURWOOD LGA) STUDY AREA

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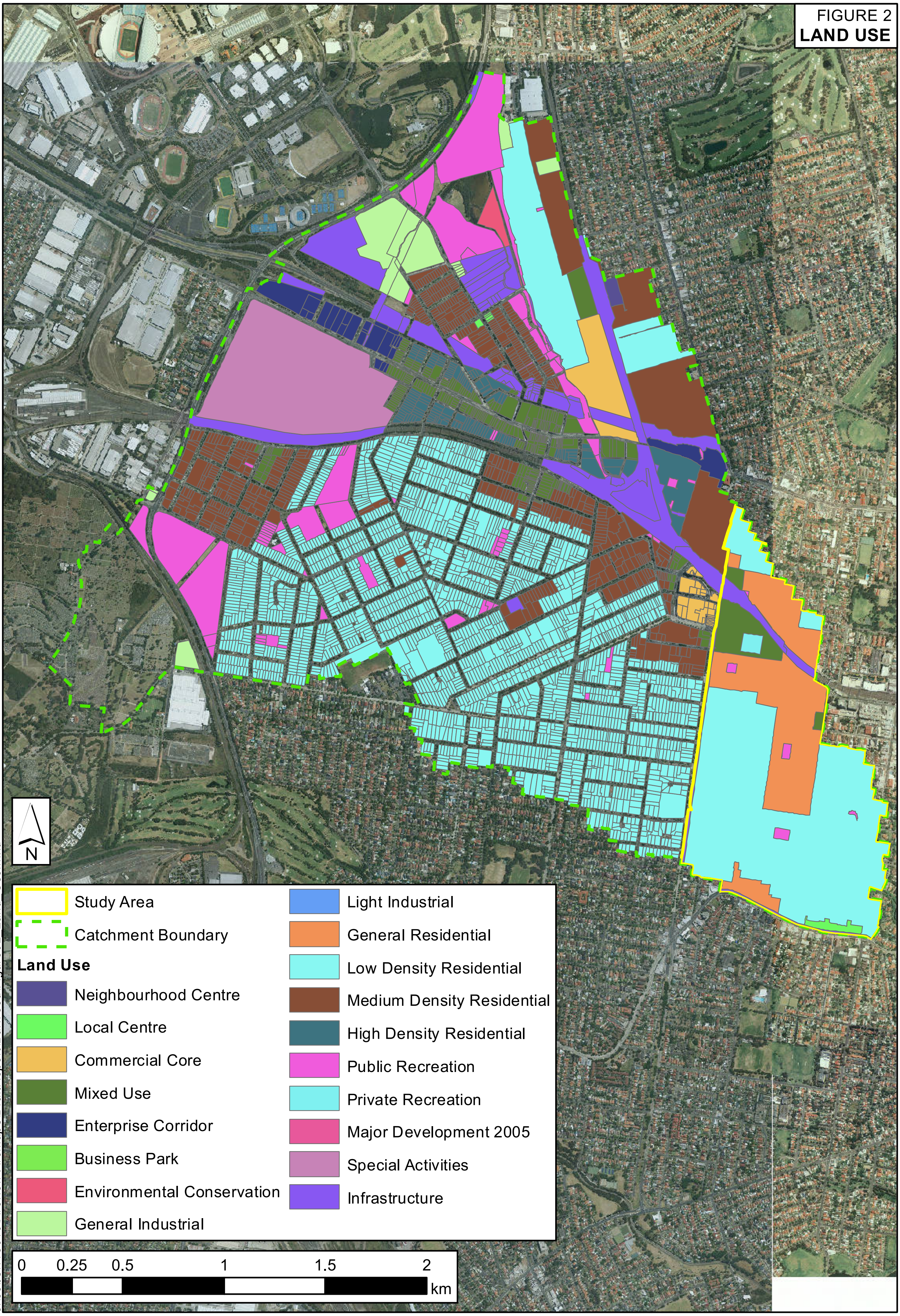

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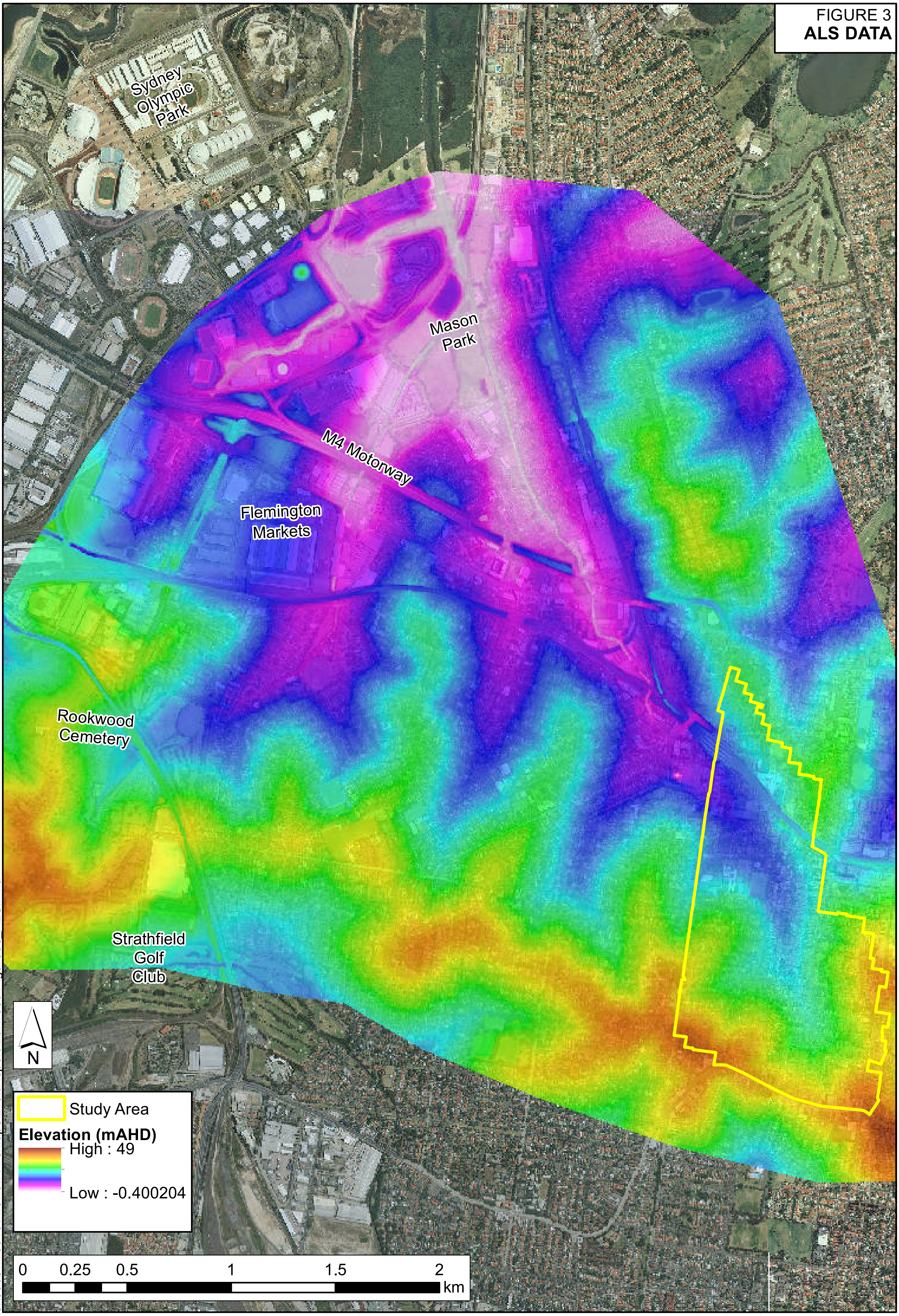
FIGURE 3
ALS DATA



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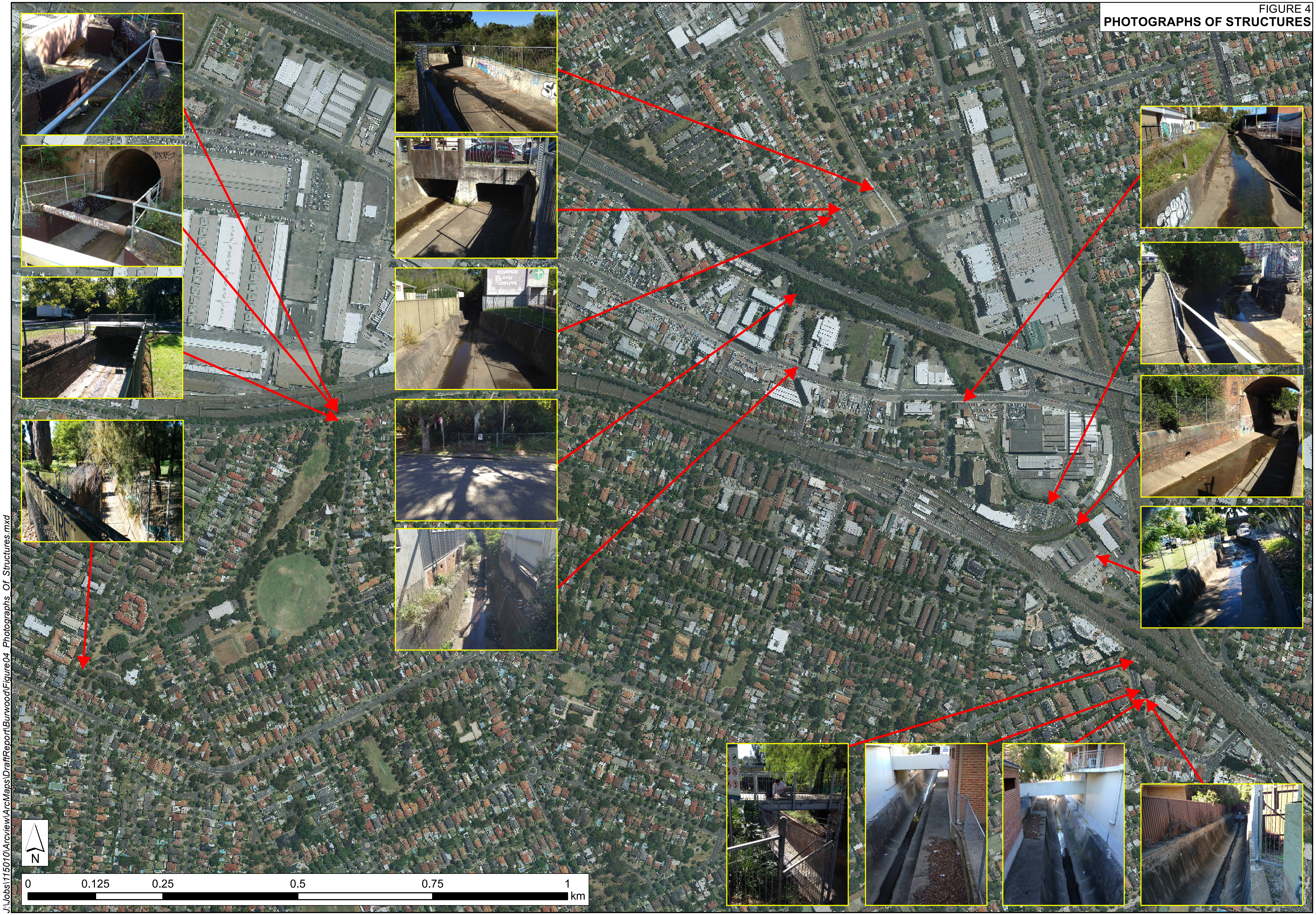

FIGURE 4
PHOTOGRAPHS OF STRUCTURES

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FIGURE 5a
HISTORICAL FLOOD PHOTOGRAPHS
SHEET 1


Airey Park 2/01/96


Airey Park 2/01/96


The Crescent opposite Airey Park 2/01/96


19 Shortland Ave 10/02/1990


Airey Park The Crescent 2/01/96


Oxford Road 2/01/96


1 Heyde Ave 2/01/96


Rochester Street eastern side, Mirabooka Ave & Broughton Rd

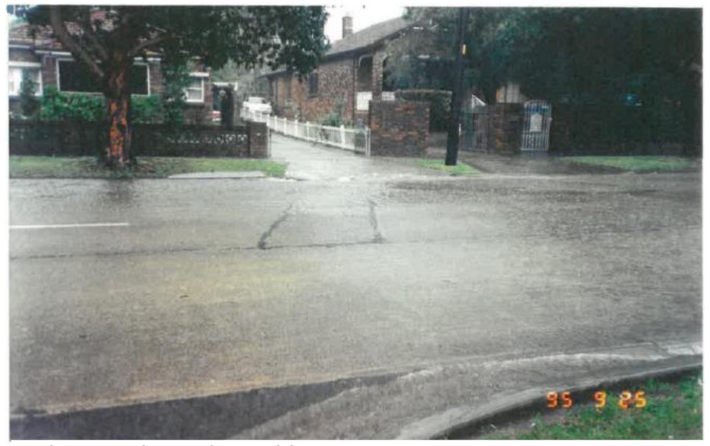

Redmyre Road 2/01/96


62 Beresford Road 10/02/1990


19 Shortland Ave 10/02/1990


Corner of Todman Ave; and Oxford Rd: eastern side

FIGURE 5b
HISTORICAL FLOOD PHOTOGRAPHS
SHEET 2



Barker Road, Southern side

Todman Ave; at Barker Road

Corner of Badgery Ave; and Bates Street


29 Badgery Avenue


Outside 29 Badgery Avenue


139 Albert Road 10.2.90


139 Albert Road 10.2.90


139 Albert Road 10.2.90

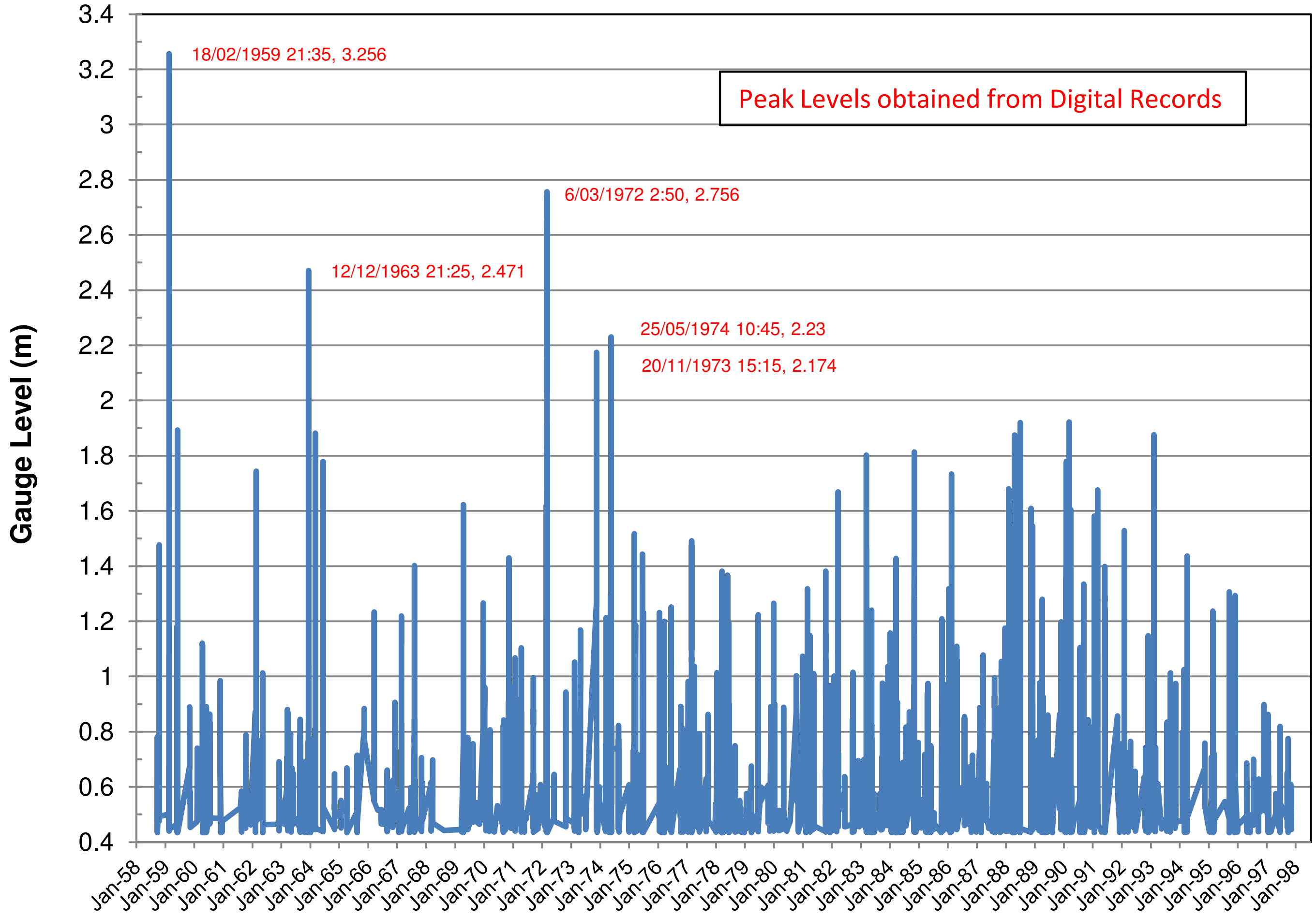

139 Albert Road 10.2.90


139 Albert Road 10.2.90


Underwood Road 18/03/1990


Underwood Road 18/03/1990

FIGURE 6
WATER LEVEL GAUGE RECORDS



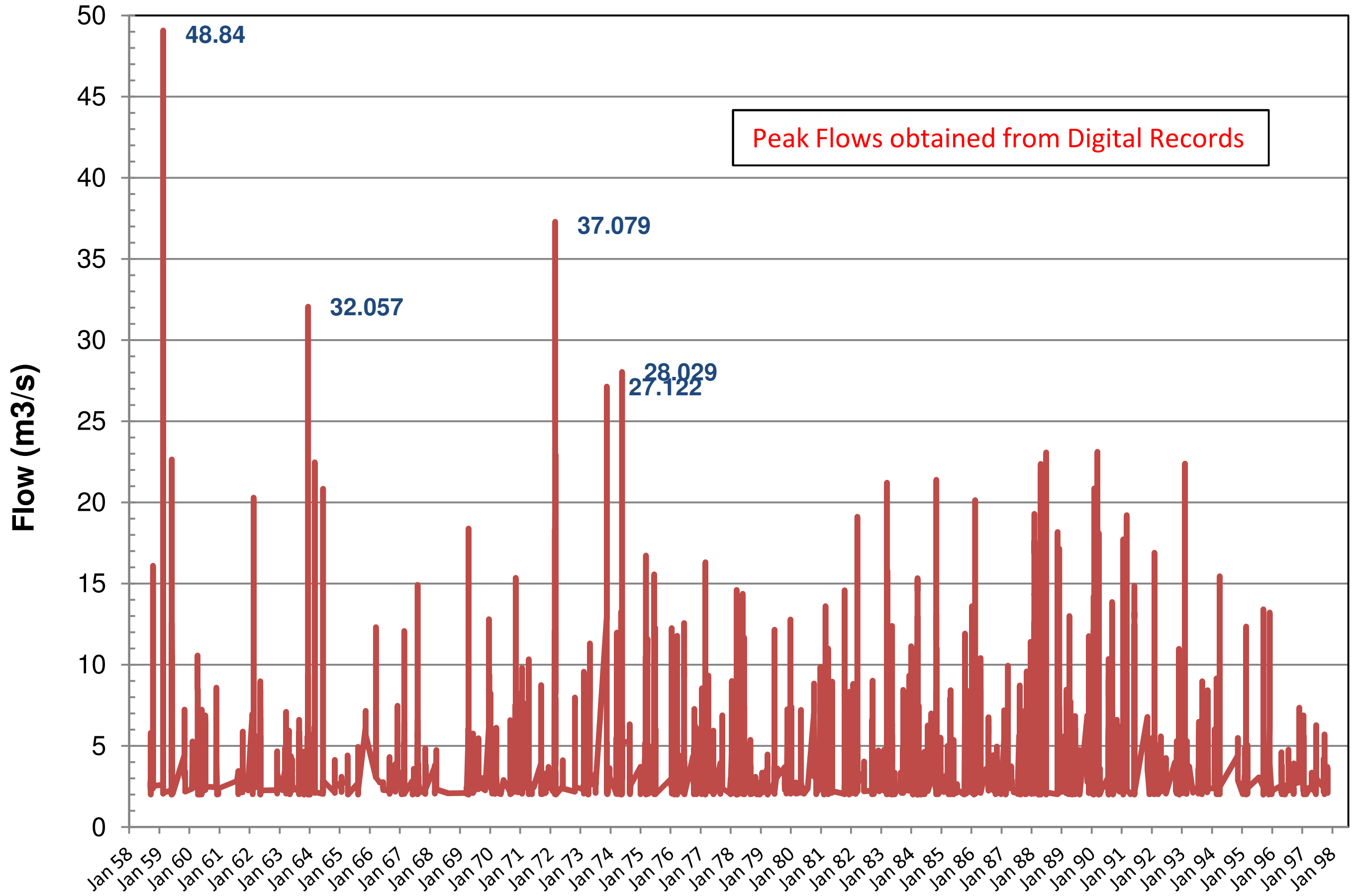


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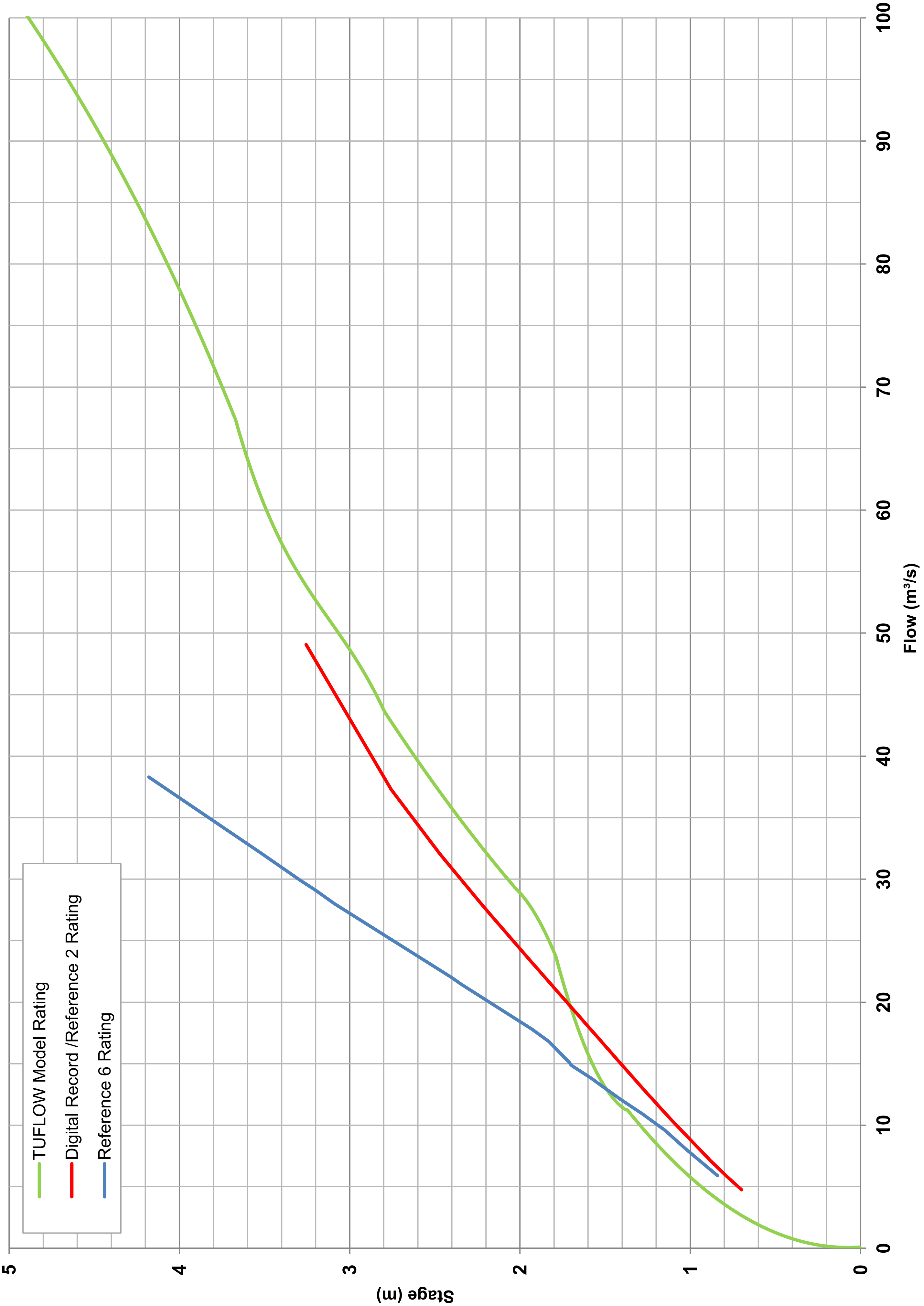
FIGURE 7
RATING CURVES AT ELVA STREET GAUGE



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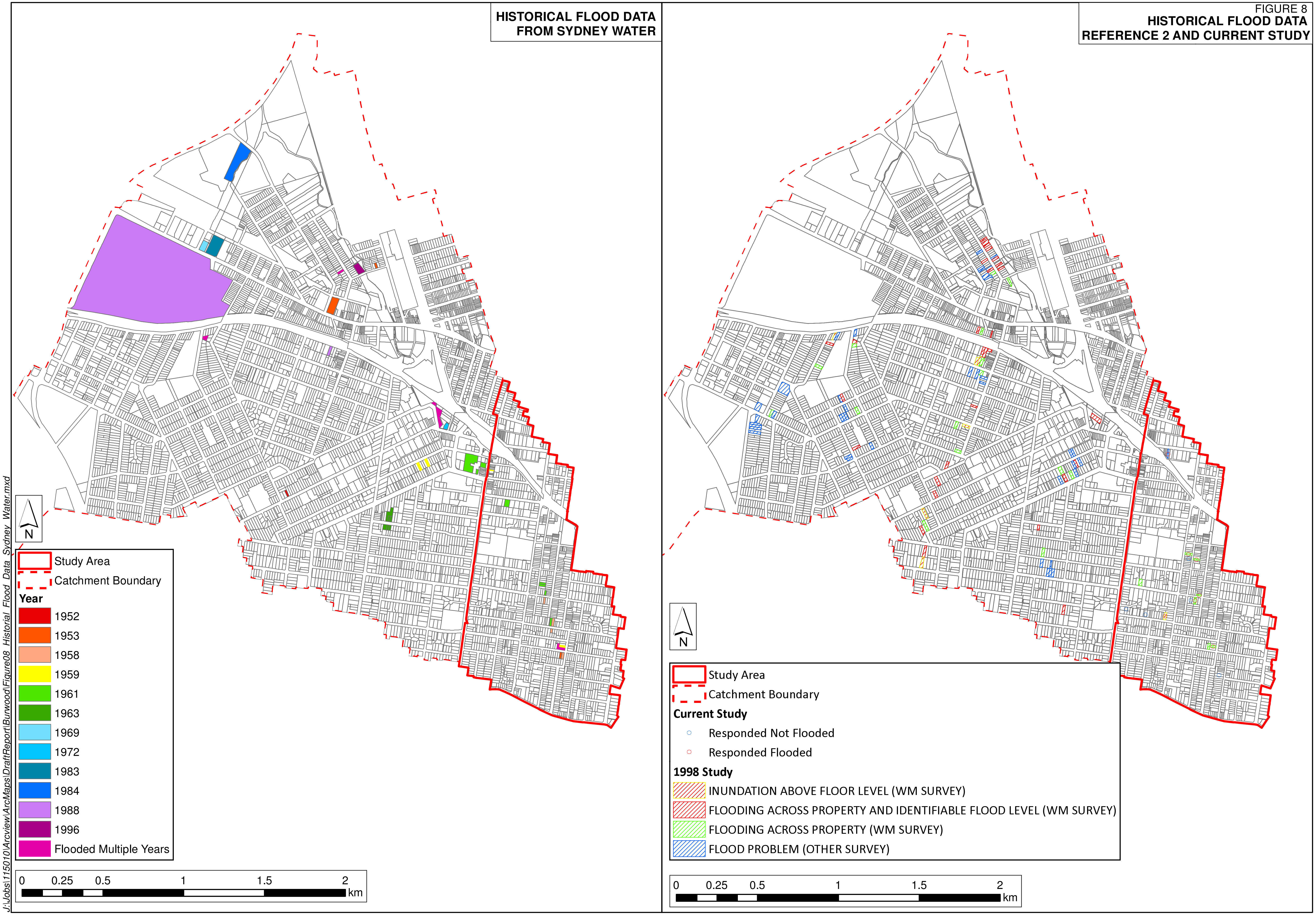

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FIGURE 9a
EVENTS OF FEBRUARY 1990

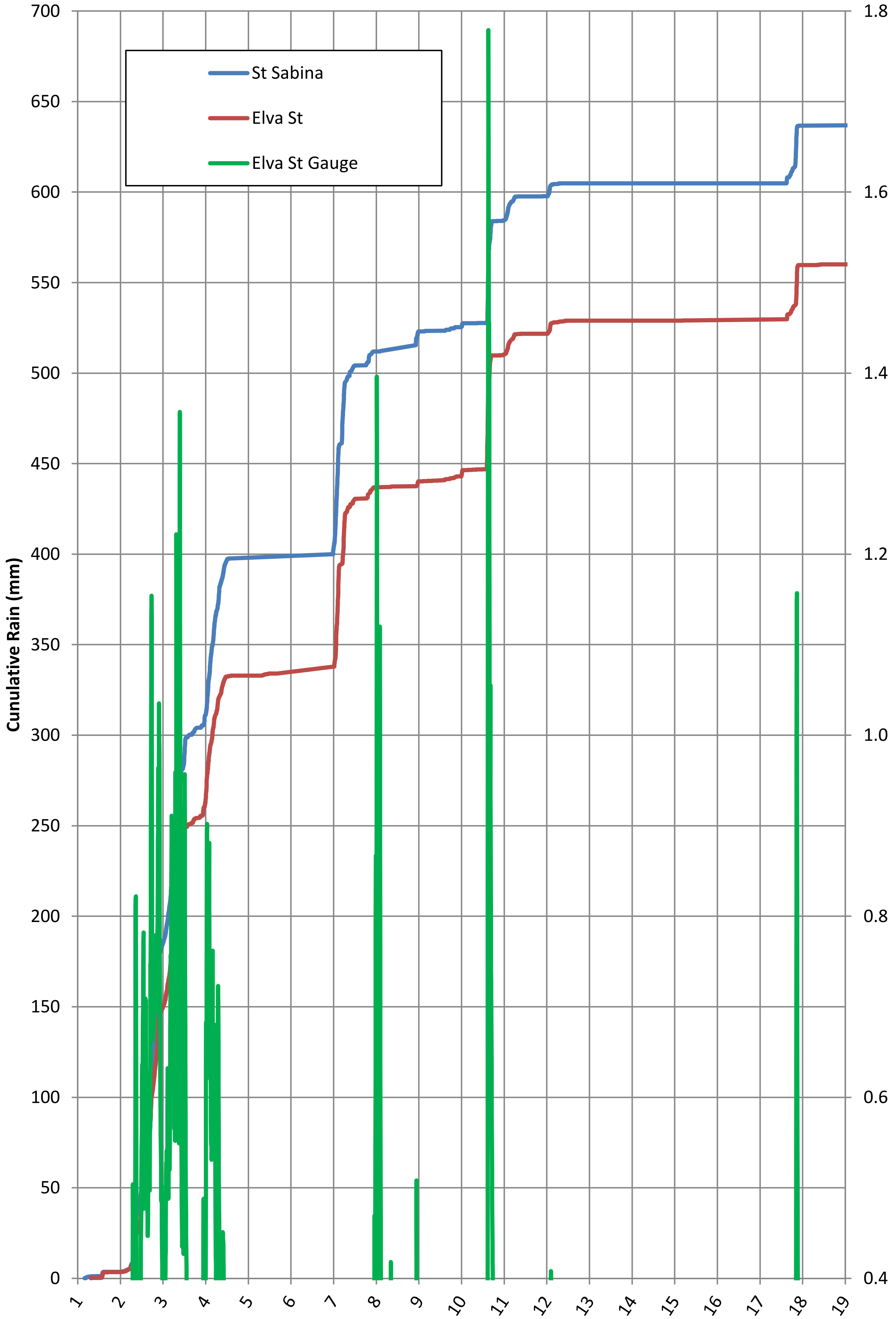

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FIGURE 9b
PLUVIOMETER DATA
2-4 FEBRUARY 1990

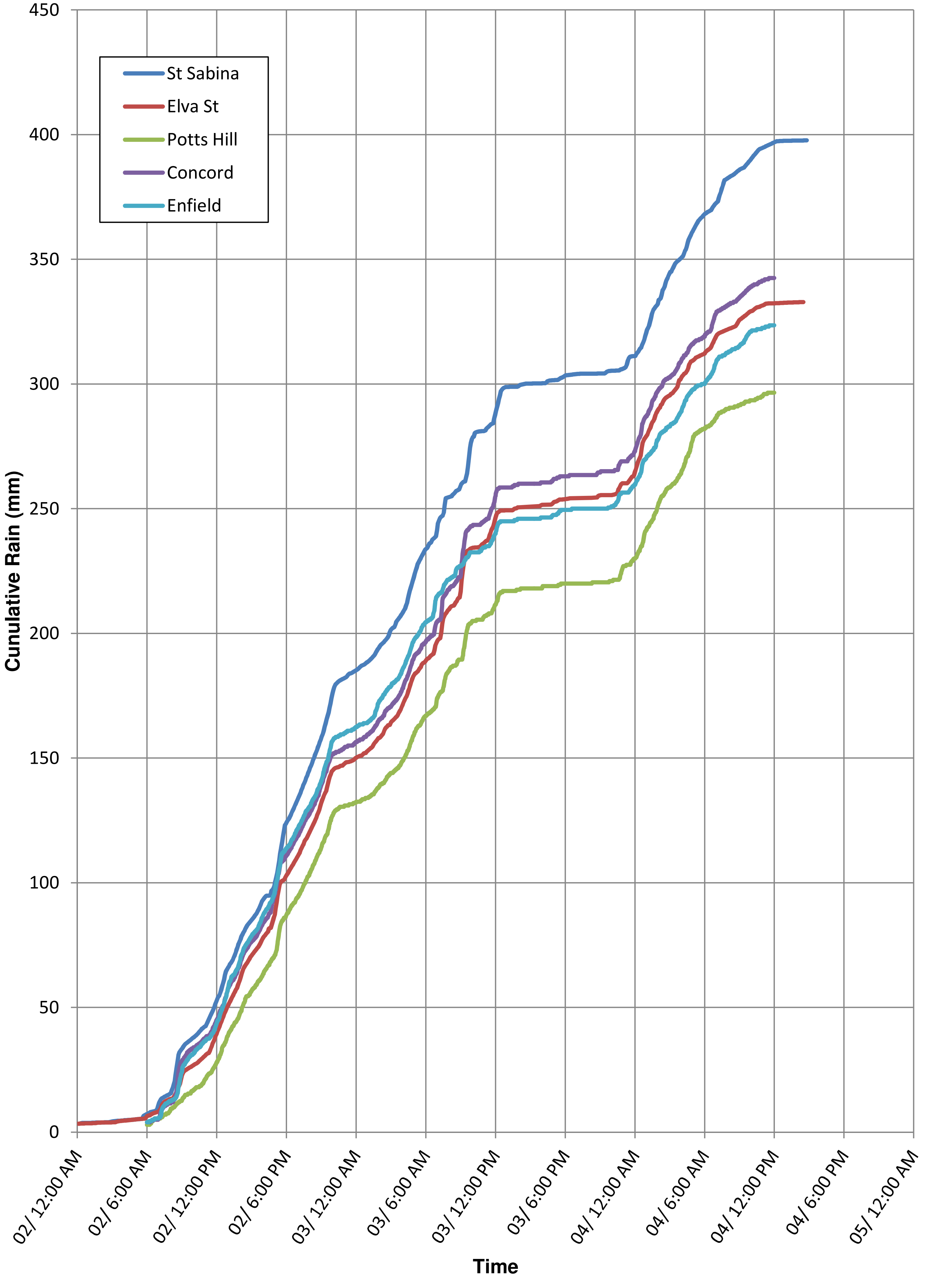

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FIGURE 9c
PLUVIOMETER DATA
7 FEBRUARY 1990

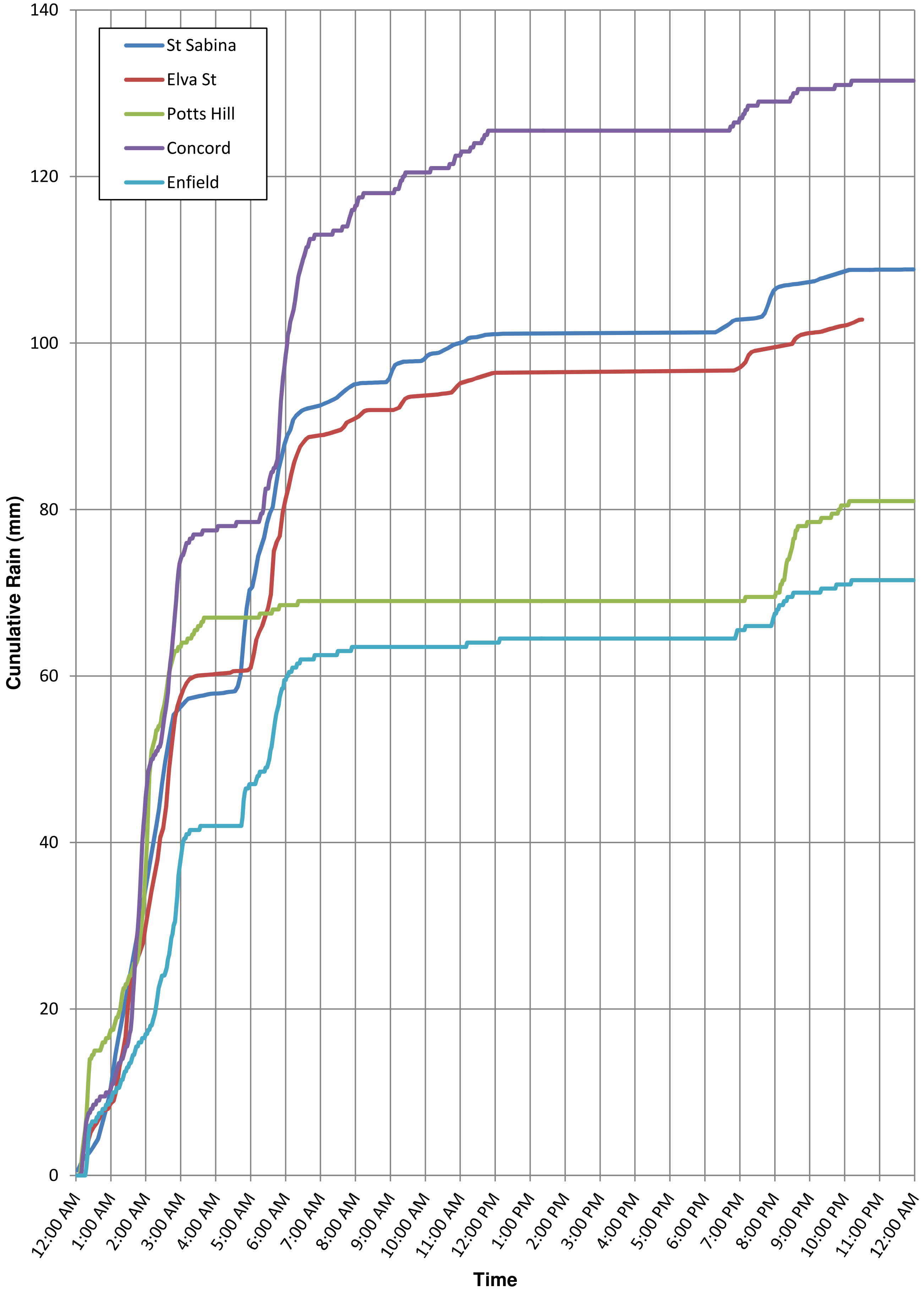

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FIGURE 9d
PLUVIOMETER DATA
10 FEBRUARY 1990

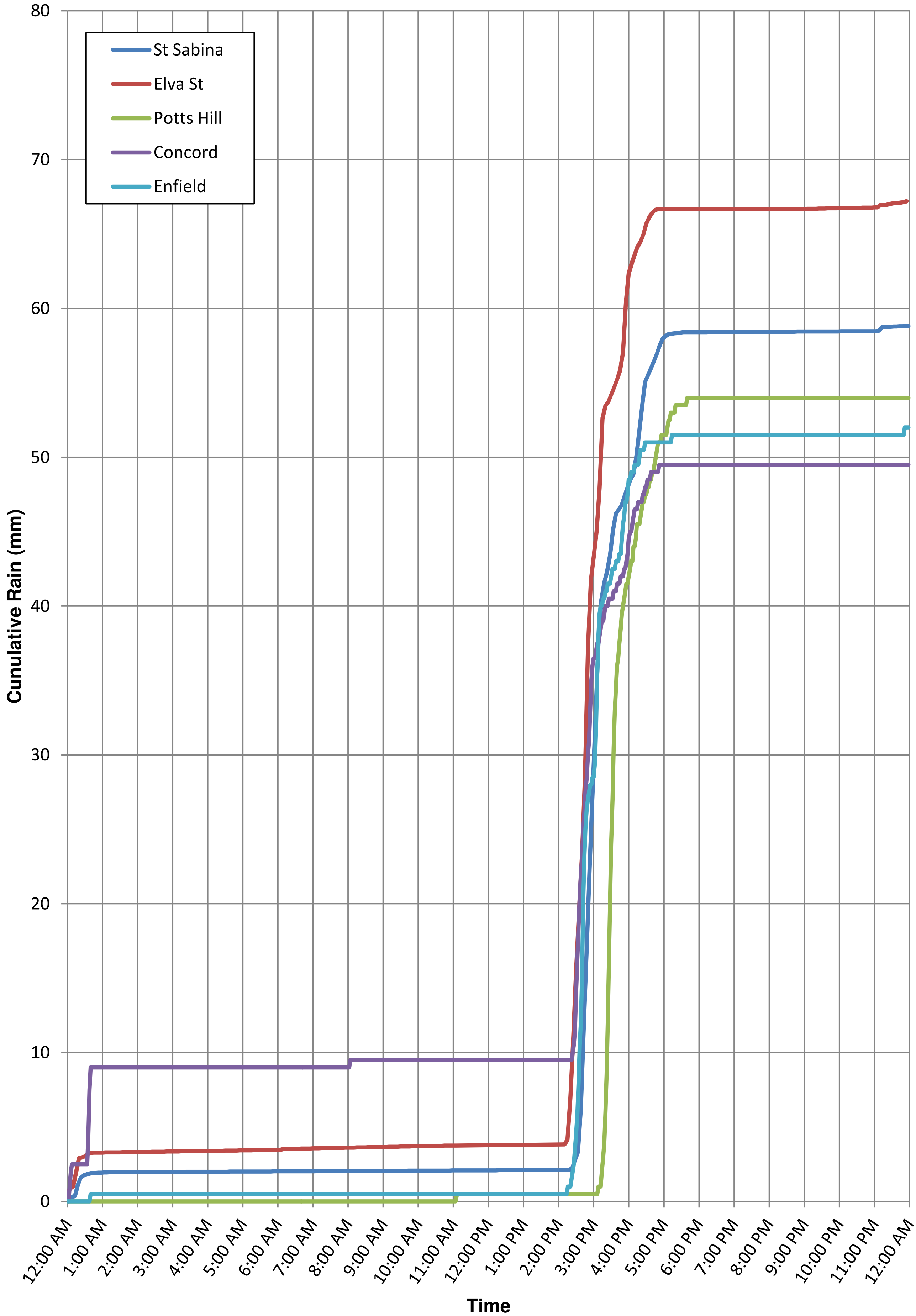

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FIGURE 9e
PLUVIOMETER DATA
17 FEBRUARY 1990

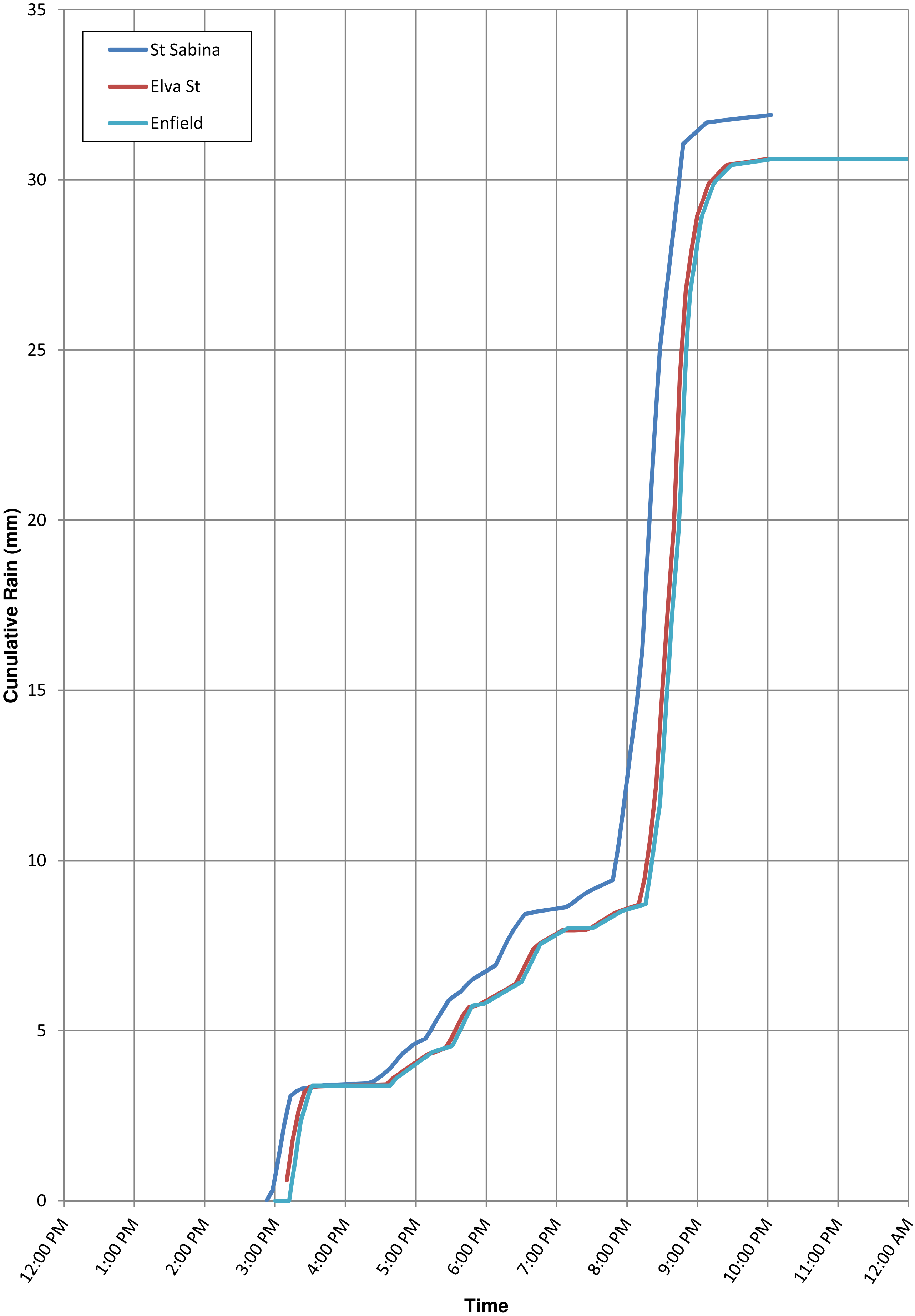

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FIGURE 9f
PLUVIOMETER DATA
18 MARCH 1990

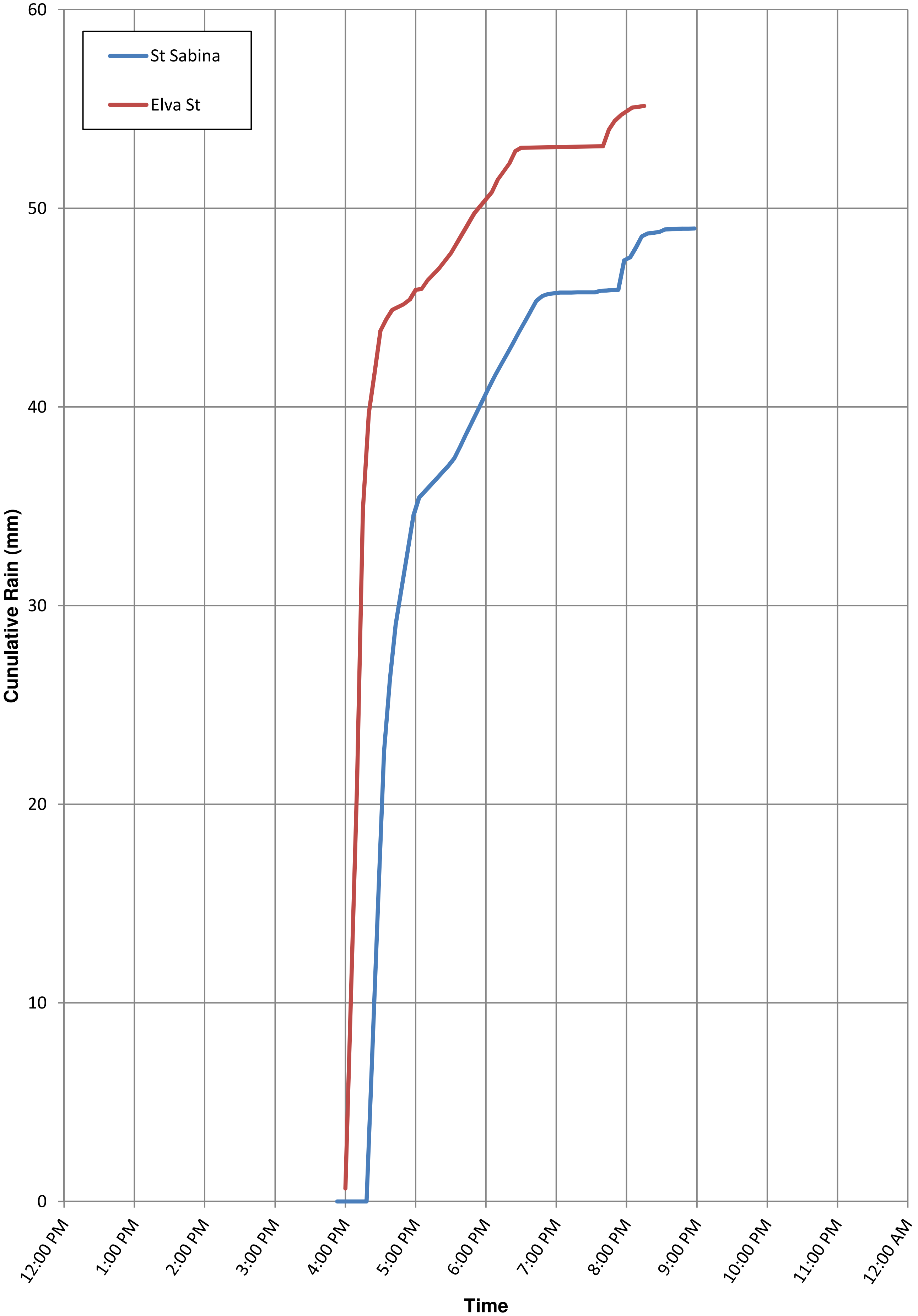

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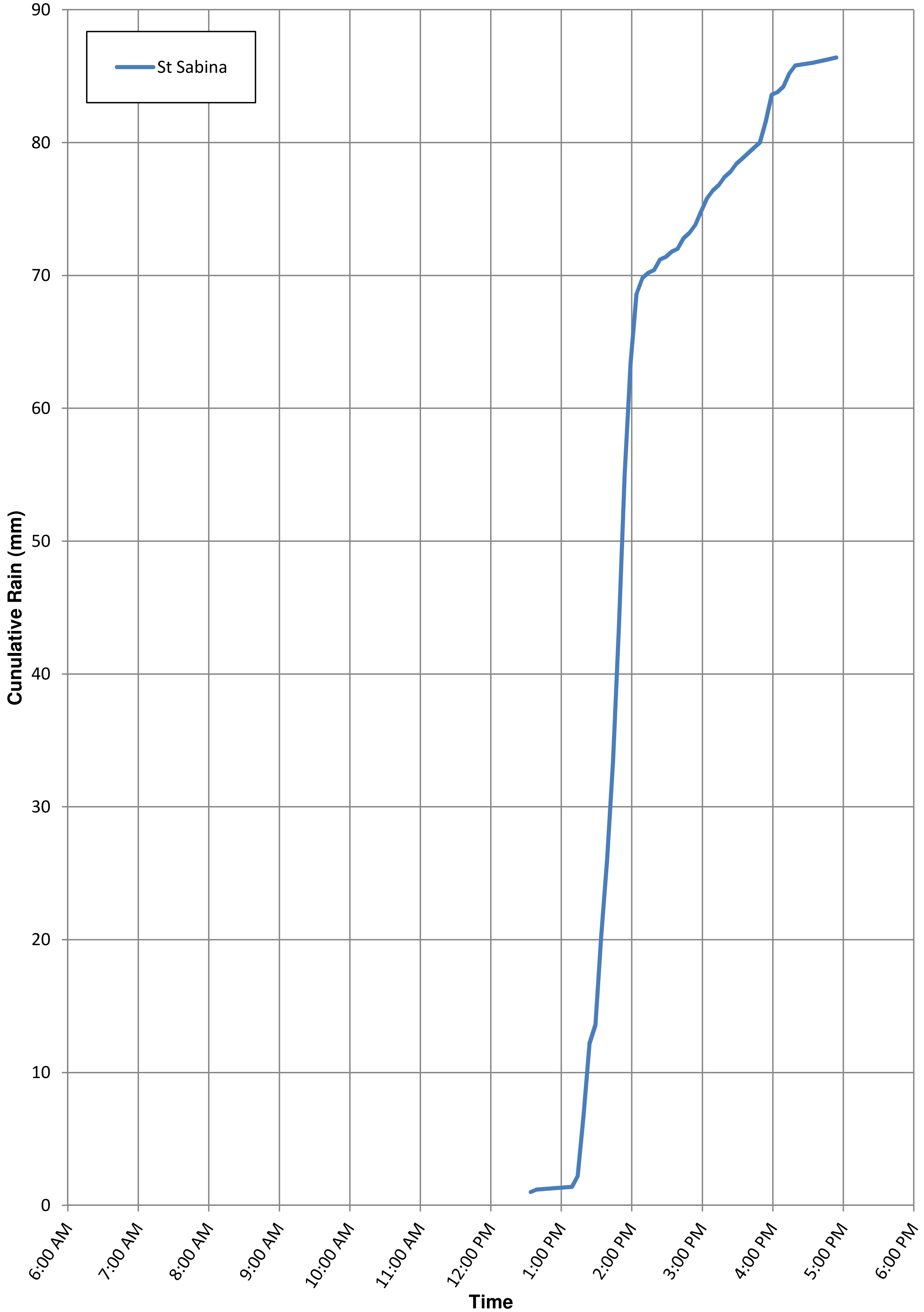
FIGURE 9g

PLUVIOMETER DATA
2 JANUARY 1996



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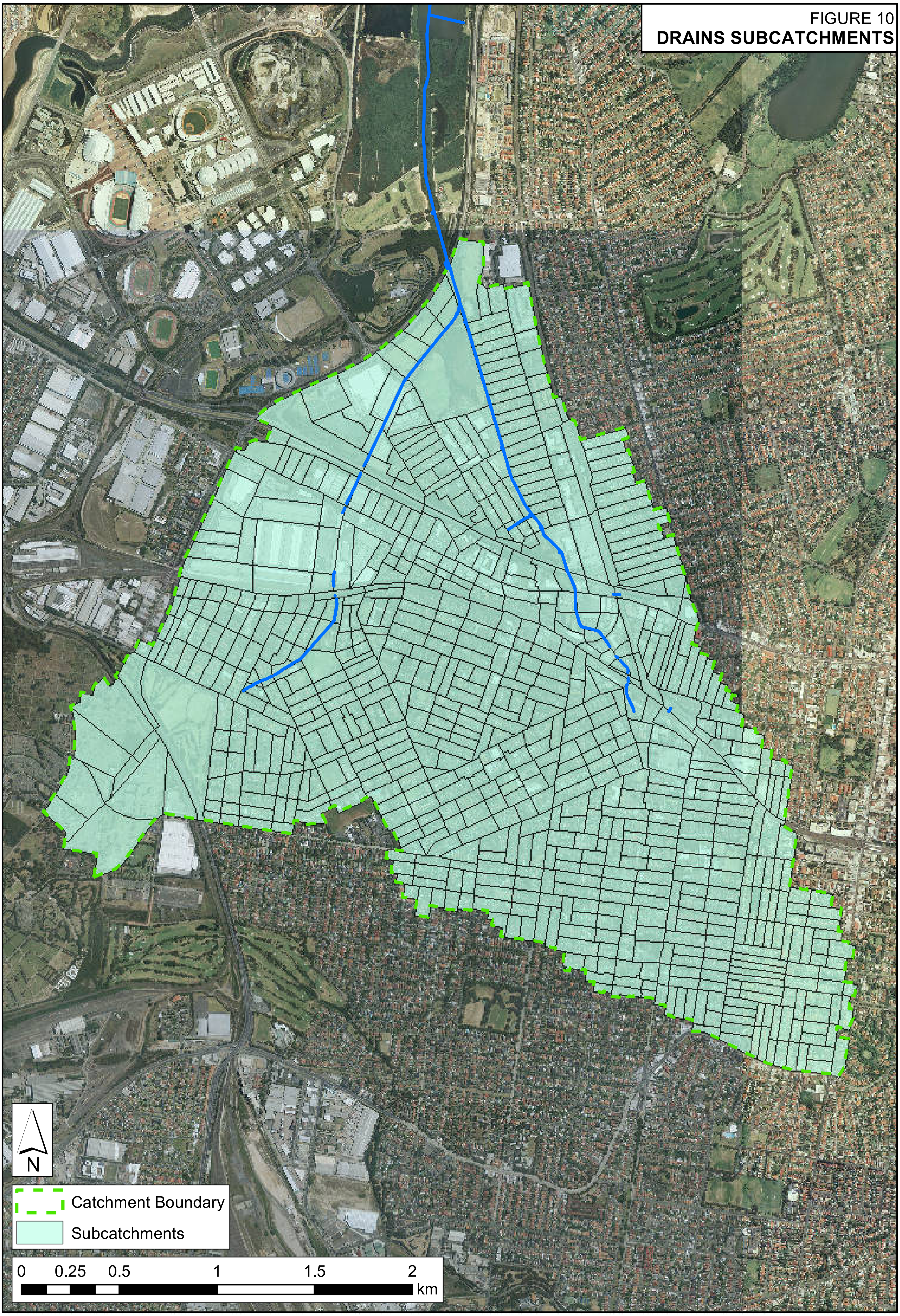
FIGURE 10
DRAINS SUBCATCHMENTS
N
Catchment Boundary
Subcatchments
0
0.25
0.5
1
1.5
2
km
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FIGURE 11
TUFLOW PITS AND PIPES

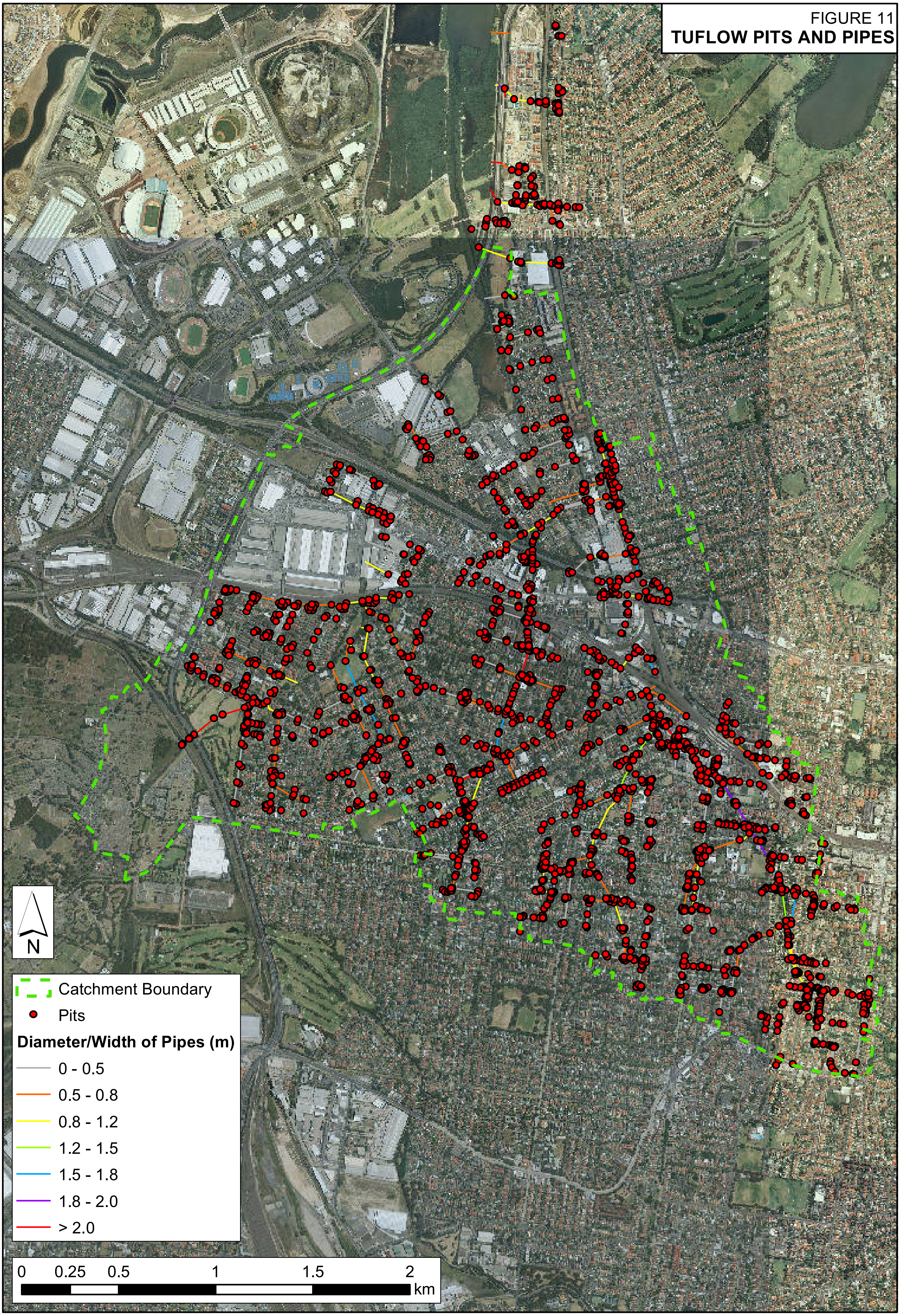

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FIGURE 12
TUFLOW MODEL EXTENT

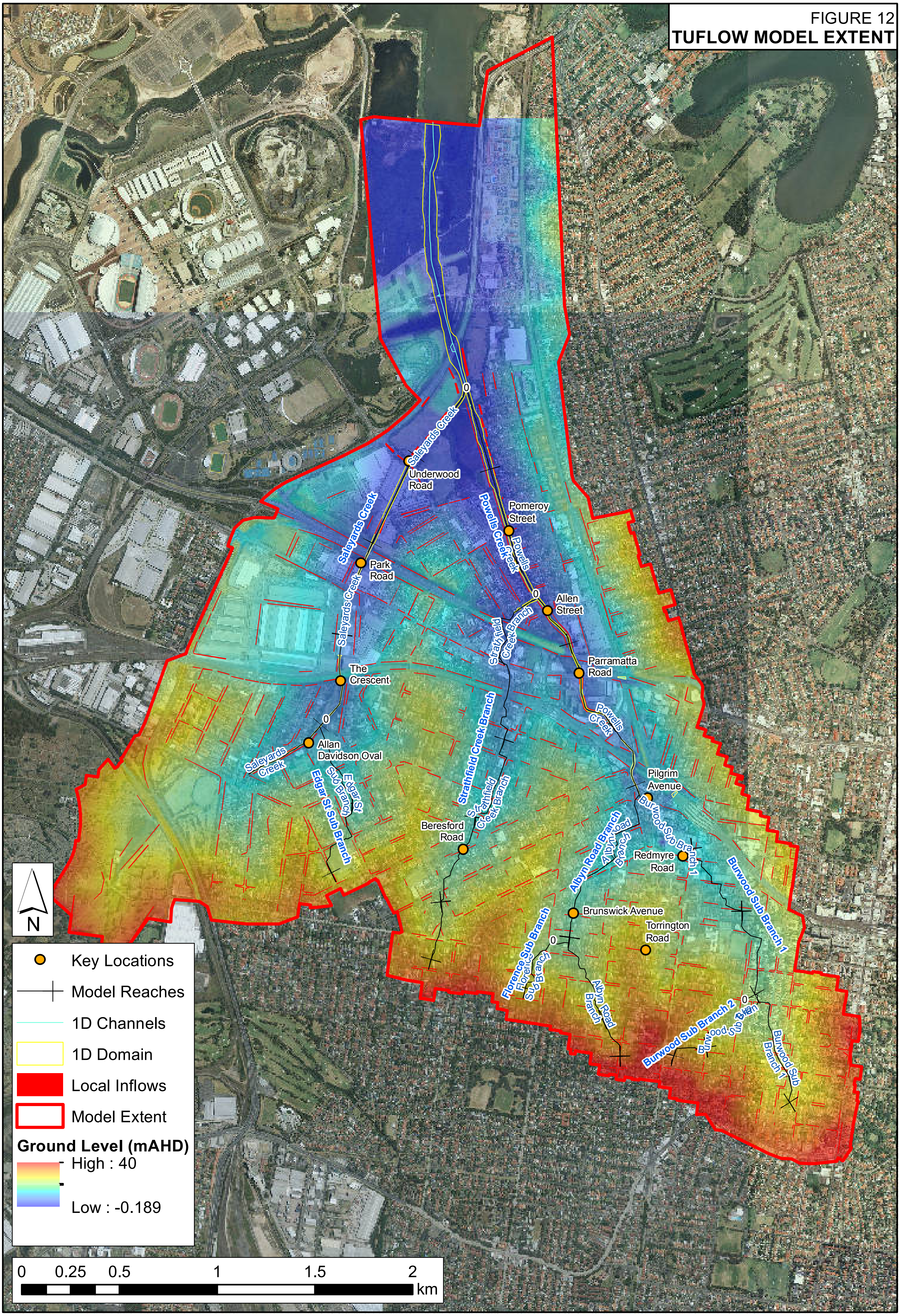

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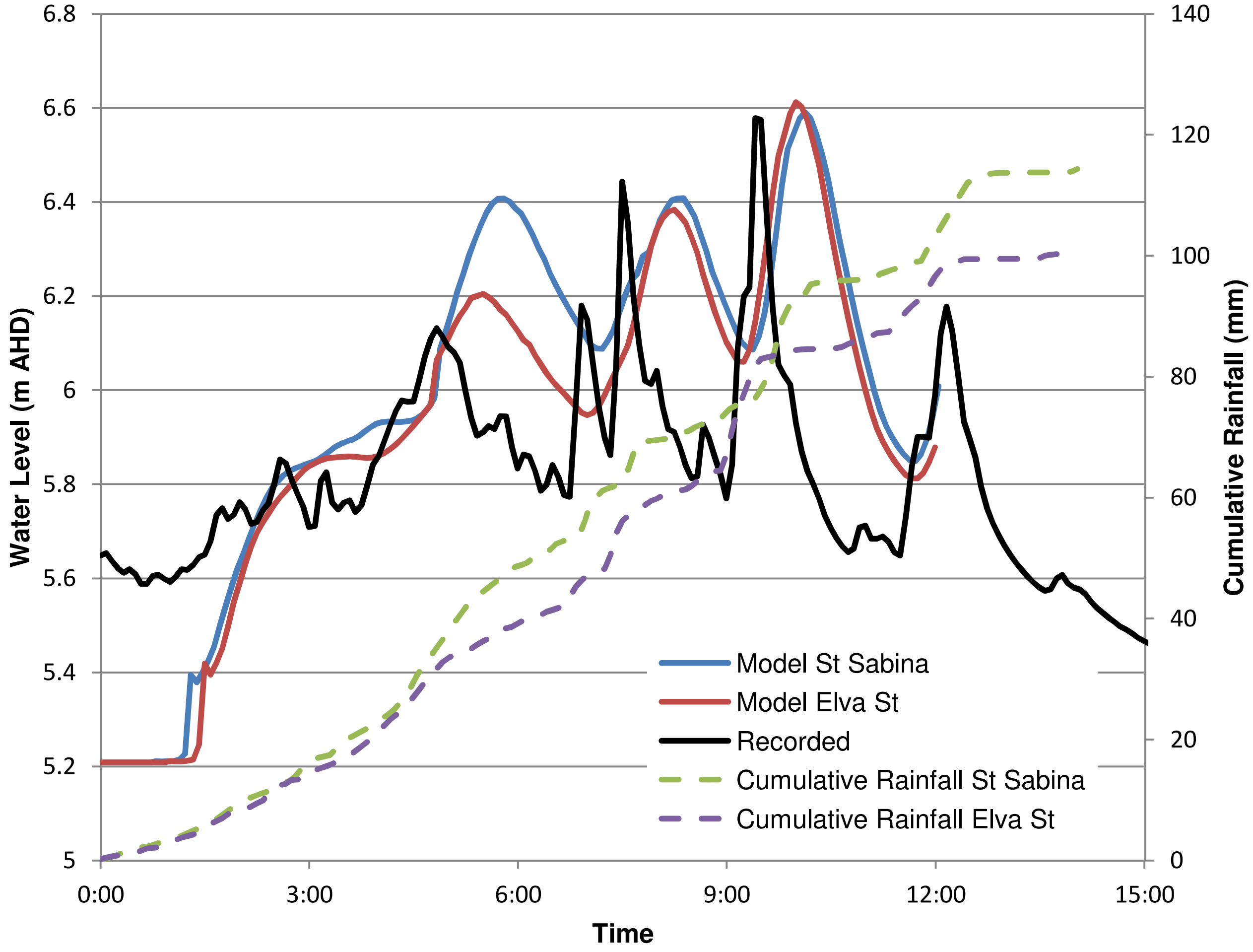
FIGURE 13a

CALIBRATION RESULTS--ELVA STREET GAUGE

3 FEBRUARY 1990



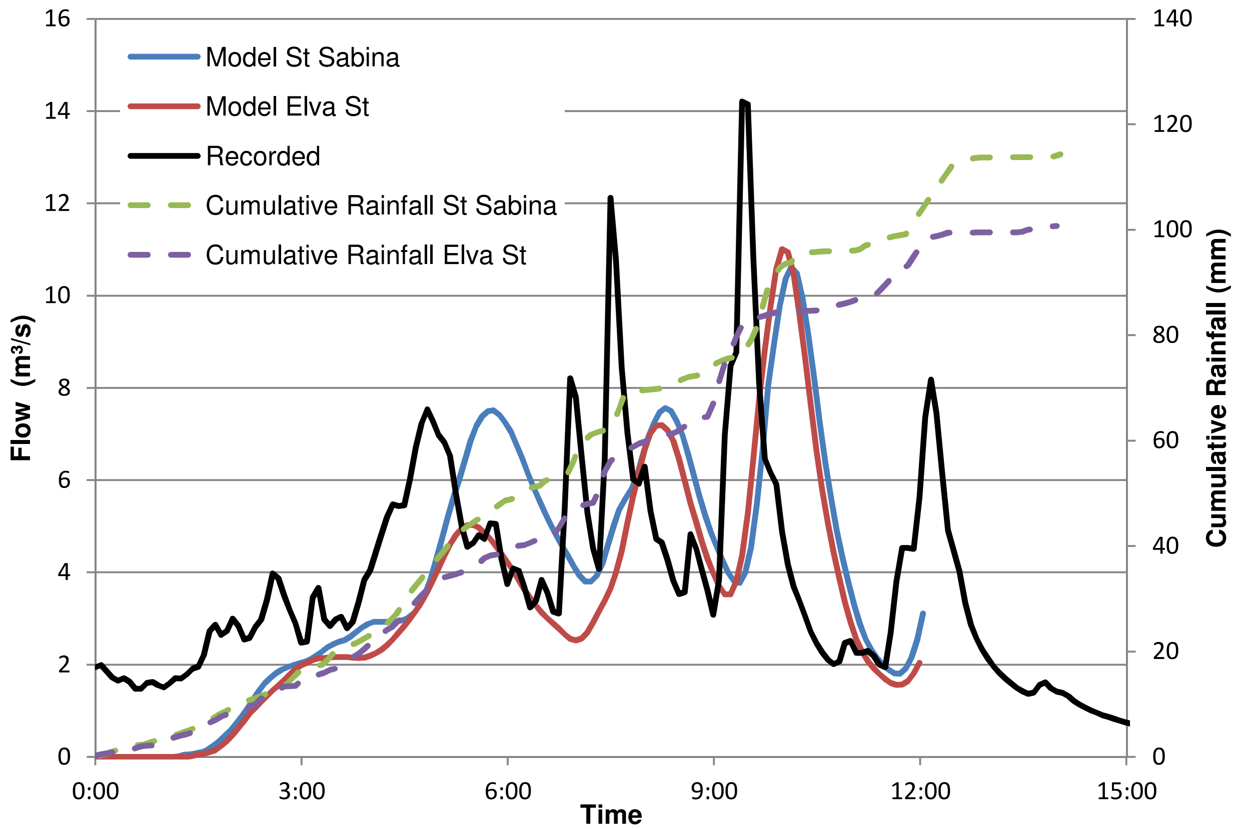


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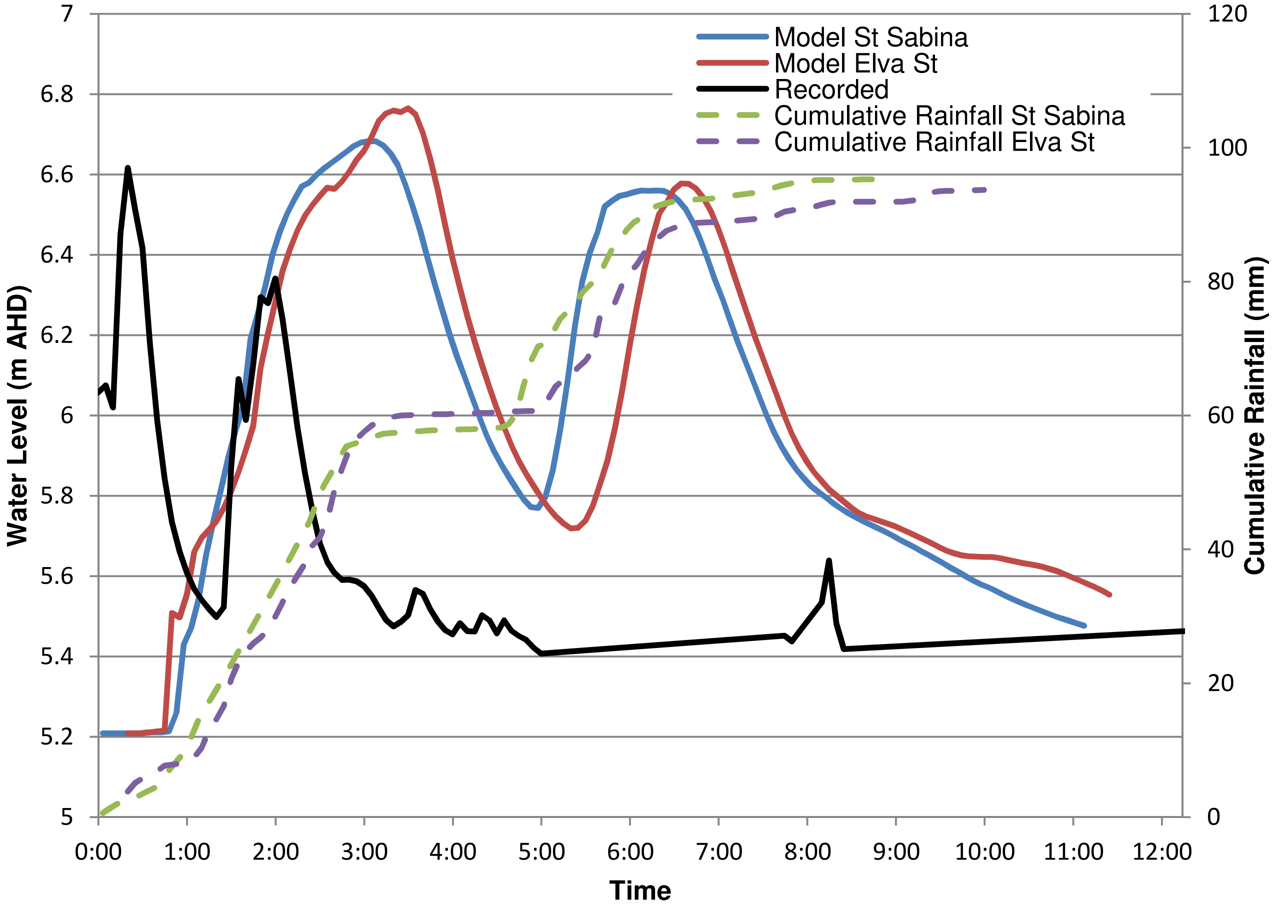
FIGURE 13b
CALIBRATION RESULTS--ELVA STREET GAUGE
7 FEBRUARY 1990



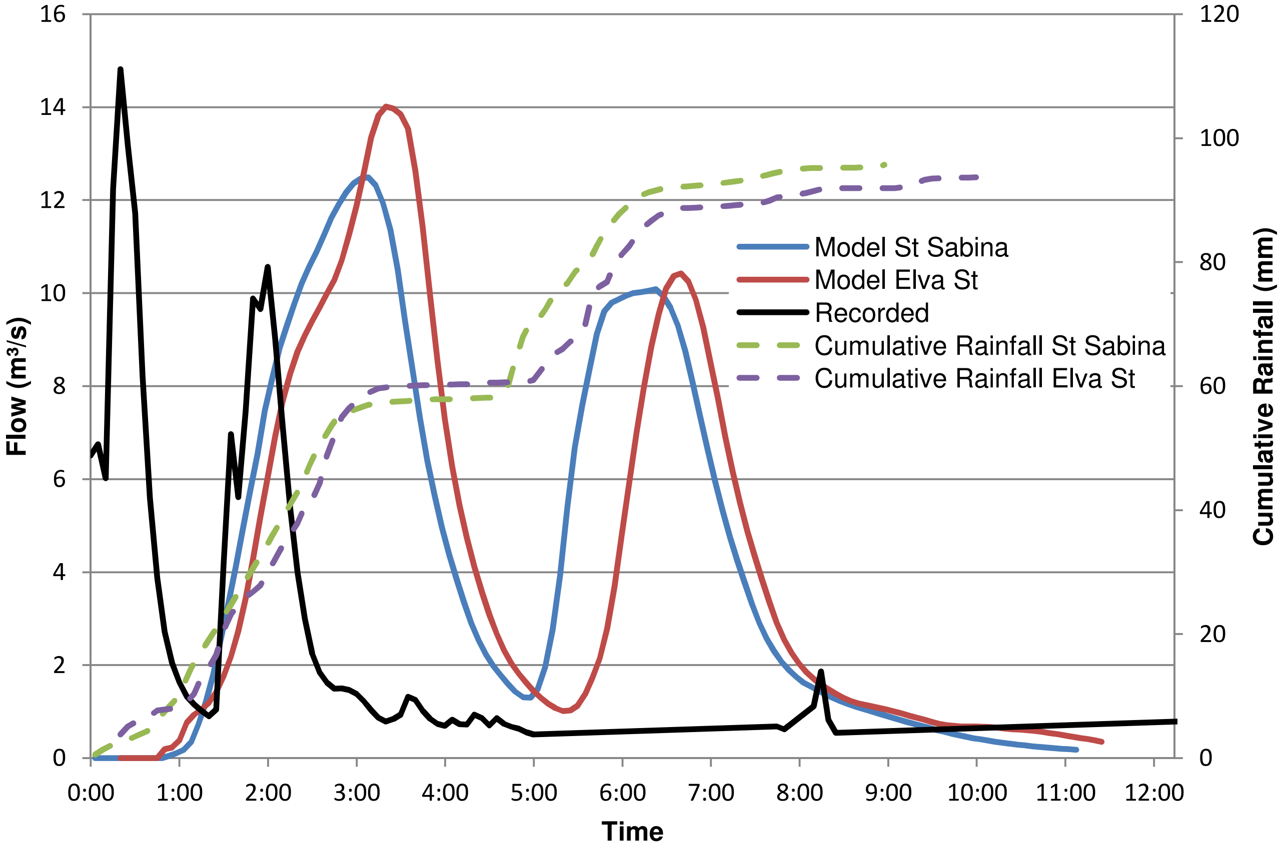


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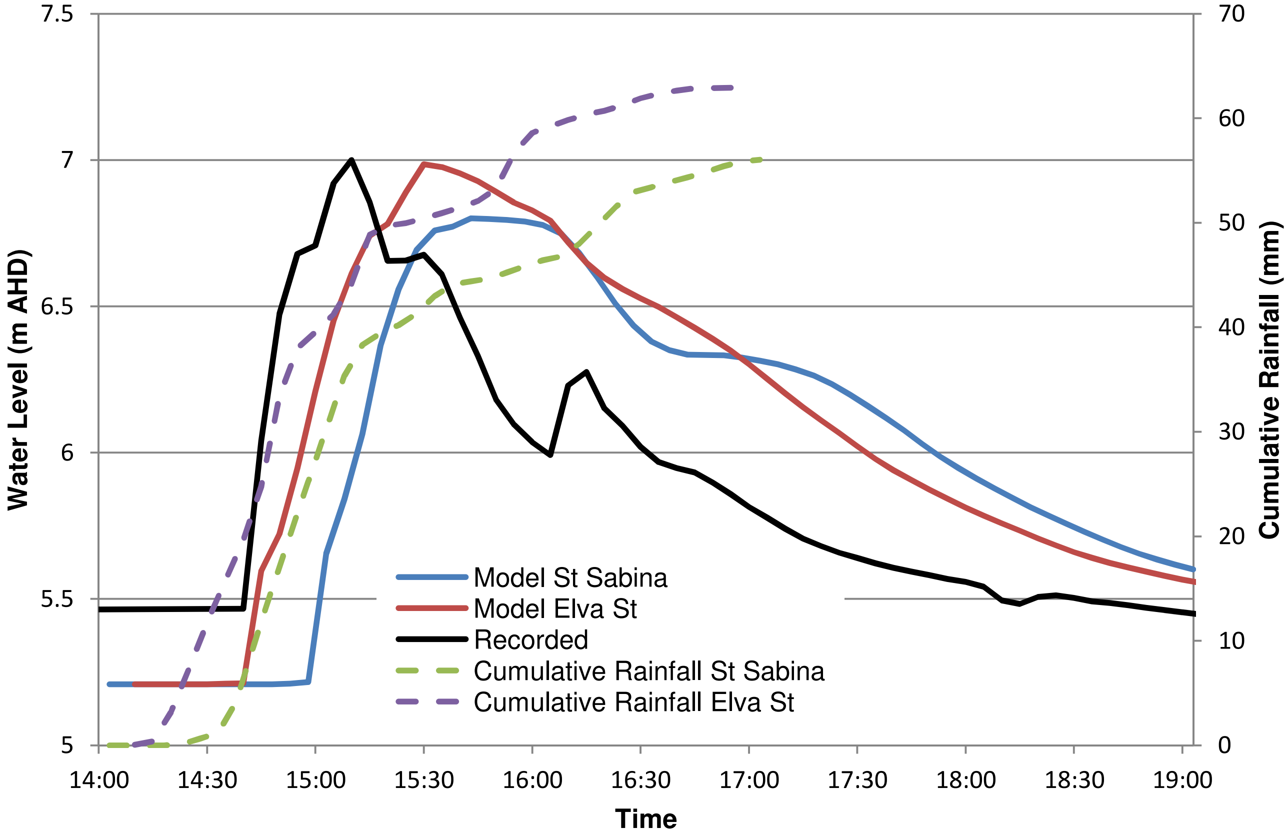
FIGURE 13c

CALIBRATION RESULTS--ELVA STREET GAUGE

10 FEBRUARY 1990



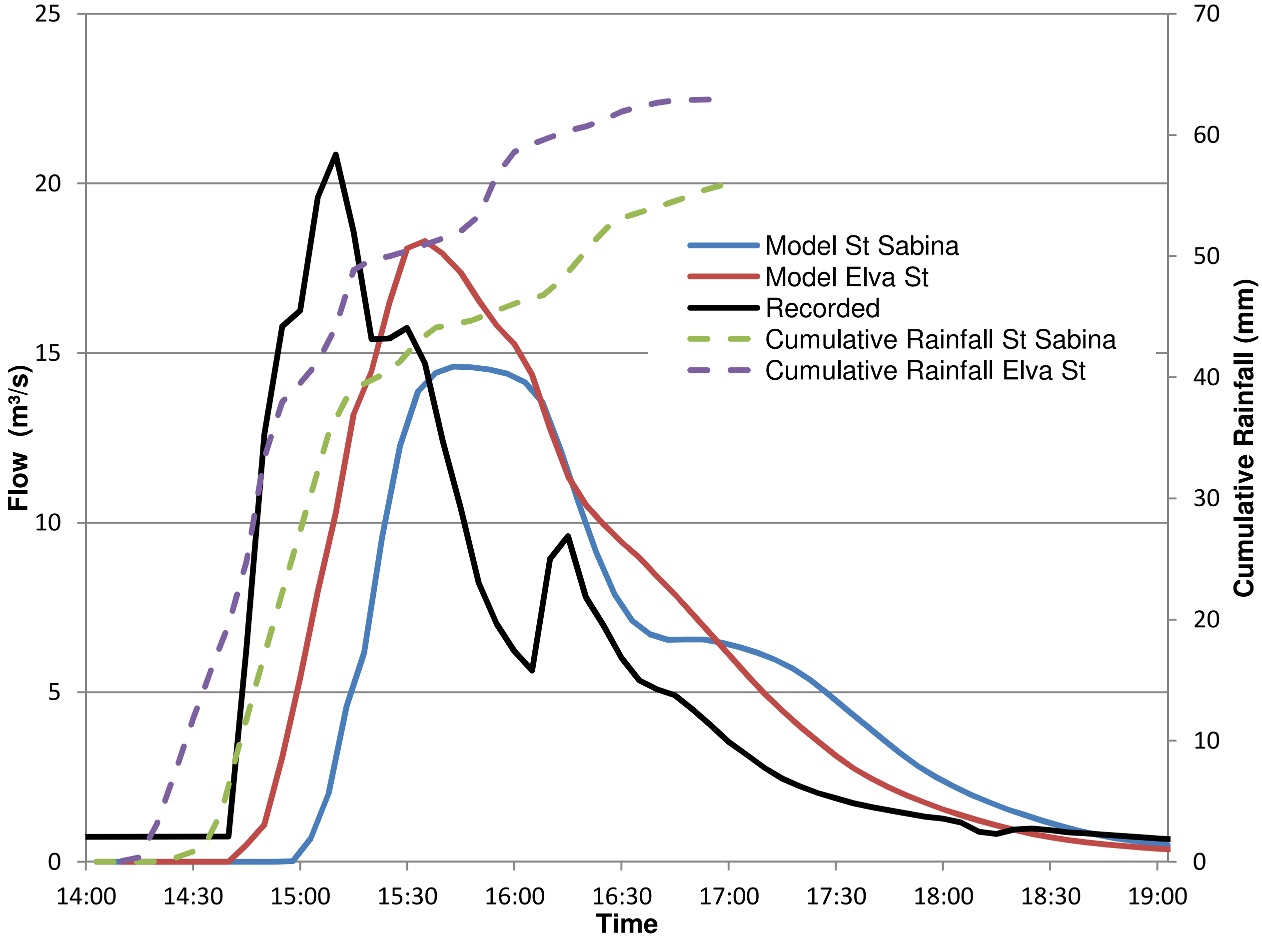
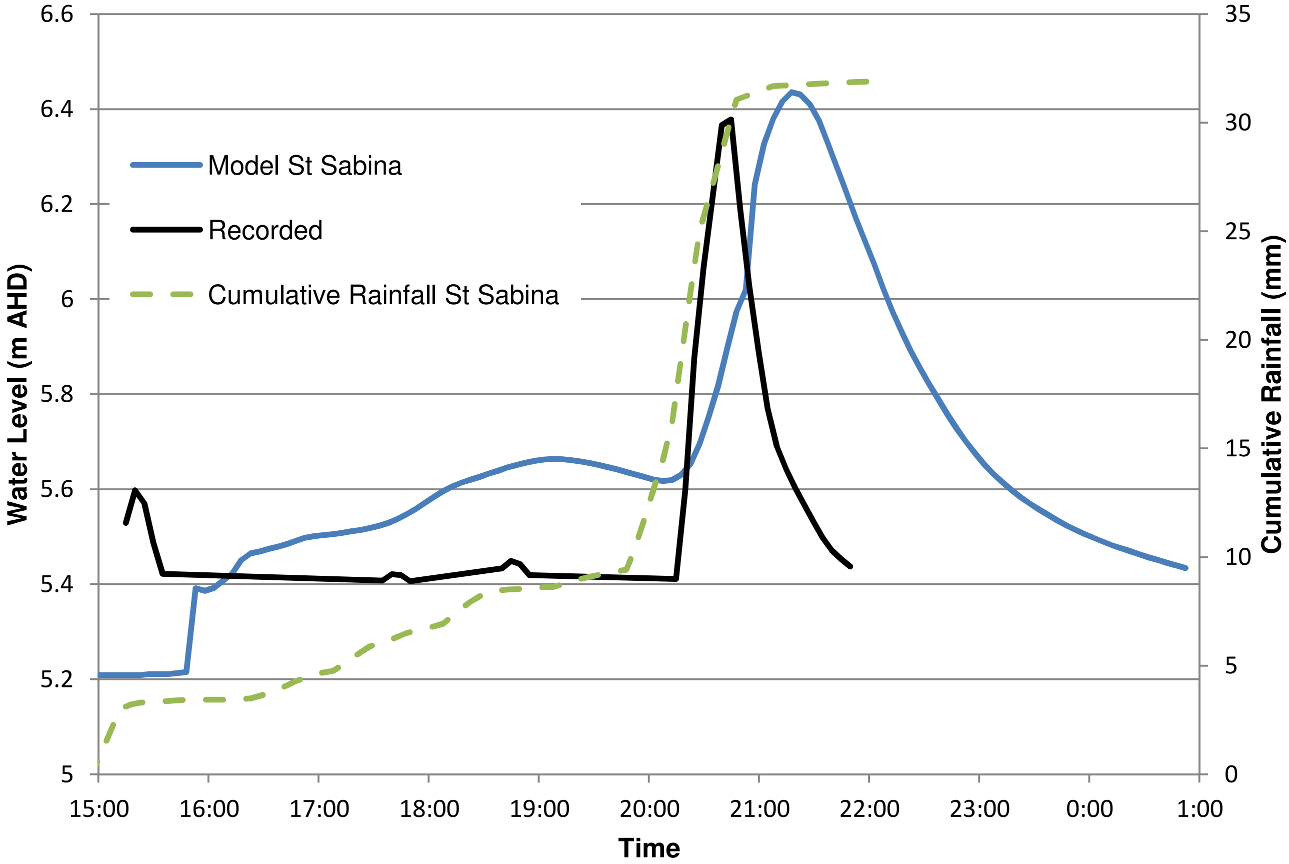


FIGURE 13d

CALIBRATION RESULTS--ELVA STREET GAUGE
17 FEBRUARY 1990



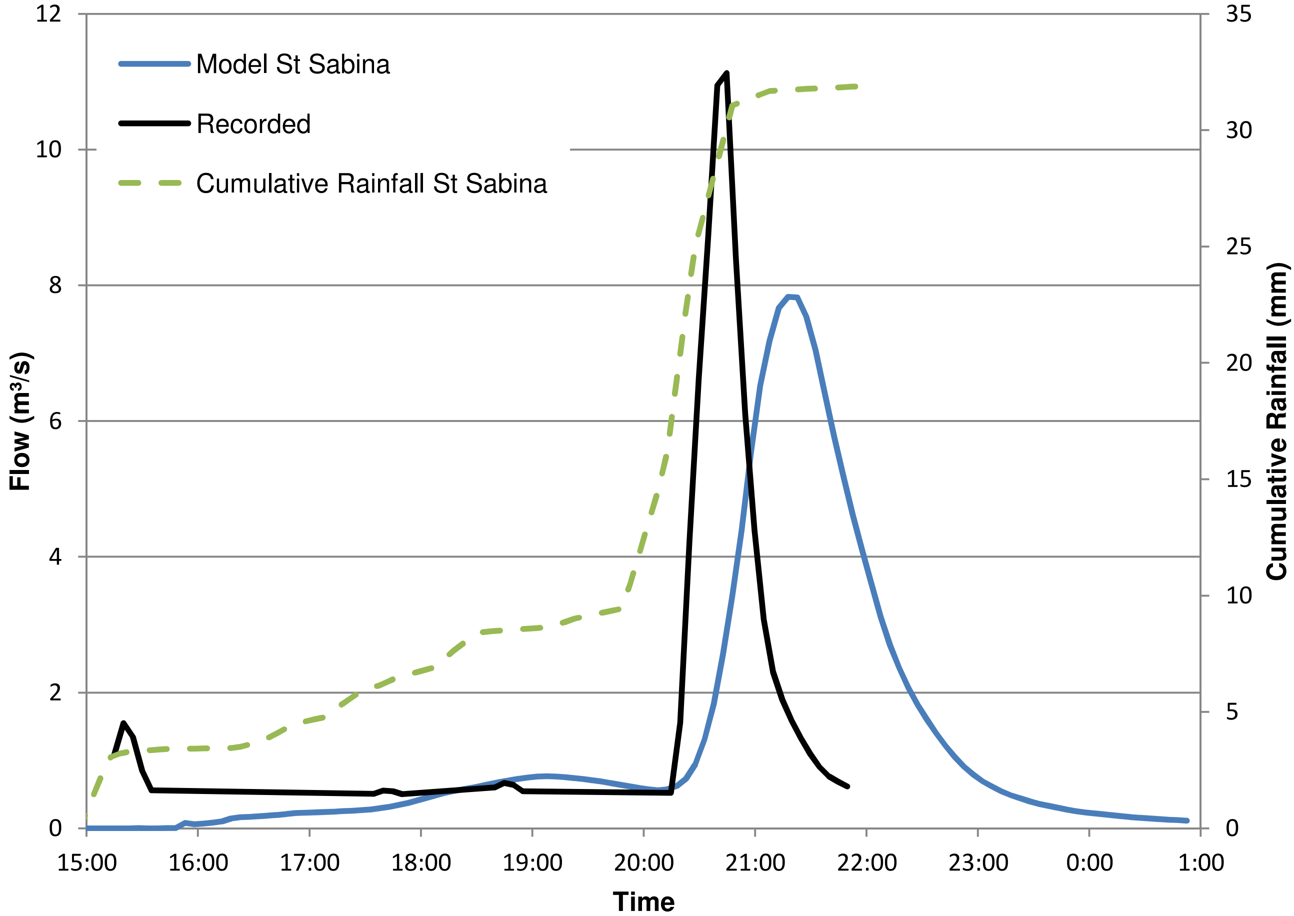


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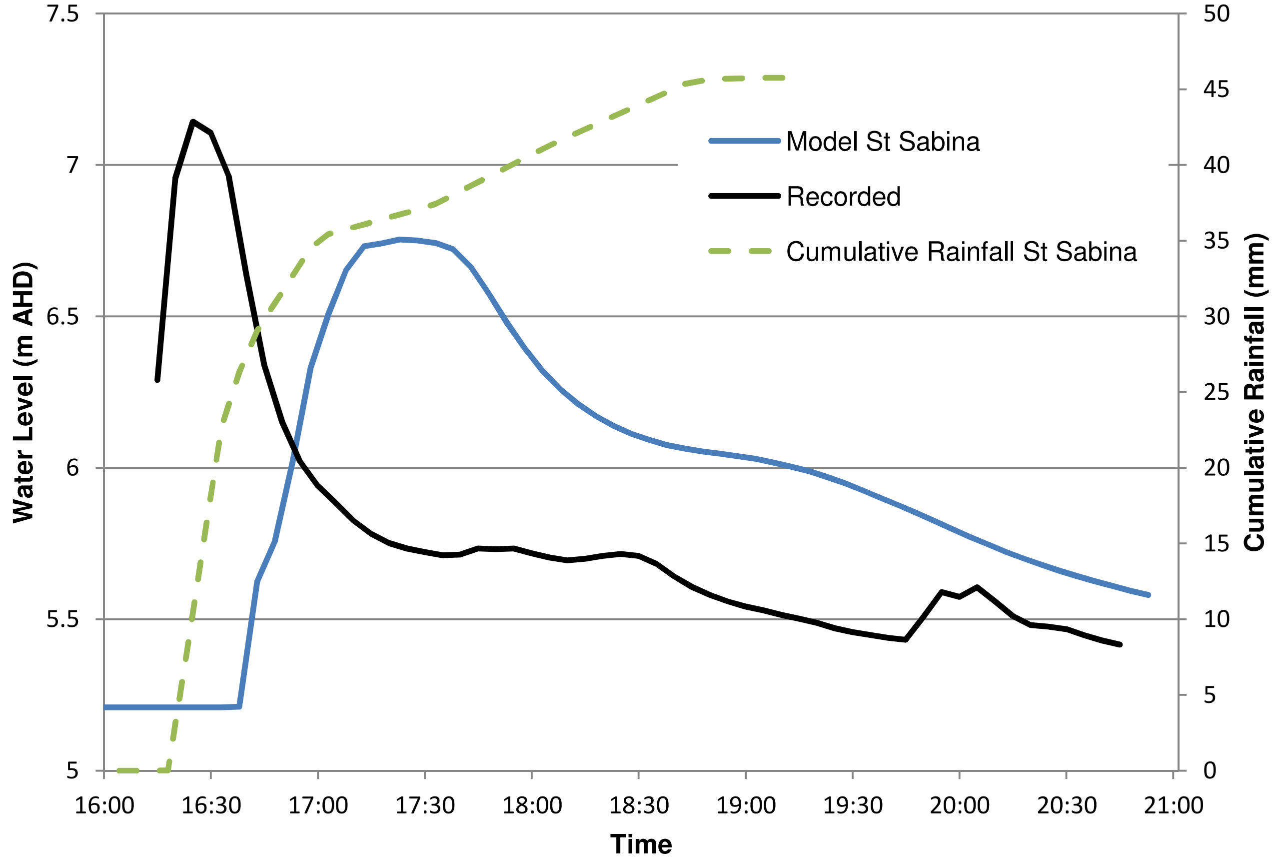
FIGURE 13e

CALIBRATION RESULTS--ELVA STREET GAUGE

18 MARCH 1990



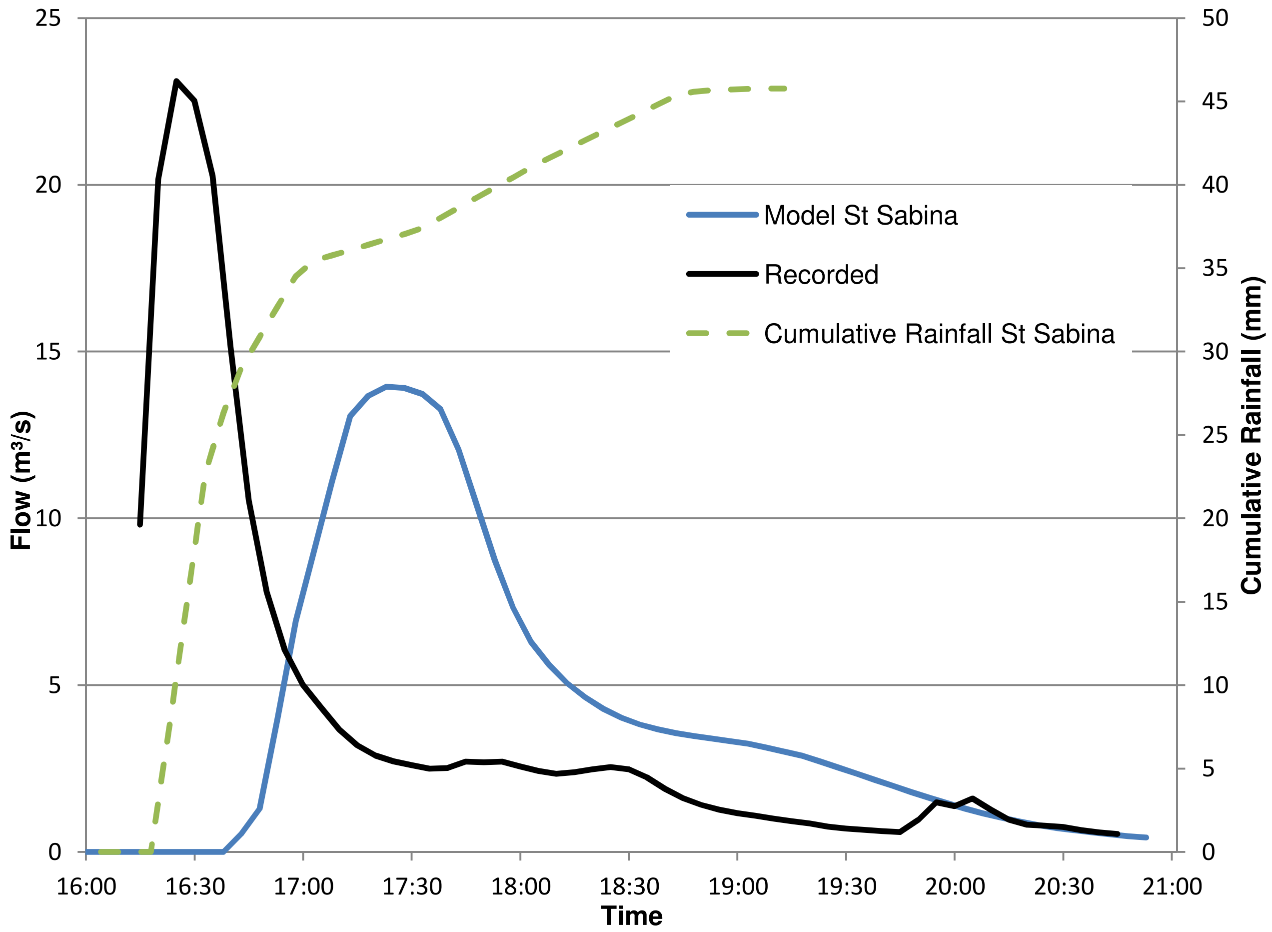


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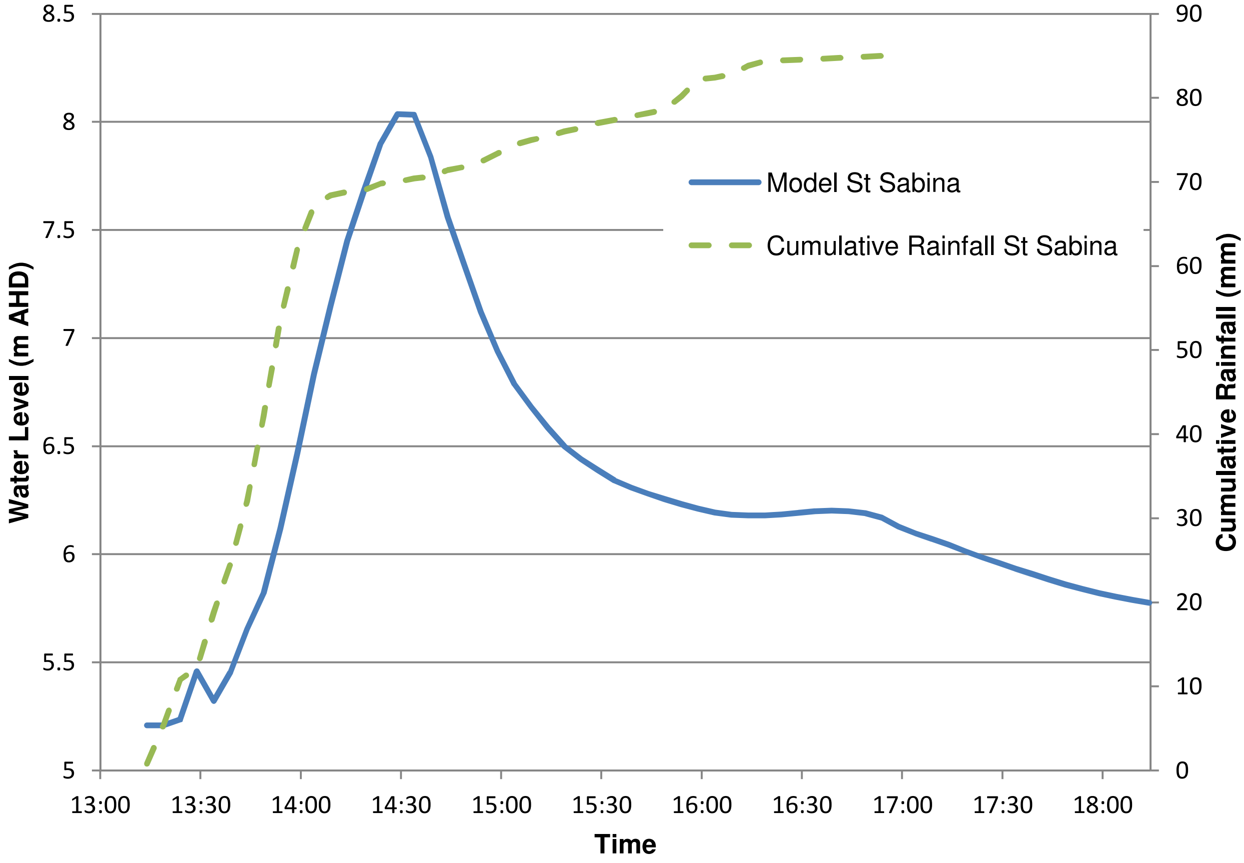
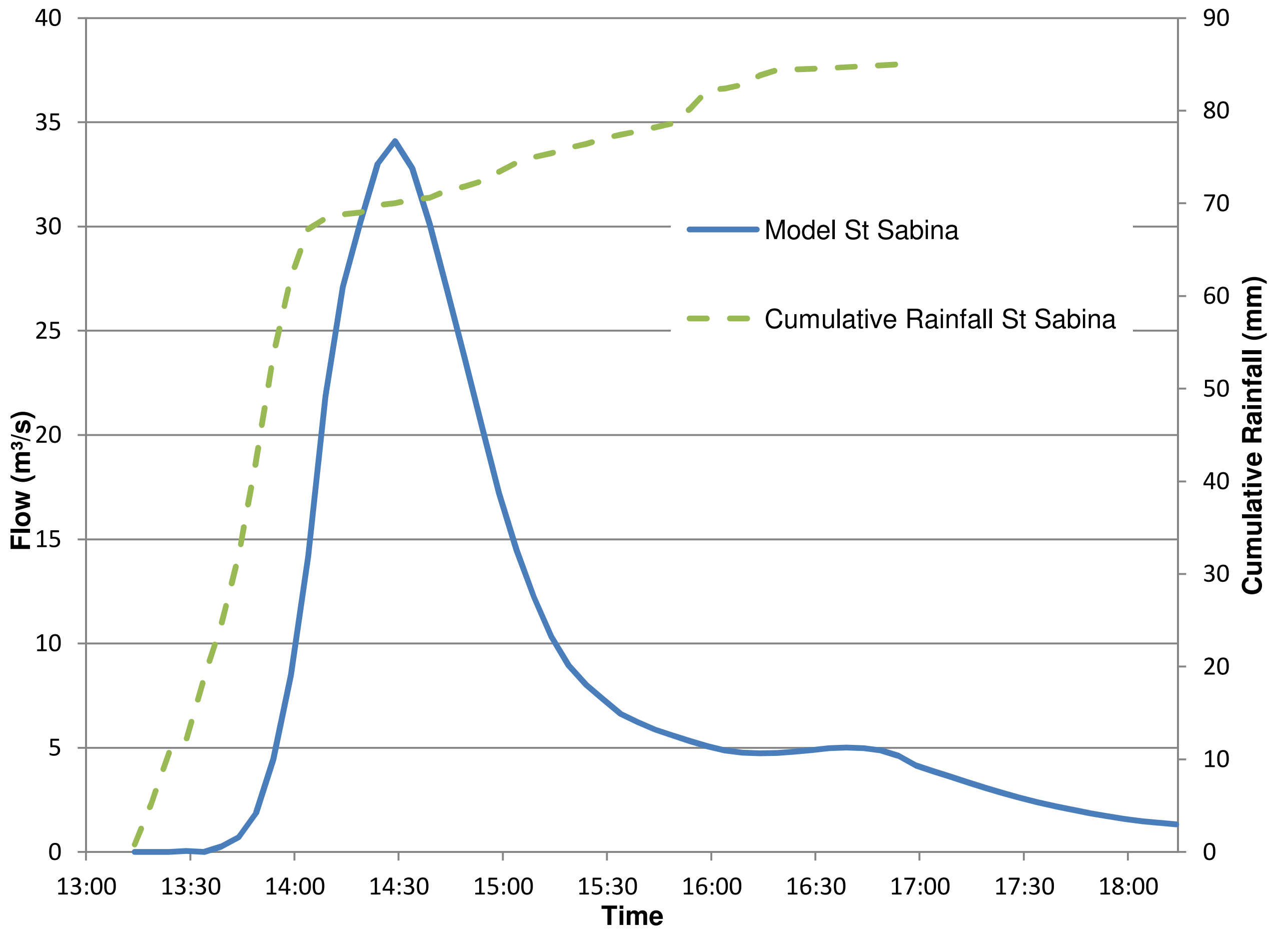
FIGURE 13f

CALIBRATION RESULTS--ELVA STREET GAUGE

2 JANUARY 1996





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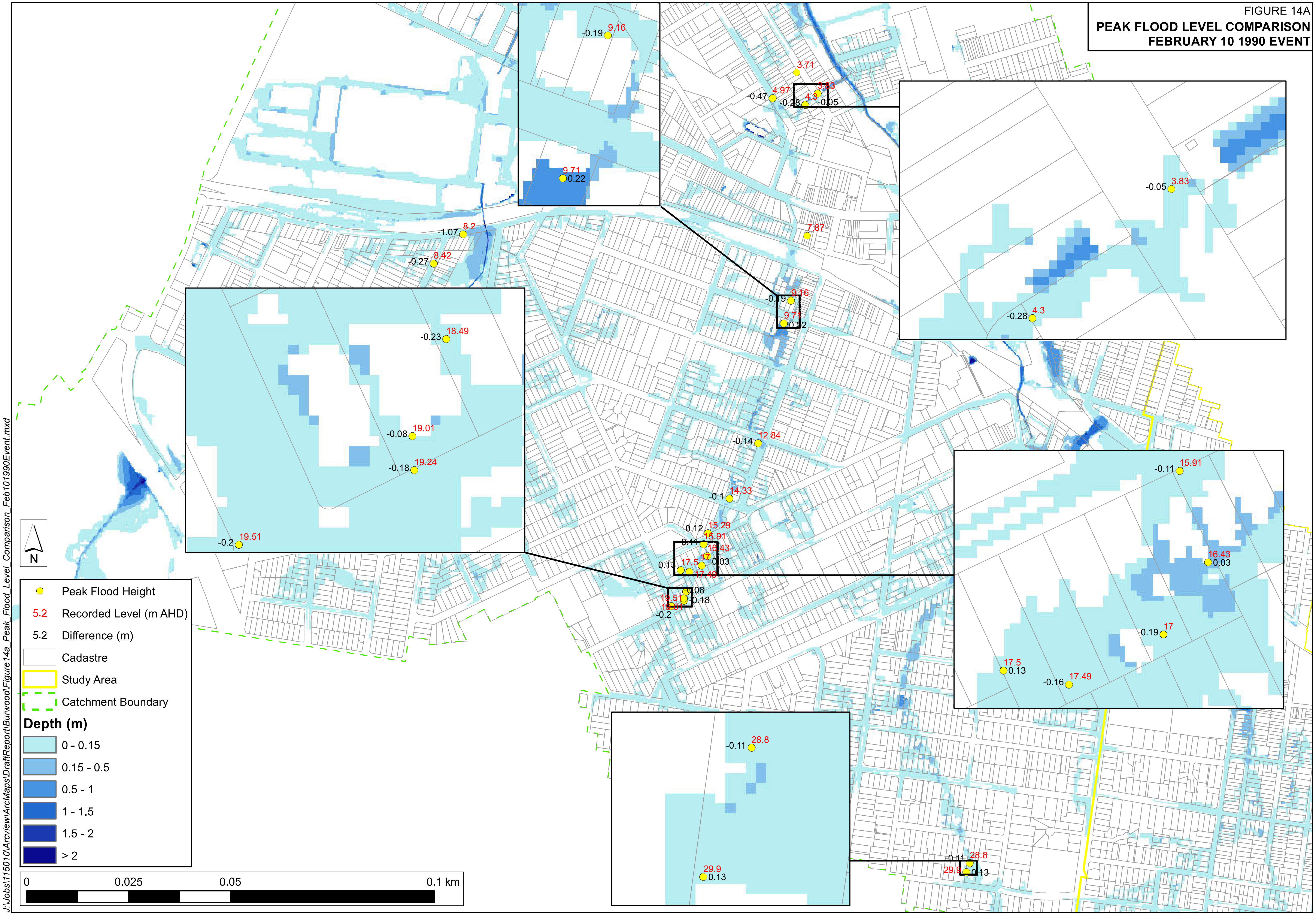

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FIGURE 14B

PEAK FLOOD LEVEL COMPARISON

JANUARY 2 1996 EVENT

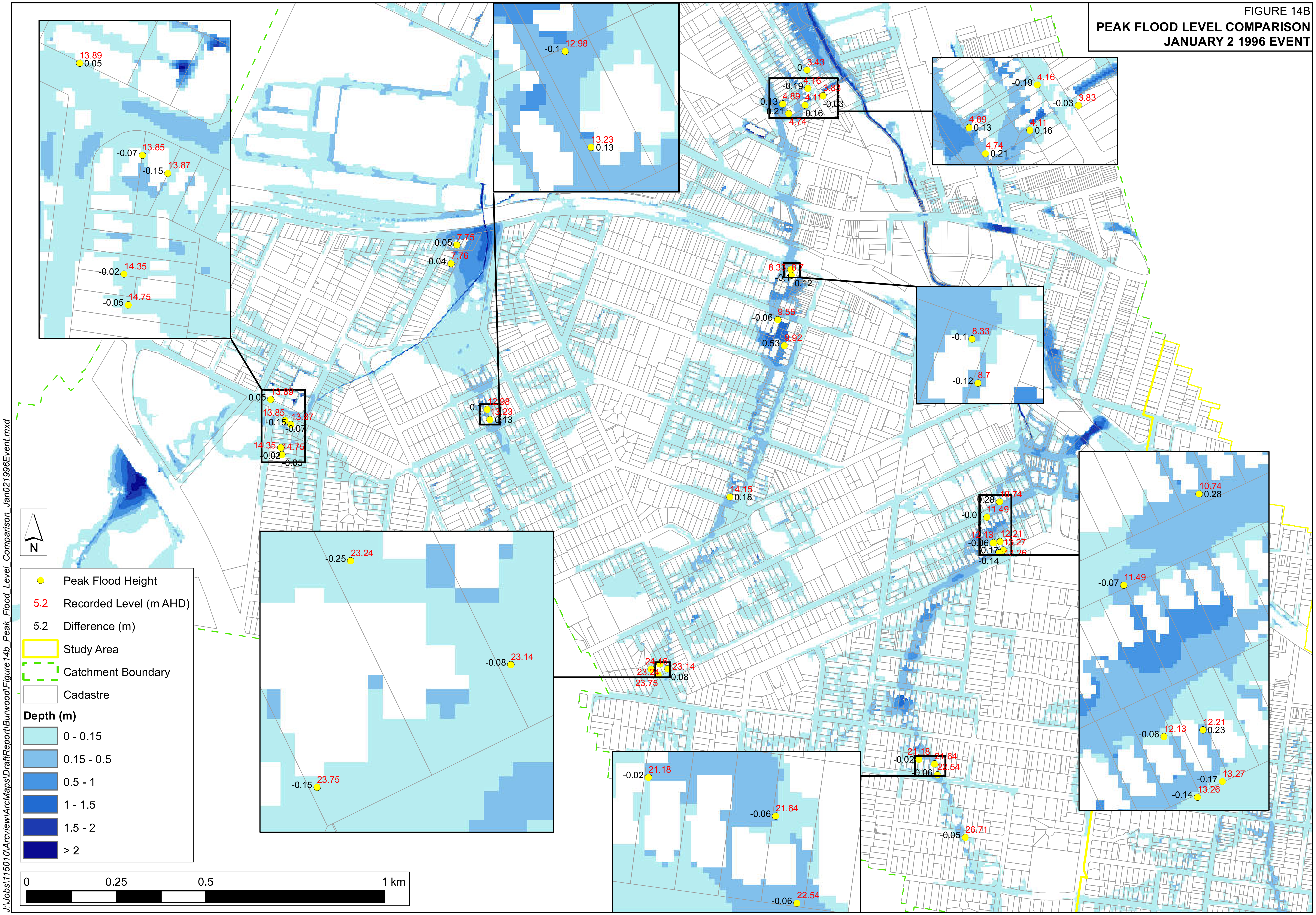

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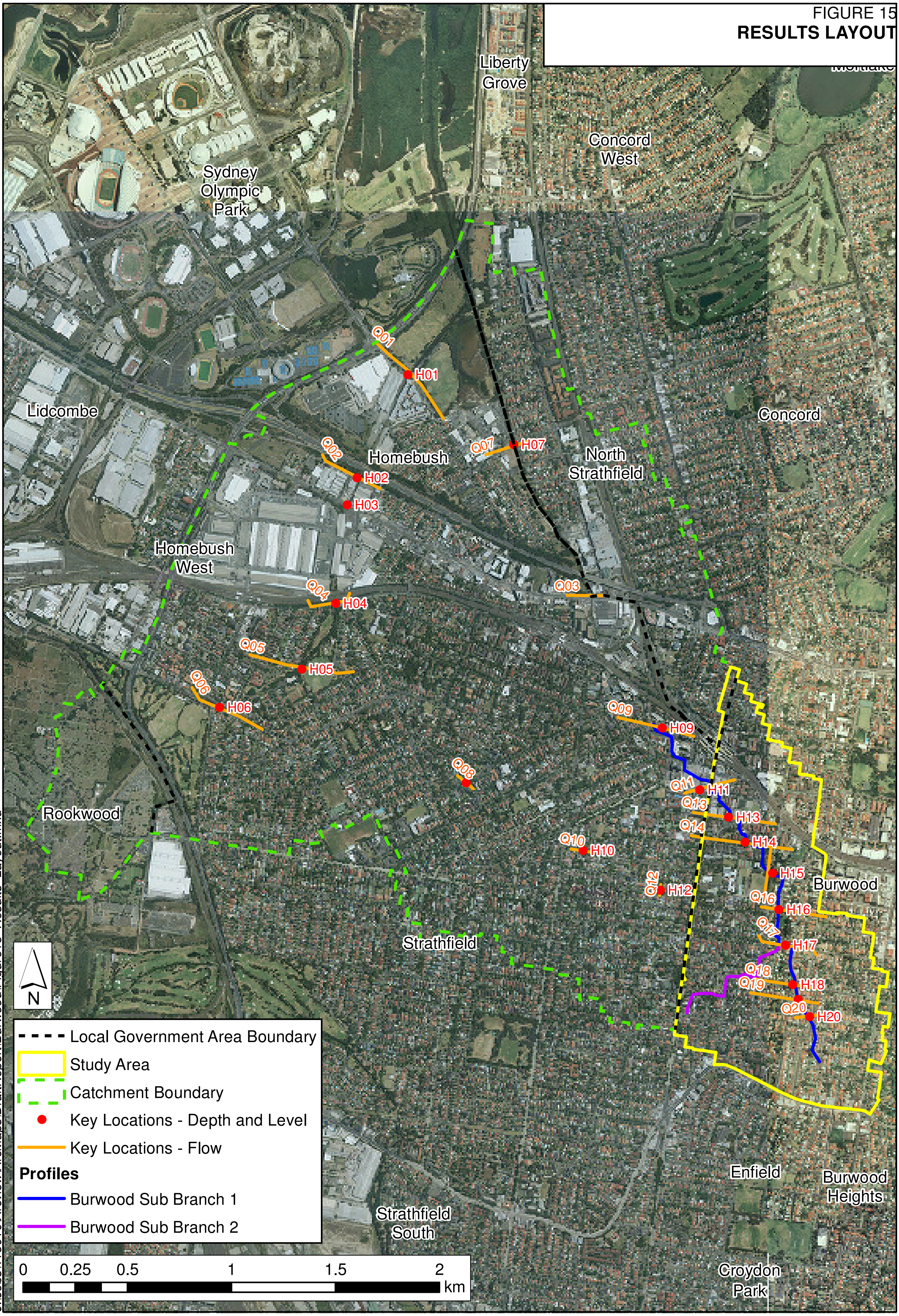
FIGURE 15
RESULTS LAYOUT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure15_Results_Layout.mxd

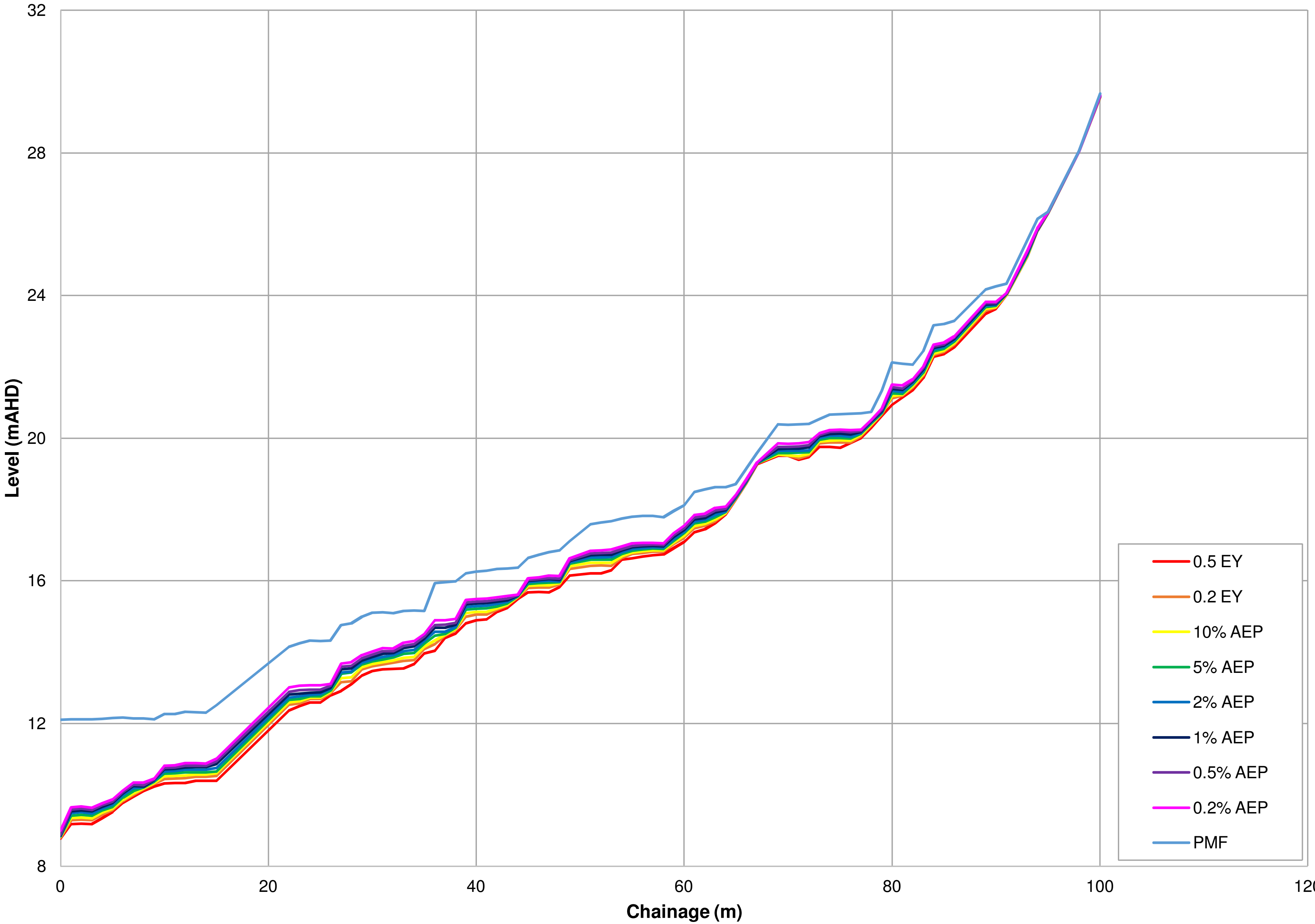
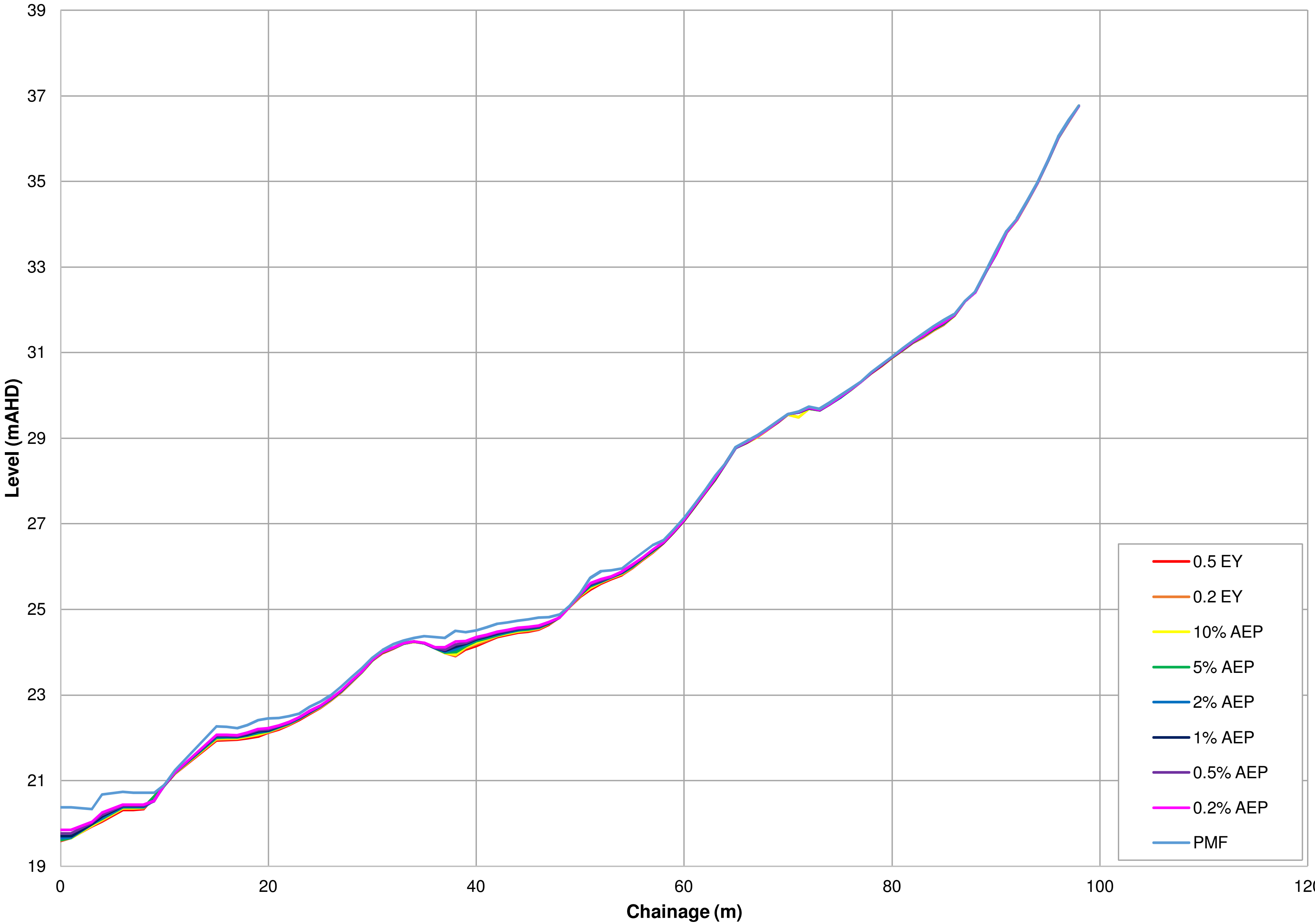


FIGURE 16A
DESIGN RESULTS
PEAK HEIGHT PROFILES - BURWOOD SUB BRANCH 1

J:\Jobs\115010\TUFLOW\results\Profile\Profile2_005.xlsx

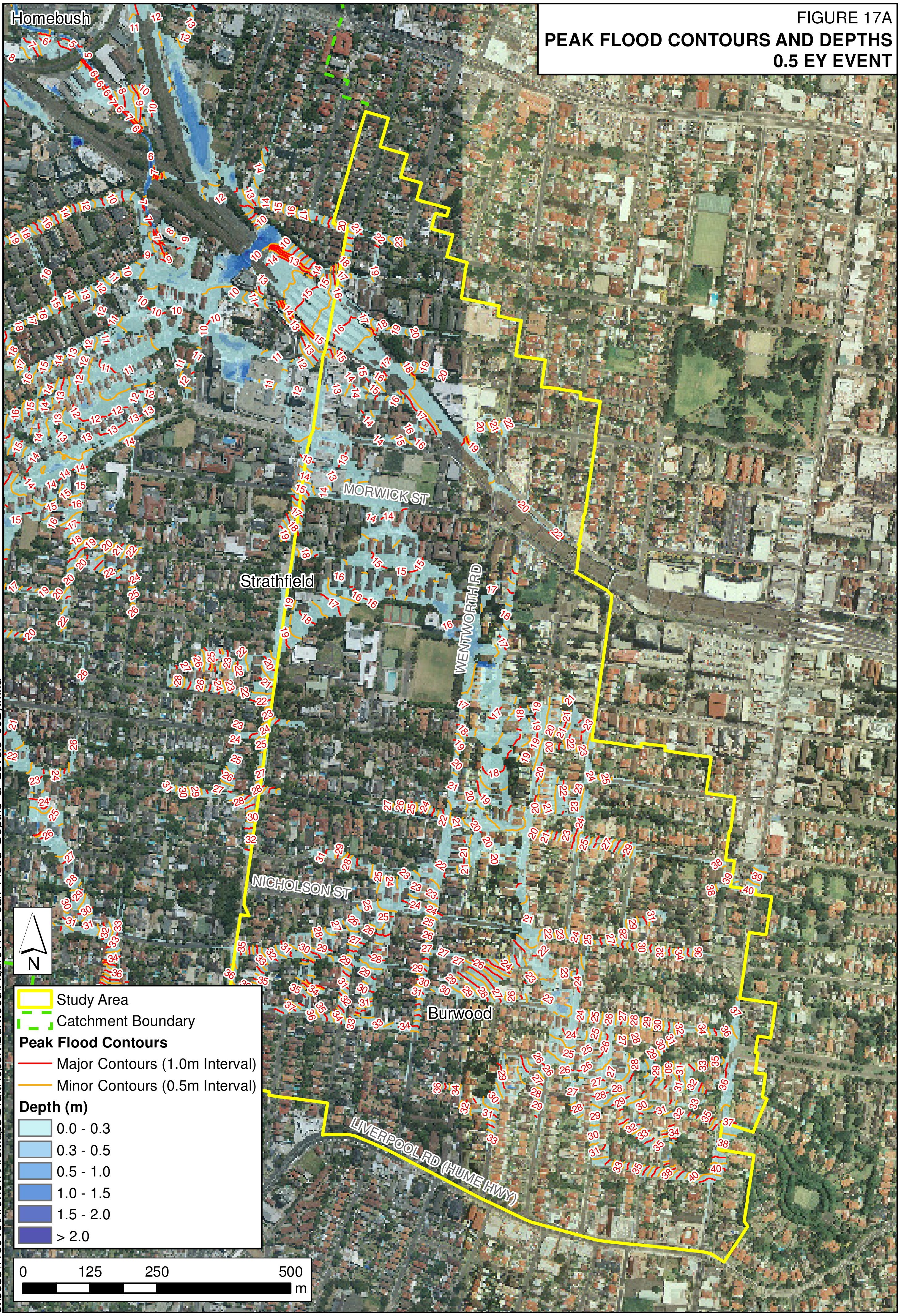
FIGURE 16B
DESIGN RESULTS
PEAK HEIGHT PROFILES - BURWOOD SUB BRANCH 2



J:\Jobs\115010\TUFLOW\results\Profile\Profile2_005.xlsx

FIGURE 17A

PEAK FLOOD CONTOURS AND DEPTHS
0.5 EY EVENT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17a Peak Flood Depths & Levels 002y.mxd

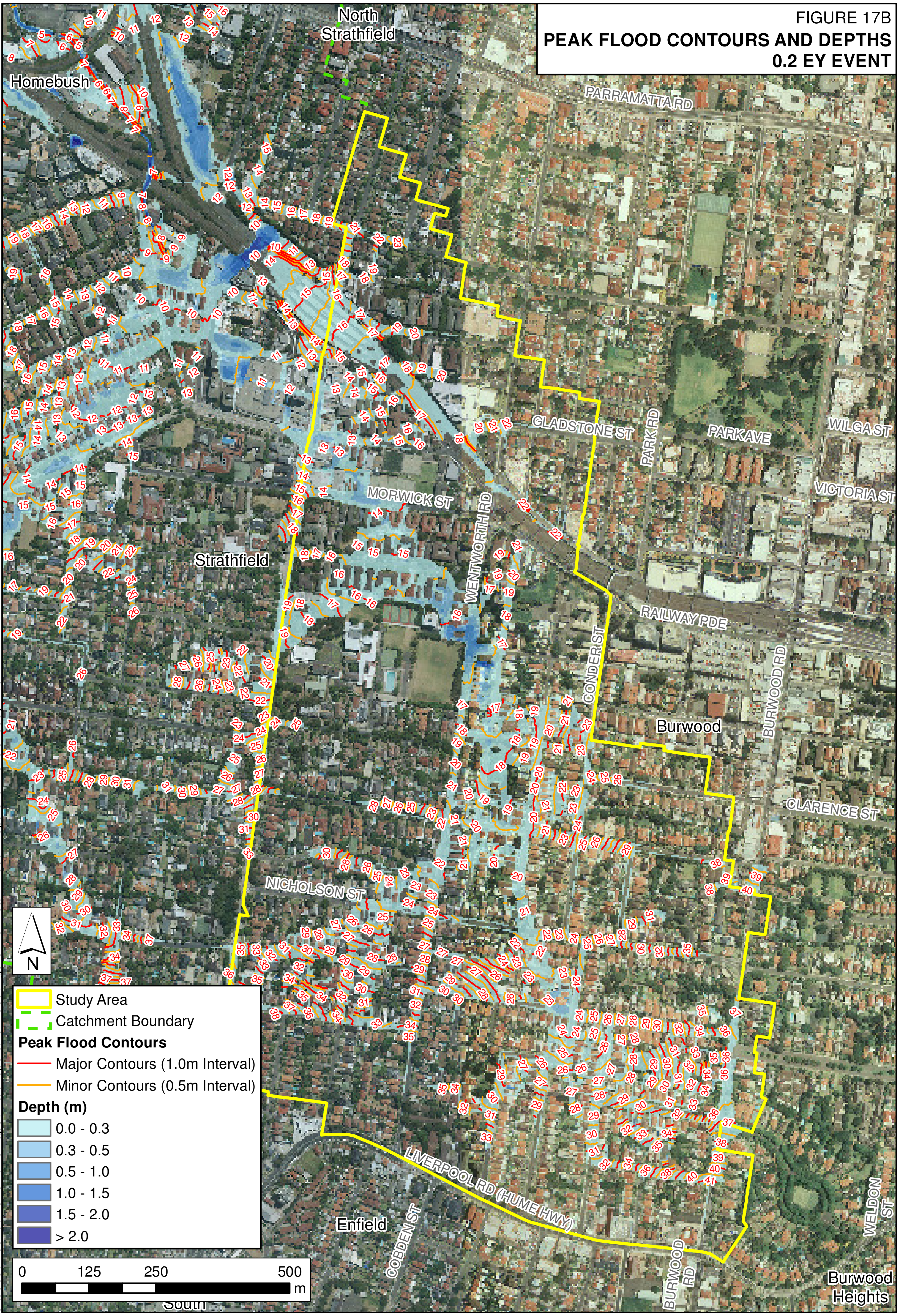
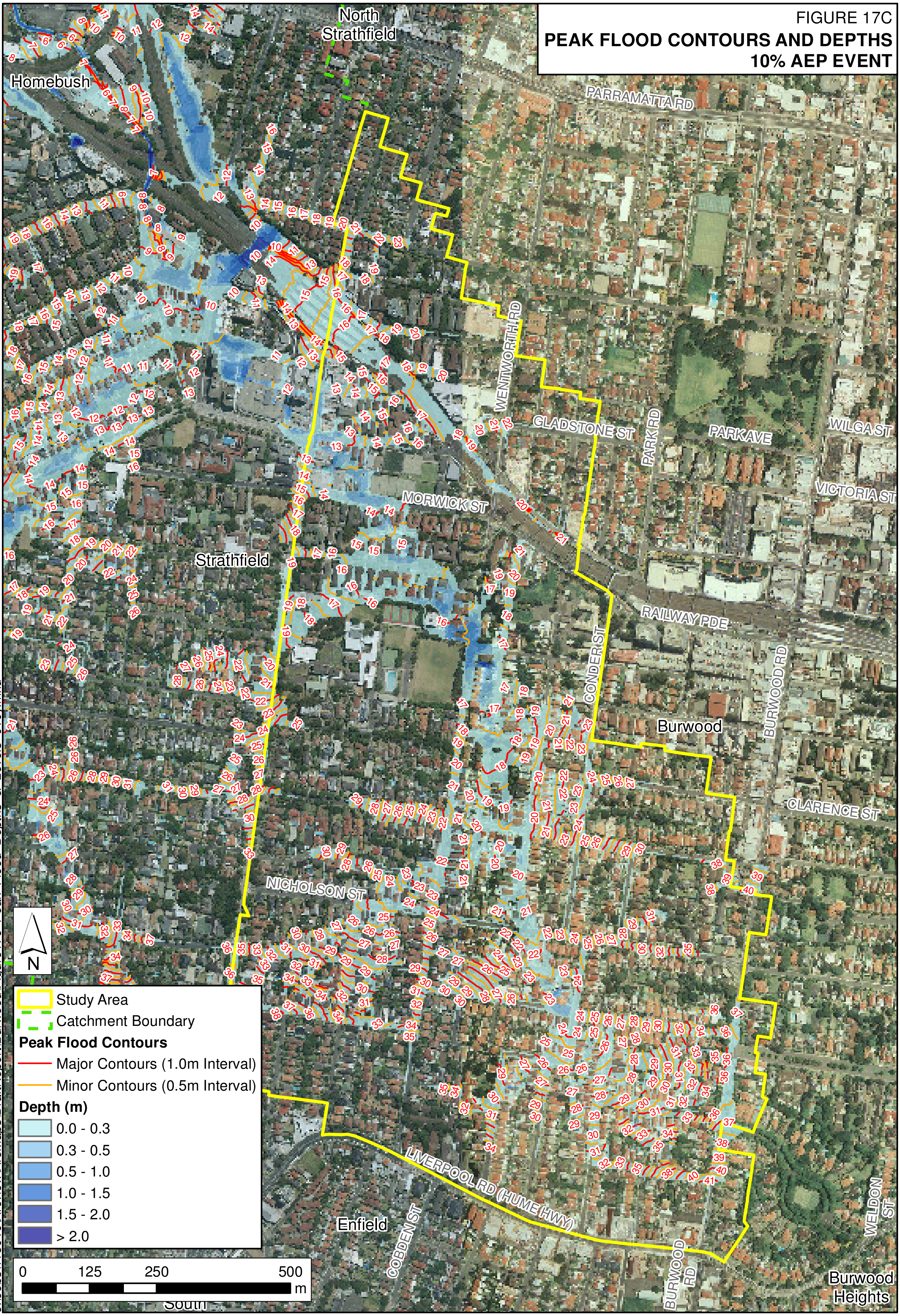


FIGURE 17B
PEAK FLOOD CONTOURS AND DEPTHS
0.2 EY EVENT
North Strathfield
Homebush
Strathfield
Burwood
Enfield
Burwood Heights
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
RAILWAY PDE
CONDER ST
BURWOOD RD
CLARENCE ST
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
COBDEN ST
WELDON ST
South
N
Study Area
Catchment Boundary
Peak Flood Contours
Major Contours (1.0m Interval)
Minor Contours (0.5m Interval)
Depth (m)
0.0 - 0.3
0.3 - 0.5
0.5 - 1.0
1.0 - 1.5
1.5 - 2.0
> 2.0
0
125
250
500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17b_Peak_Flood_Depths_&_Levels_005y.mxd

FIGURE 17C
PEAK FLOOD CONTOURS AND DEPTHS
10% AEP EVENT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17c Peak Flood Depths & Levels 010v.mxd

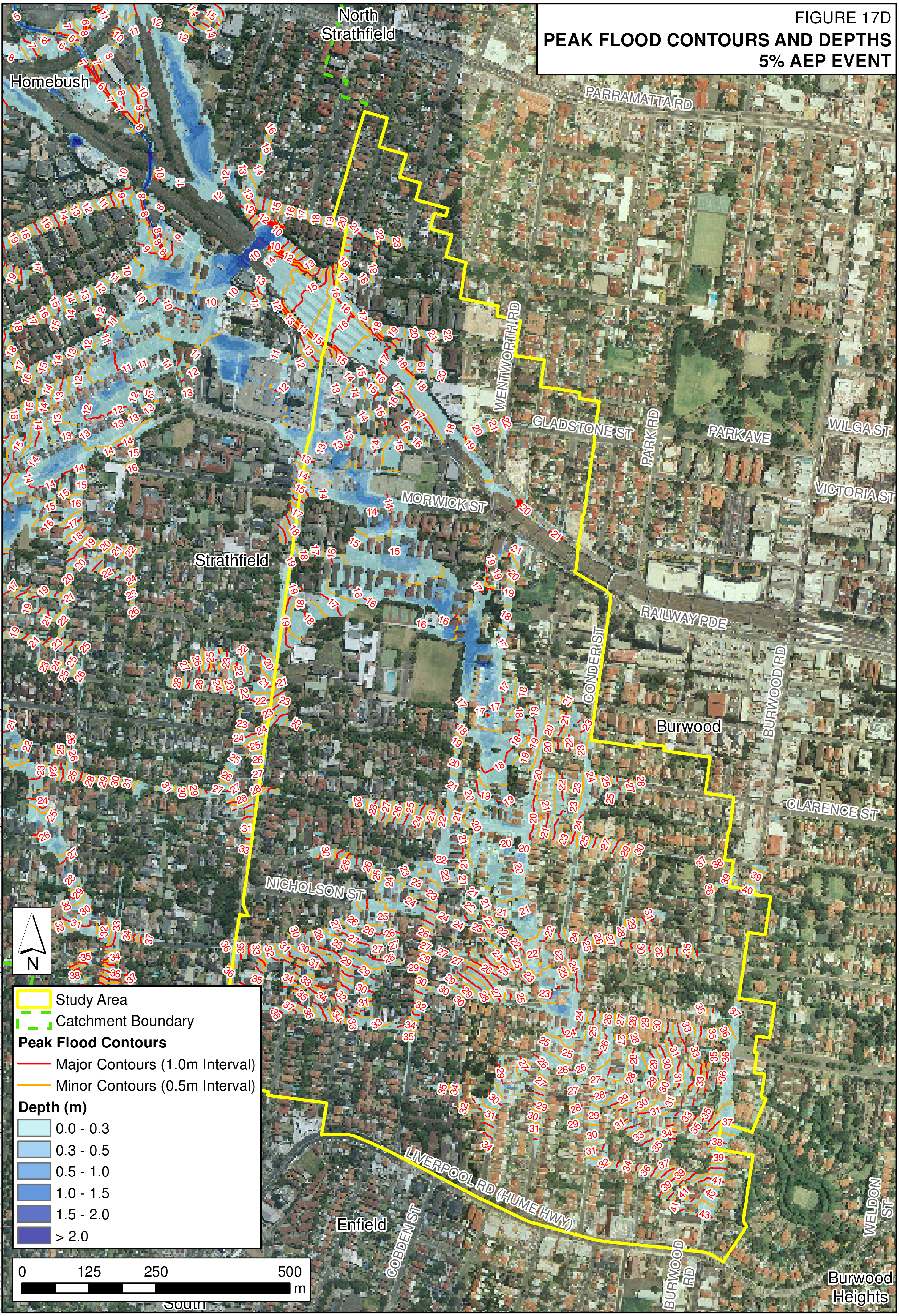
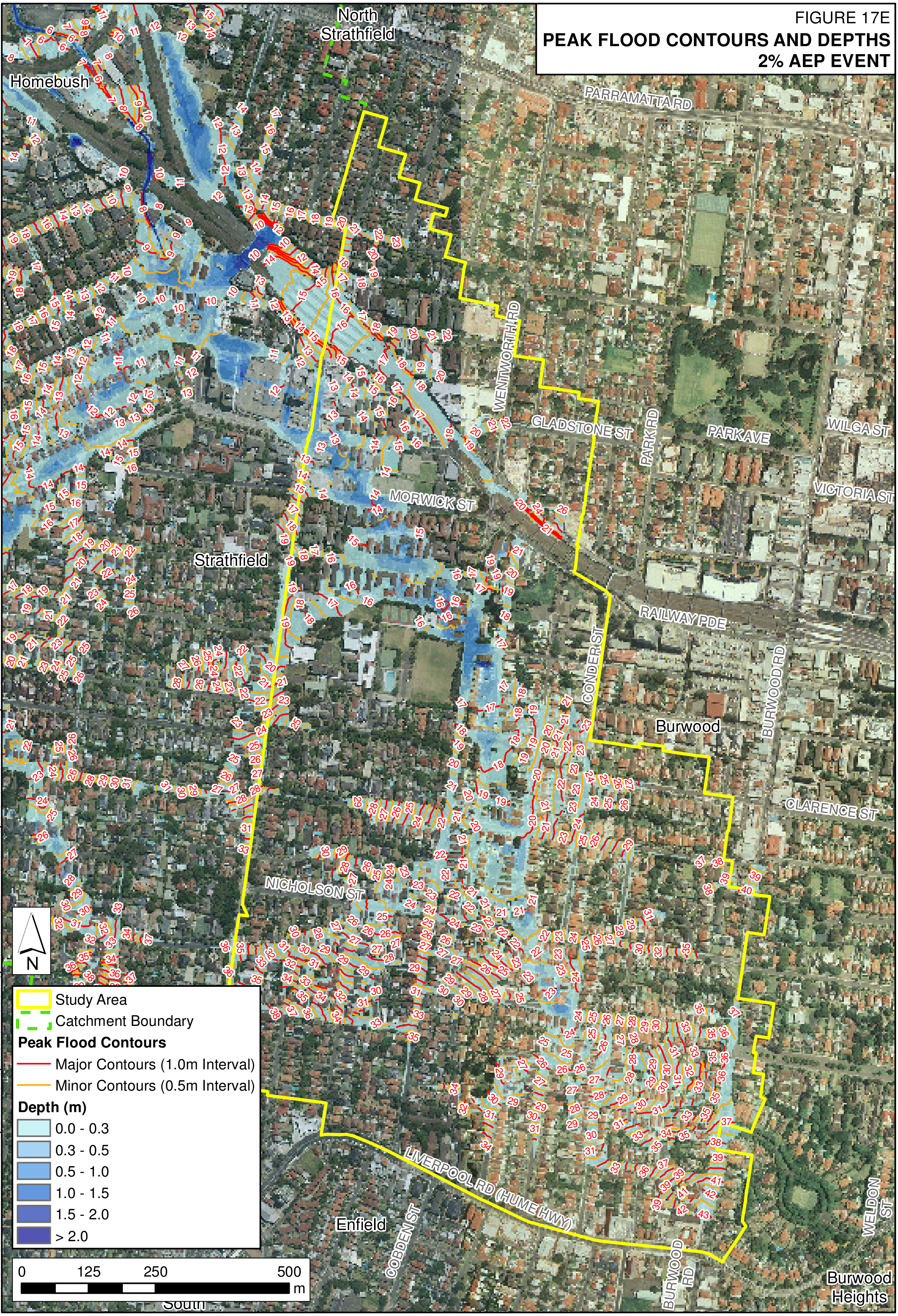


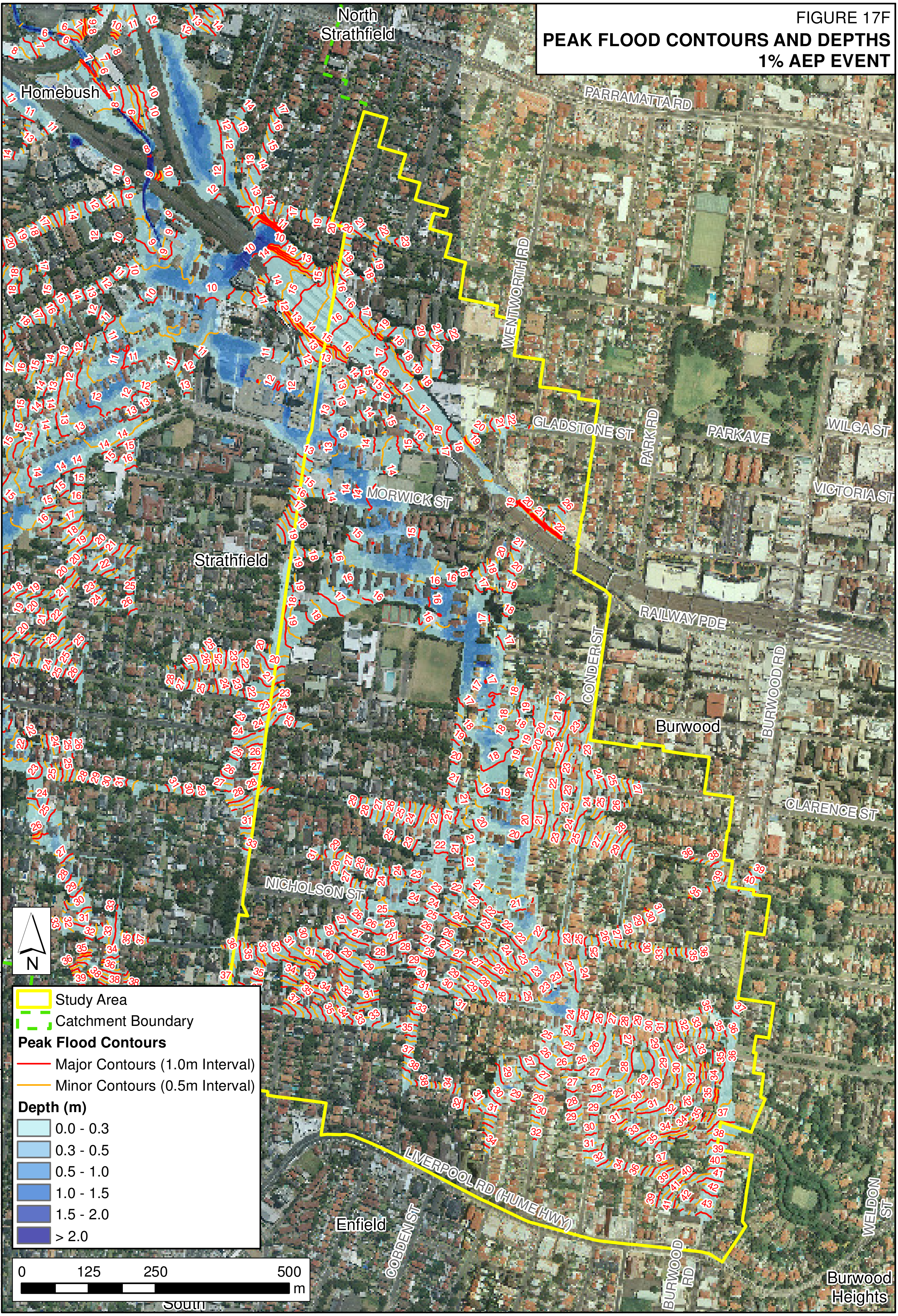
FIGURE 17D
PEAK FLOOD CONTOURS AND DEPTHS
5% AEP EVENT

J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17d_Peak_Flood_Depths_&_Levels_020y.mxd



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17e Peak Flood Depths & Levels 050y.mxd

FIGURE 17F
PEAK FLOOD CONTOURS AND DEPTHS
1% AEP EVENT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17f Peak Flood Depths & Levels 100y.mxd

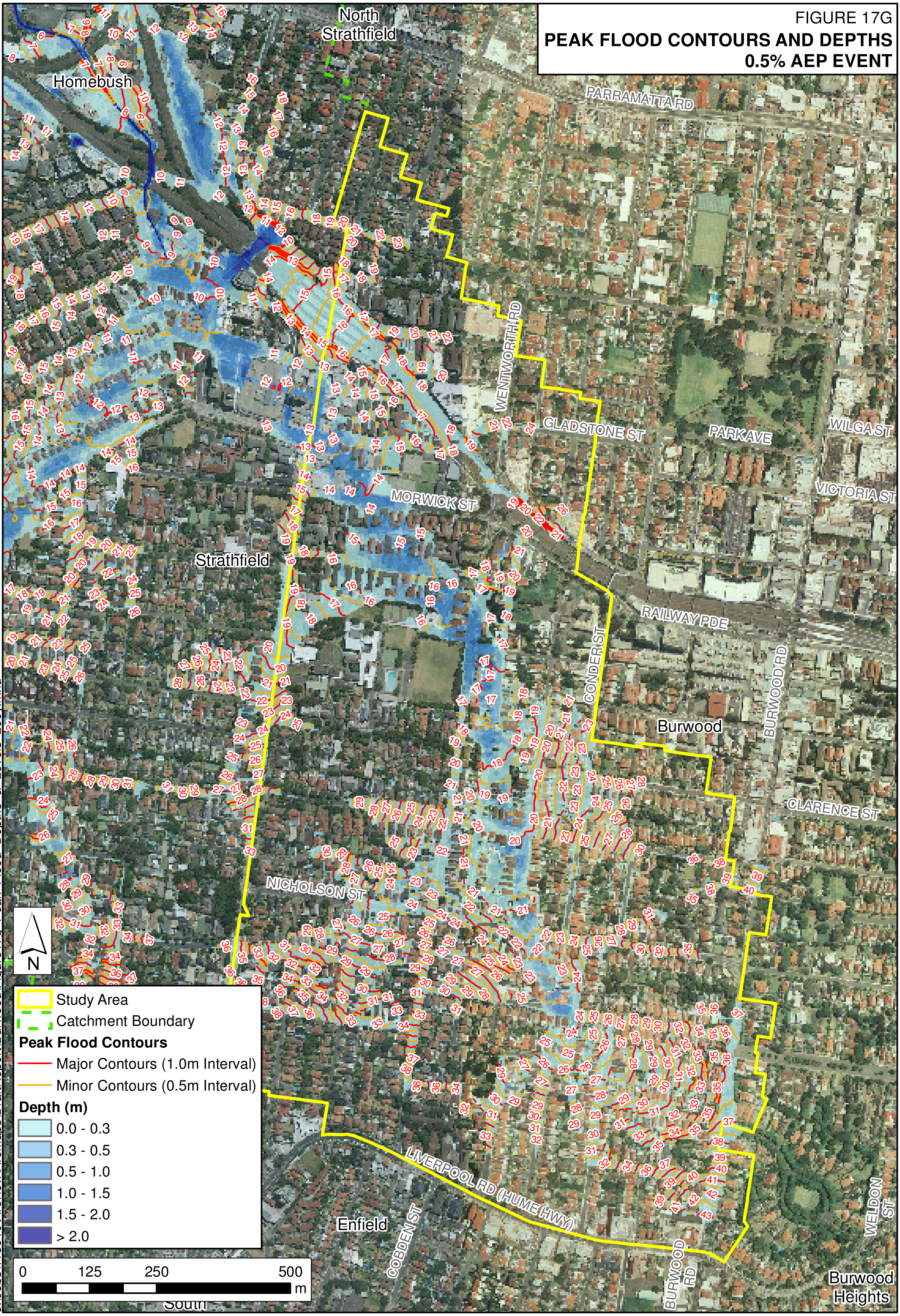
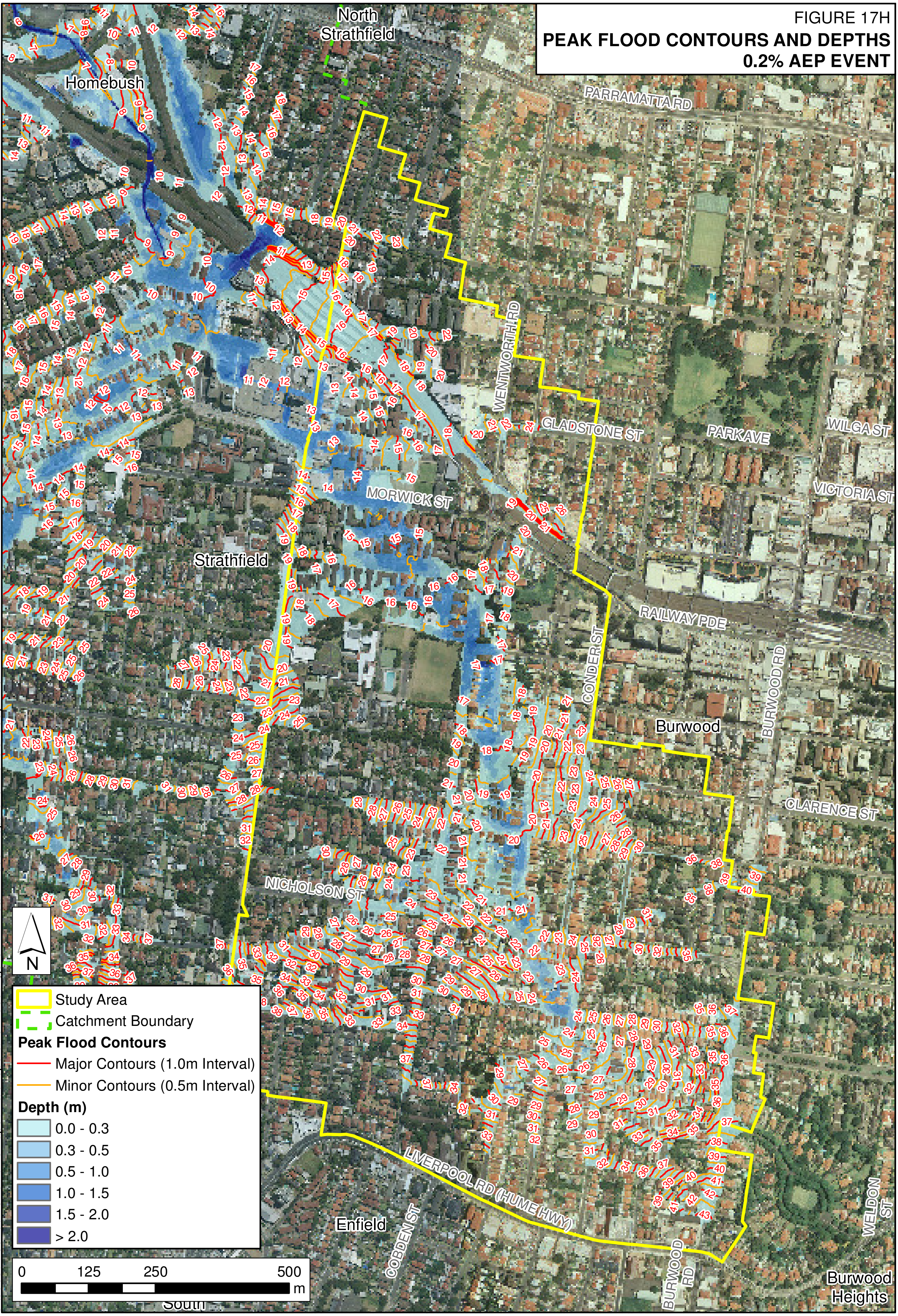


FIGURE 17G
PEAK FLOOD CONTOURS AND DEPTHS
0.5% AEP EVENT

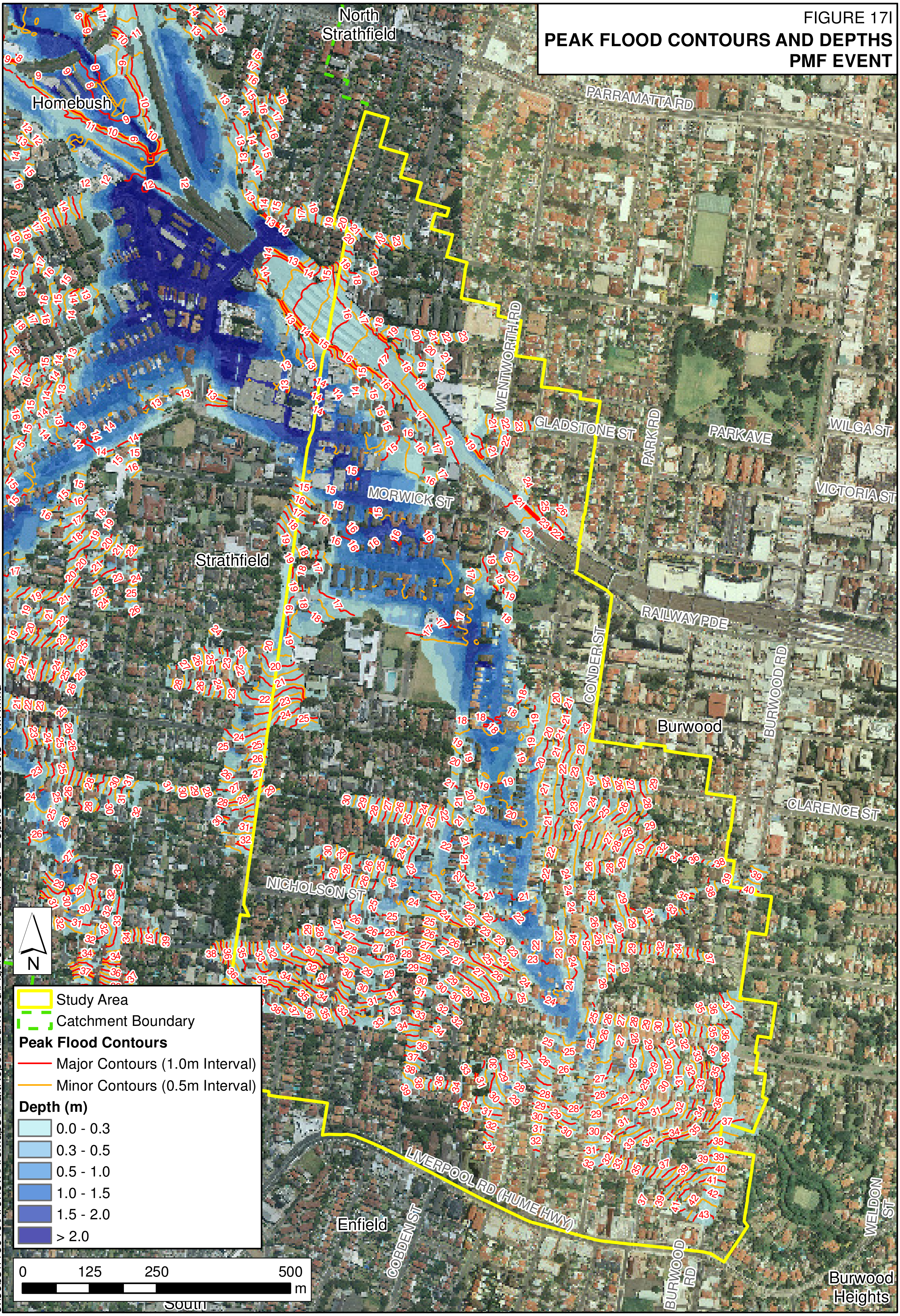
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17g Peak Flood Depths & Levels 200y.mxd

FIGURE 17H
PEAK FLOOD CONTOURS AND DEPTHS
0.2% AEP EVENT



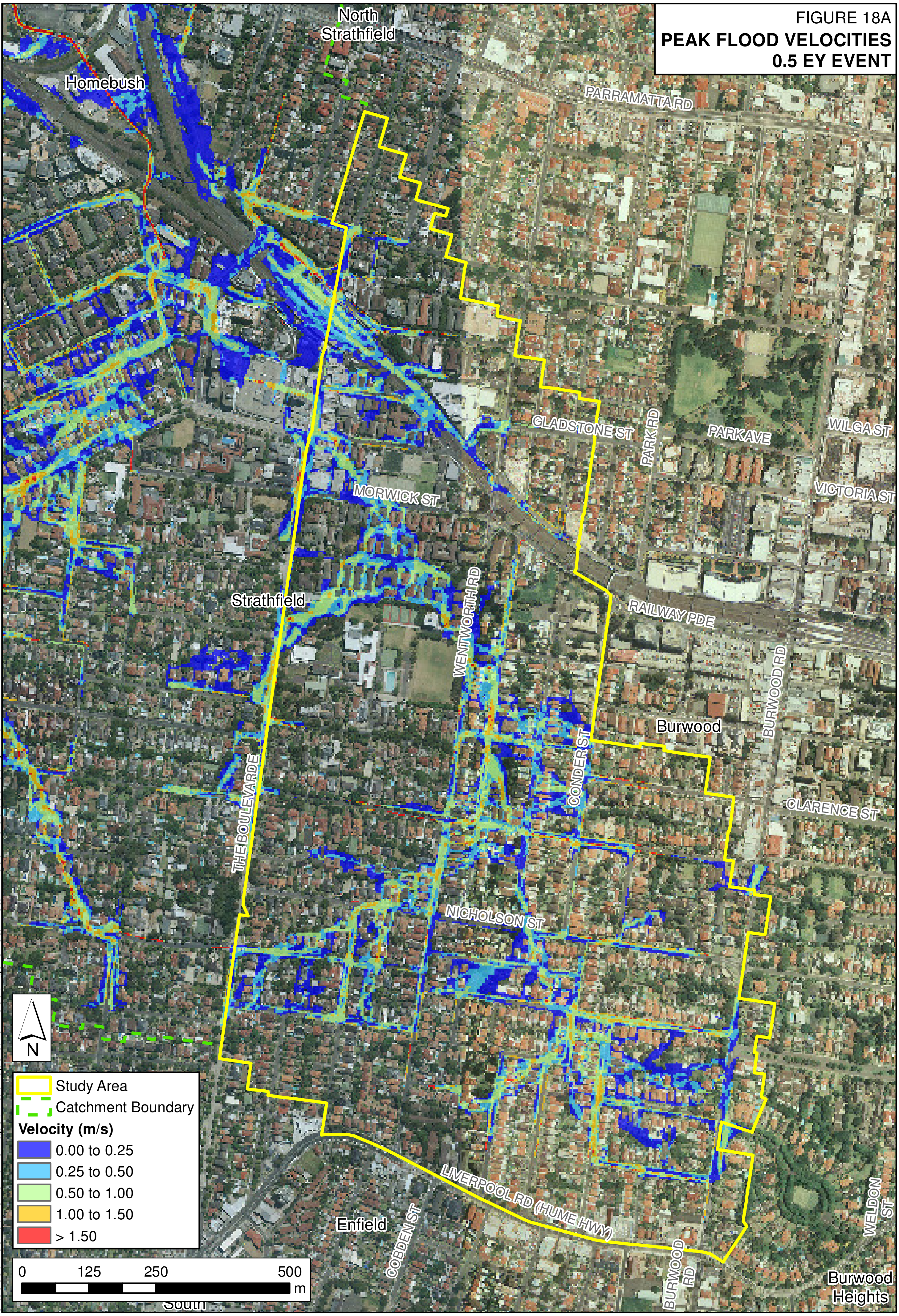
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17h Peak Flood Depths & Levels 500y.mxd

FIGURE 17I
PEAK FLOOD CONTOURS AND DEPTHS PMF EVENT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure17i Peak Flood Depths & Levels PMF.mxd

FIGURE 18A
PEAK FLOOD VELOCITIES
0.5 EY EVENT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18a_Peak_Flood_Velocities_002y.mxd

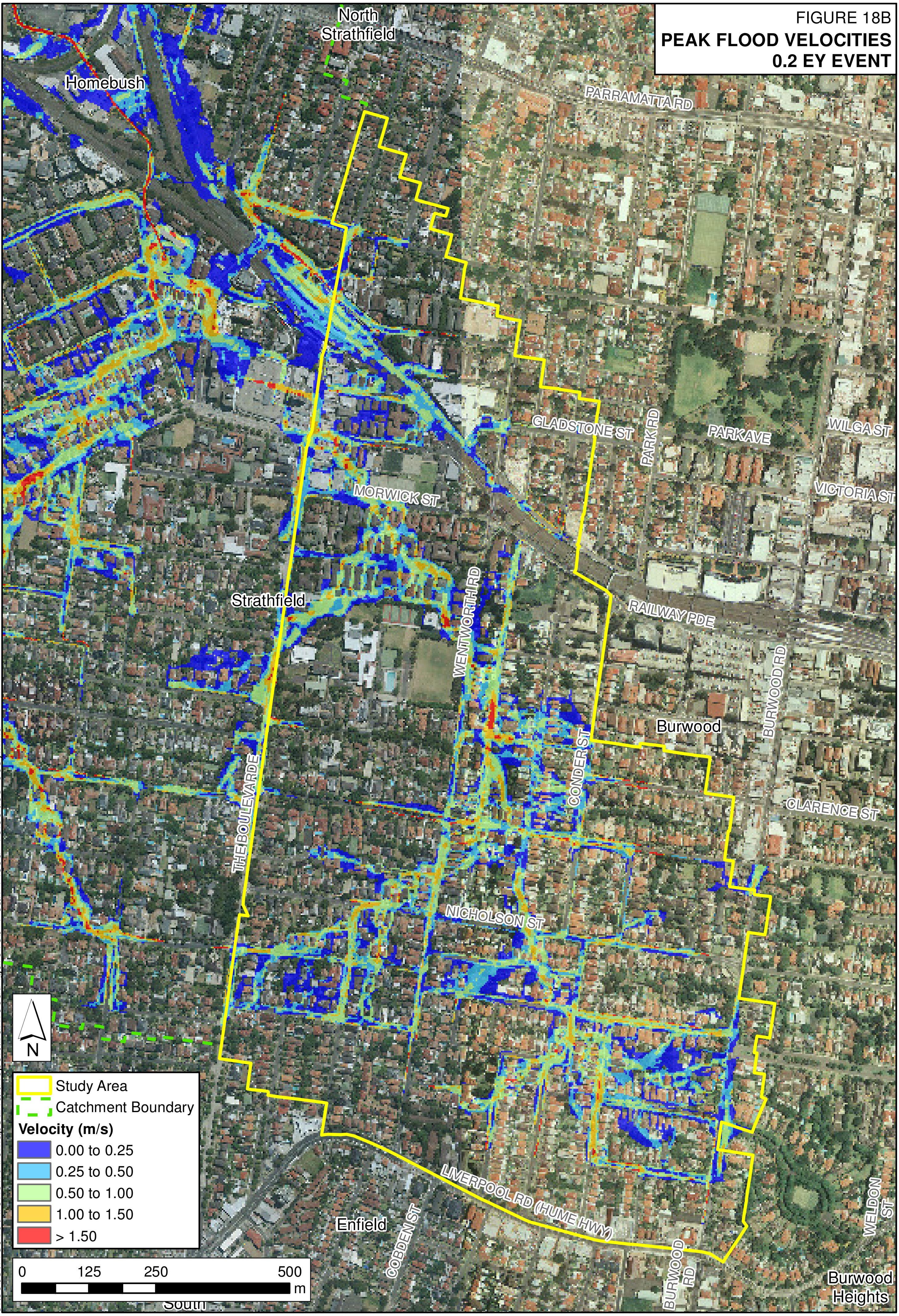


FIGURE 18B
PEAK FLOOD VELOCITIES
0.2 EY EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
Strathfield
WENTWORTH RD
RAILWAY PDE
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
N
Study Area
Catchment Boundary
Velocity (m/s)
0.00 to 0.25
0.25 to 0.50
0.50 to 1.00
1.00 to 1.50
> 1.50
0 125 250 500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18b_Peak_Flood_Velocities_005y.mxd

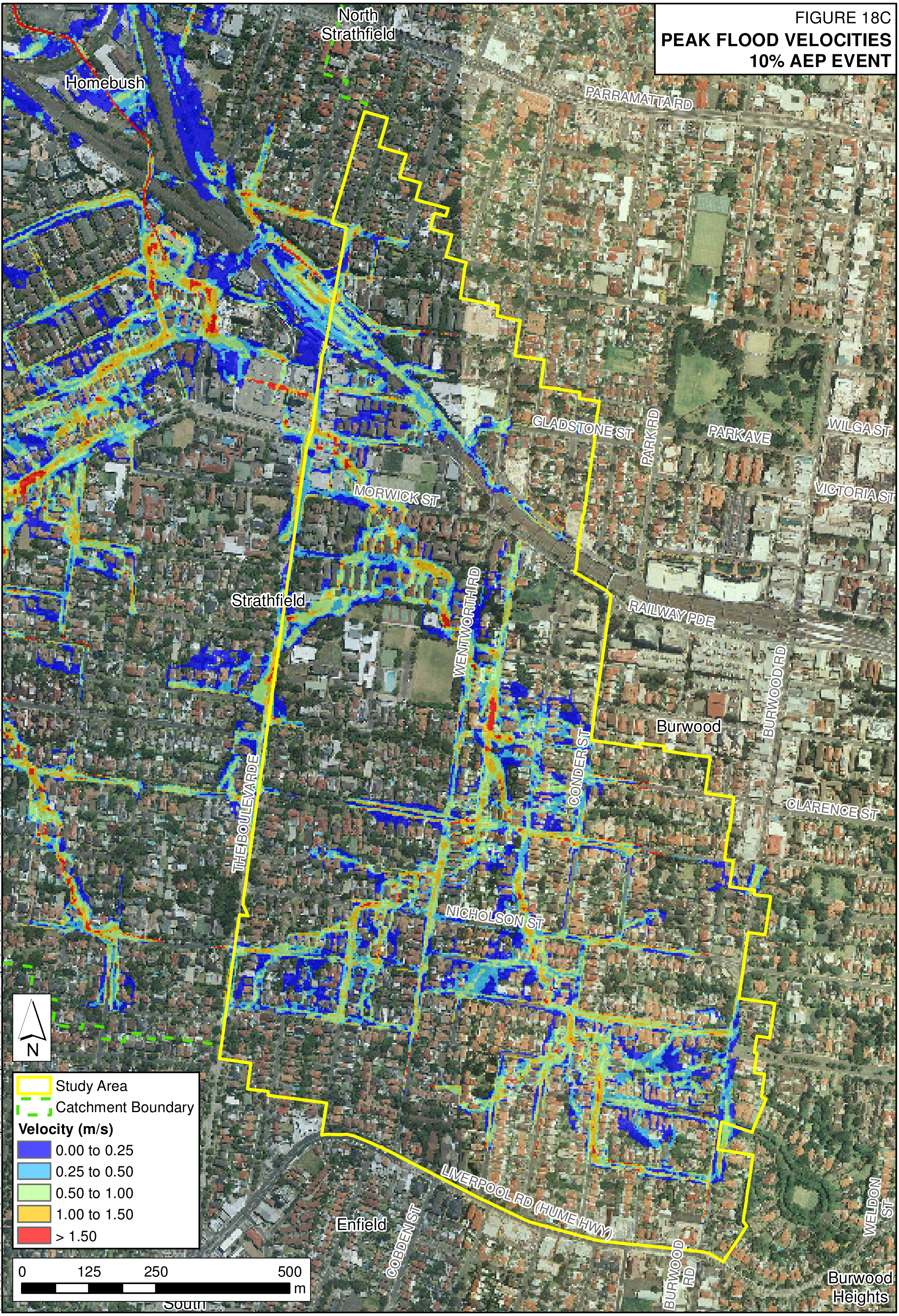
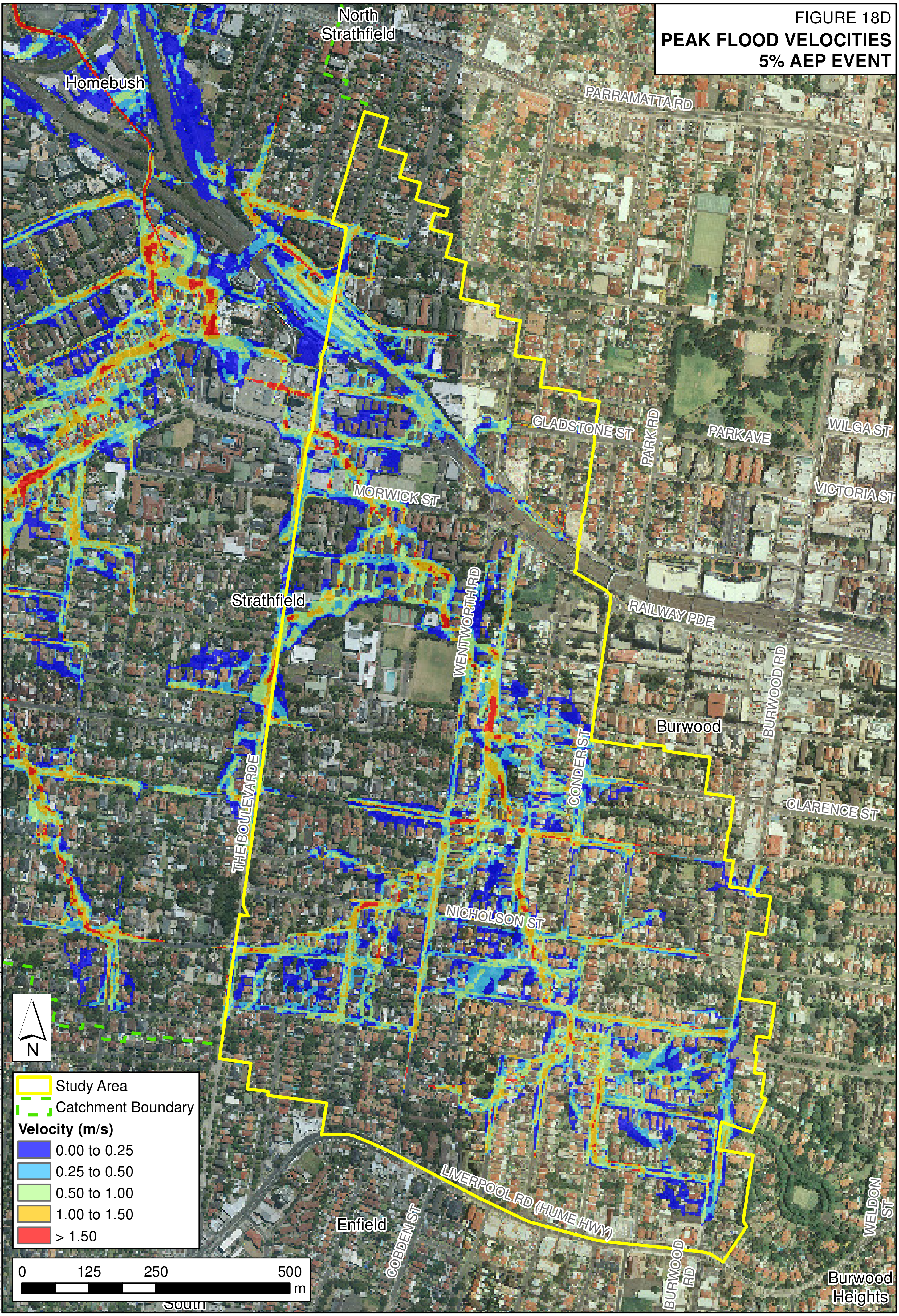


FIGURE 18C
PEAK FLOOD VELOCITIES
10% AEP EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
RAILWAY PDE
Strathfield
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
N
Study Area
Catchment Boundary
Velocity (m/s)
0.00 to 0.25
0.25 to 0.50
0.50 to 1.00
1.00 to 1.50
> 1.50
0 125 250 500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18c Peak Flood Velocities 010v.mxd

FIGURE 18D
PEAK FLOOD VELOCITIES
5% AEP EVENT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18d_Peak_Flood_Velocities_020y.mxd

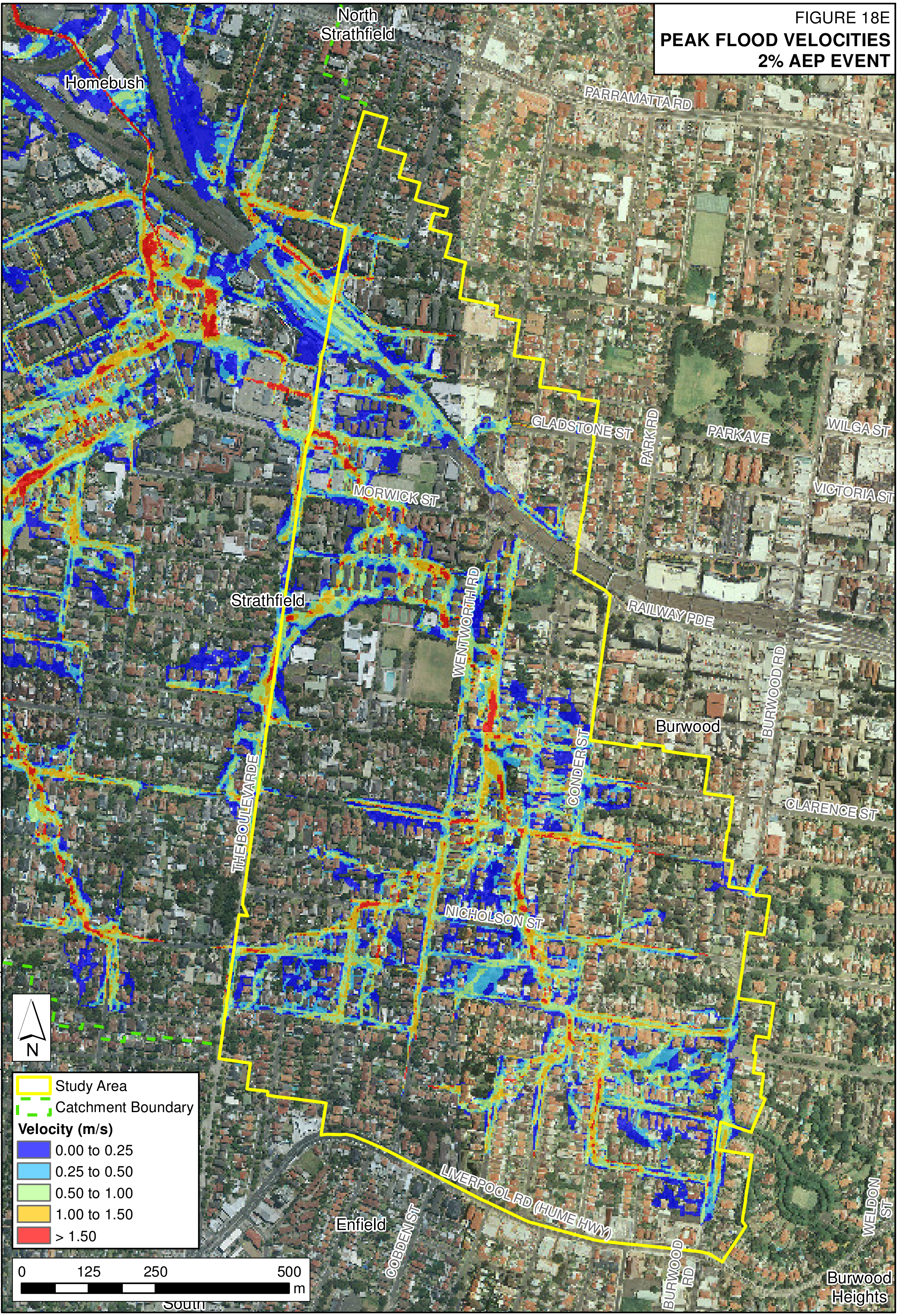


FIGURE 18E
PEAK FLOOD VELOCITIES
2% AEP EVENT
North
Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
Strathfield
RAILWAY PDE
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood
Heights
South
N
Study Area
Catchment Boundary
Velocity (m/s)
0.00 to 0.25
0.25 to 0.50
0.50 to 1.00
1.00 to 1.50
> 1.50
0
125
250
500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18e_Peak_Flood_Velocities_050y.mxd

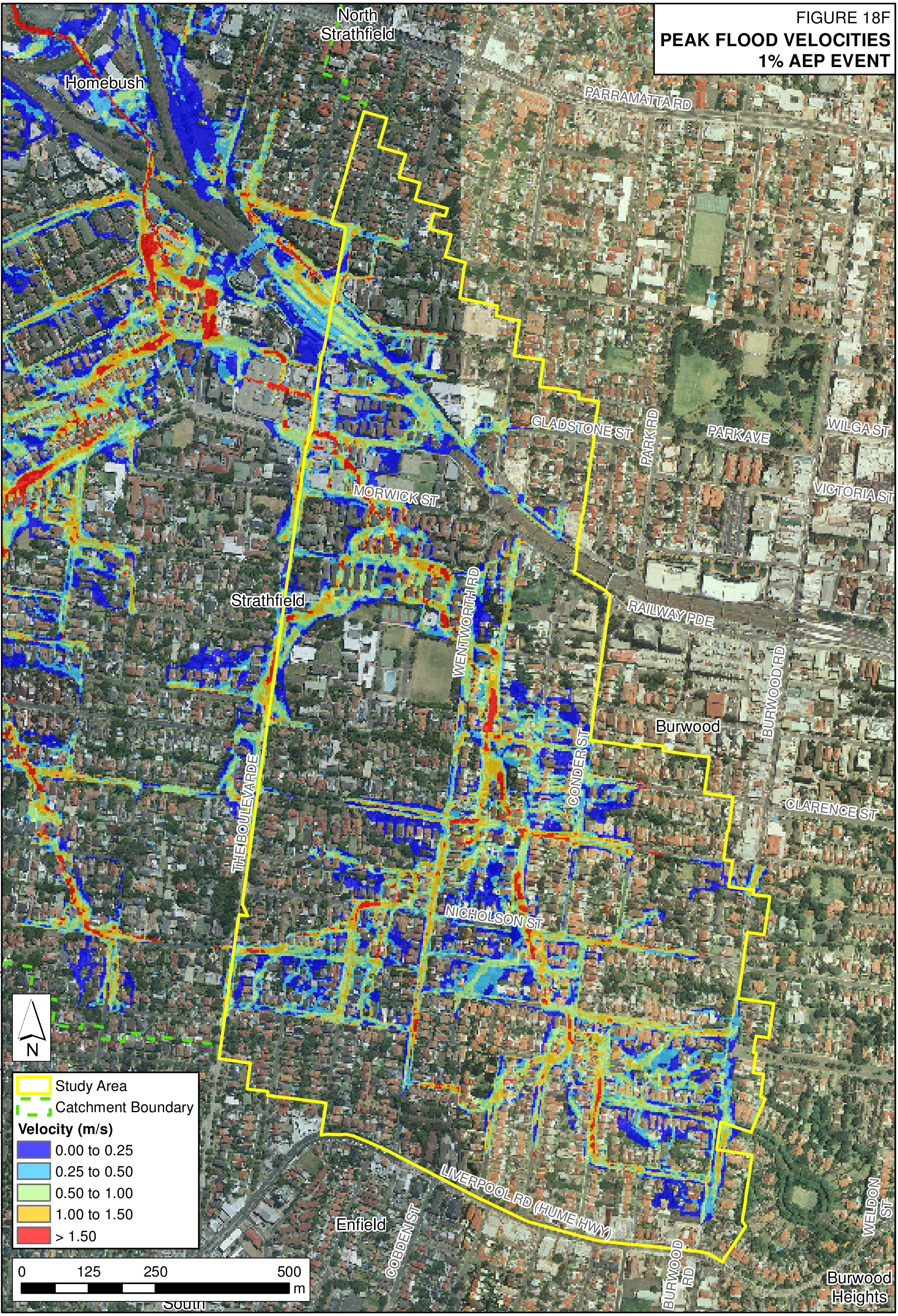
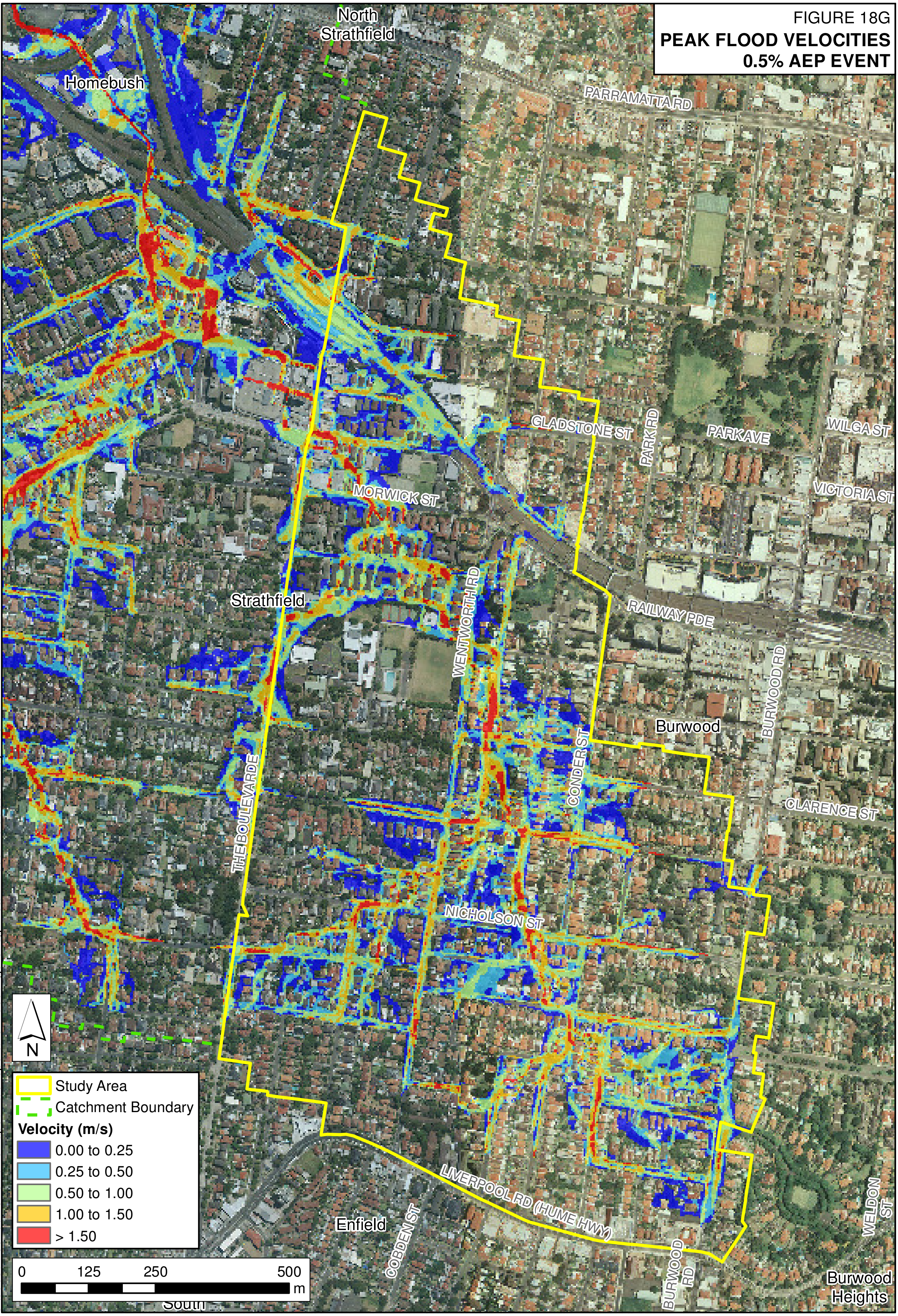


FIGURE 18F
PEAK FLOOD VELOCITIES
1% AEP EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
RAILWAY PDE
Strathfield
Burwood
BURWOOD RD
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
N
Study Area
Catchment Boundary
Velocity (m/s)
0.00 to 0.25
0.25 to 0.50
0.50 to 1.00
1.00 to 1.50
> 1.50
0 125 250 500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18f Peak Flood Velocities 100y.mxd

FIGURE 18G
PEAK FLOOD VELOCITIES
0.5% AEP EVENT



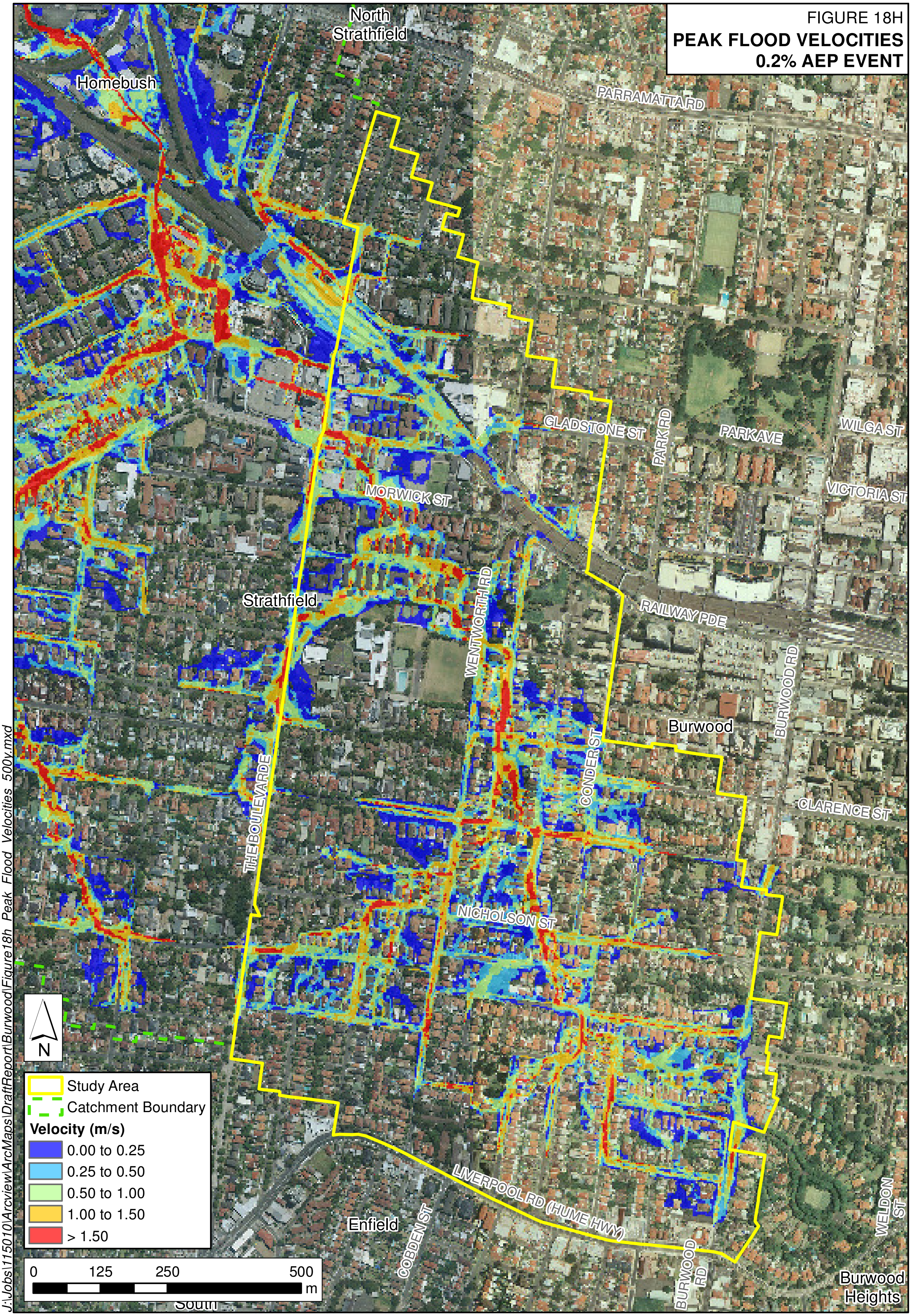


FIGURE 18H
PEAK FLOOD VELOCITIES
0.2% AEP EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
Strathfield
RAILWAY PDE
WENTWORTH RD
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
Study Area
Catchment Boundary
Velocity (m/s)
0.00 to 0.25
0.25 to 0.50
0.50 to 1.00
1.00 to 1.50
> 1.50
0
125
250
500
m
N
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18h Peak Flood Velocities 500y.mxd

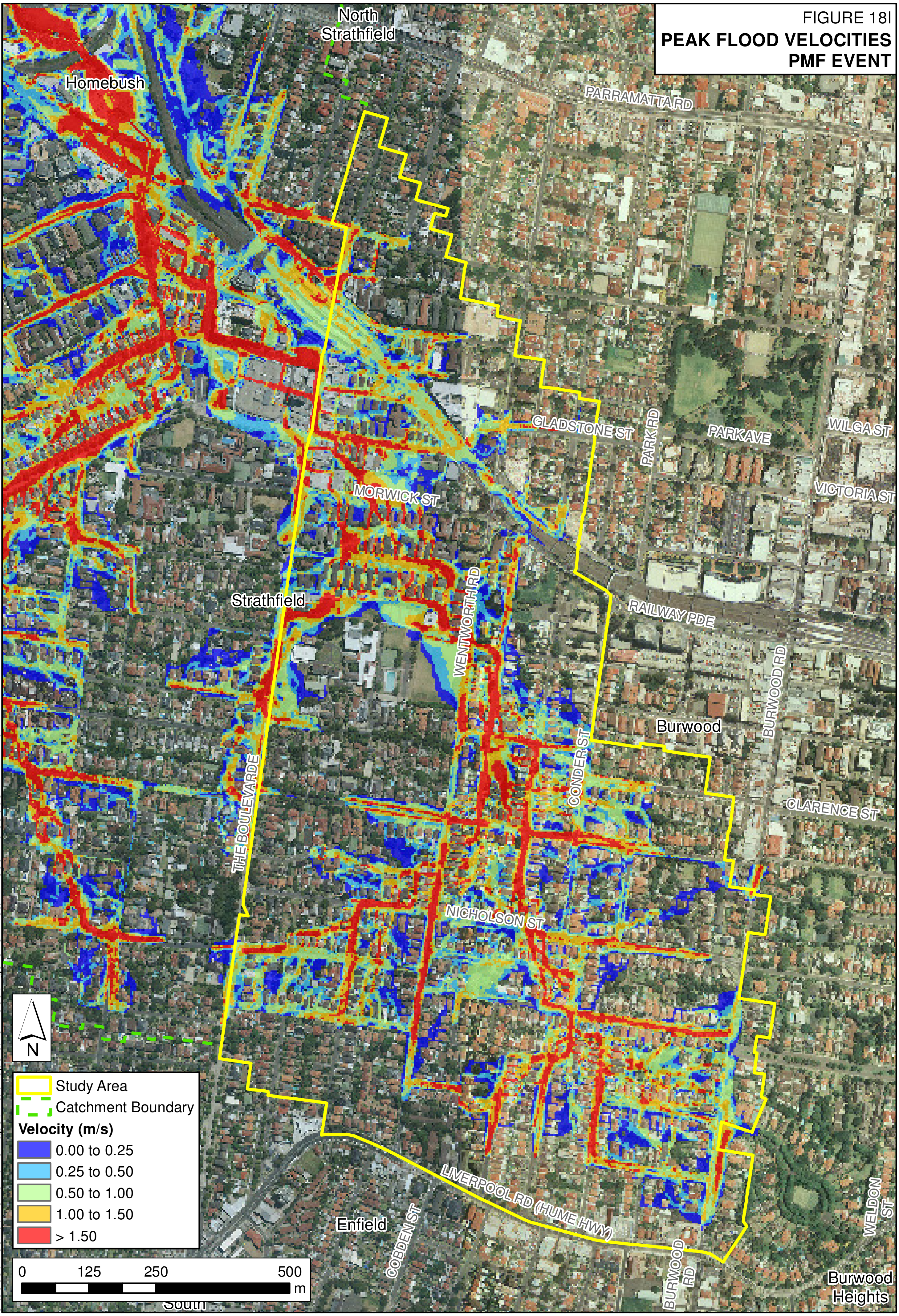


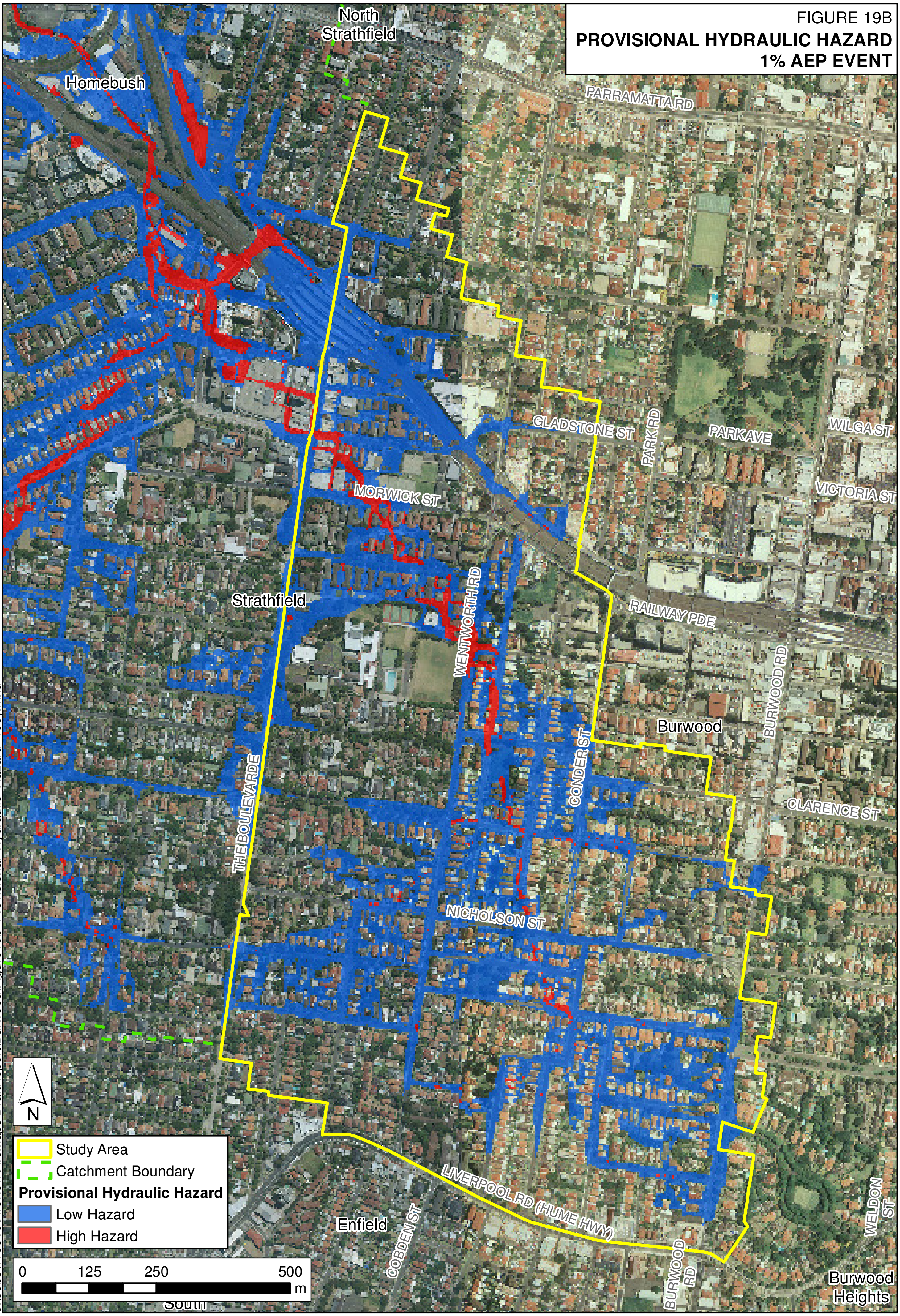
FIGURE 18I
PEAK FLOOD VELOCITIES
PMF EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
RAILWAY PDE
Strathfield
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
N
Study Area
Catchment Boundary
Velocity (m/s)
0.00 to 0.25
0.25 to 0.50
0.50 to 1.00
1.00 to 1.50
> 1.50
0 125 250 500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure18i_Peak_Flood_Velocities_PMF.mxd

FIGURE 19A
PROVISIONAL HYDRAULIC HAZARD
5% AEP EVENT

J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure19a_Provisional_Hydraulic_Hazard_020v.mxd

FIGURE 19B

PROVISIONAL HYDRAULIC HAZARD 1% AEP EVENT



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure19b_Provisional_Hydraulic_Hazard_100y.mxd

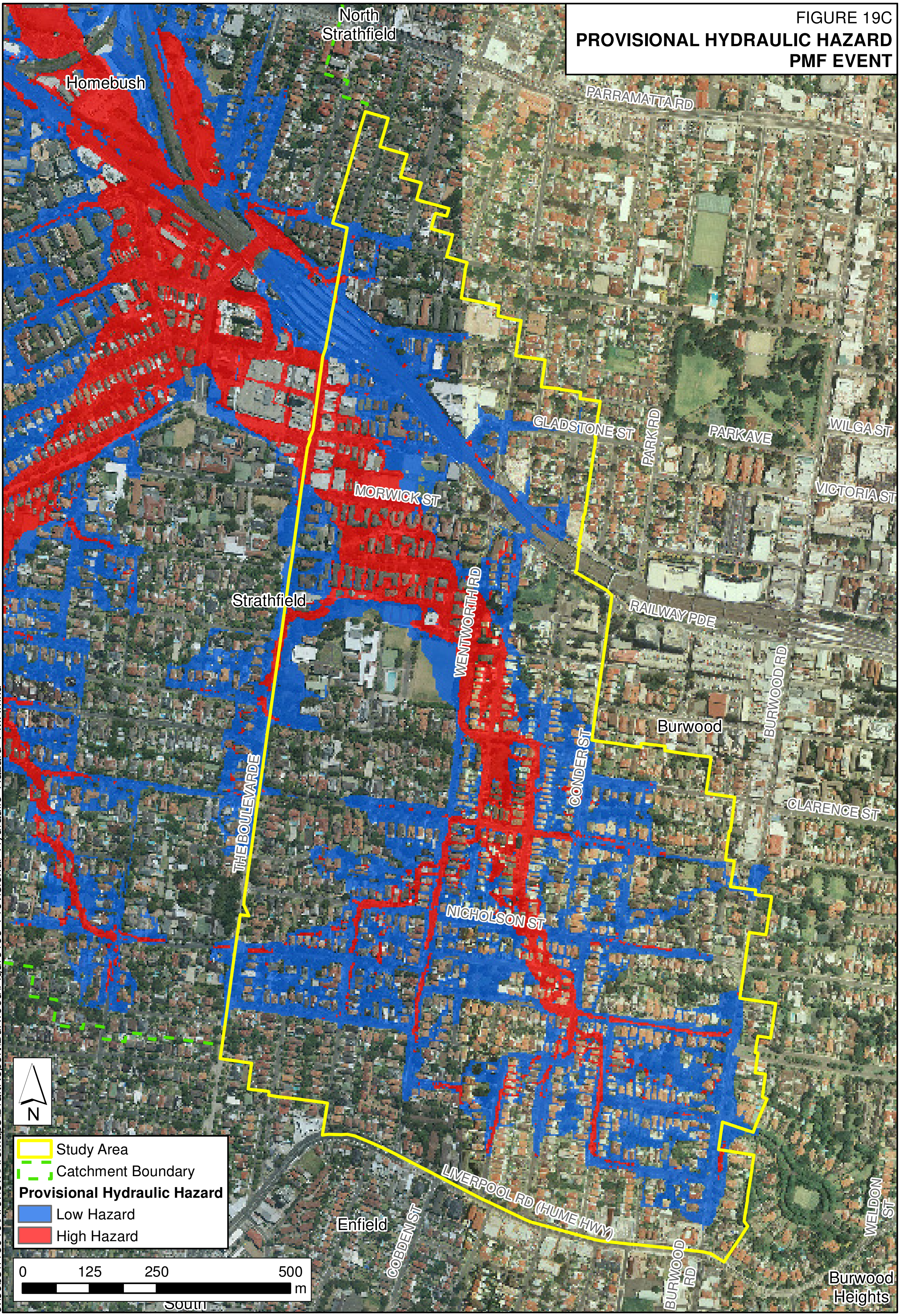


FIGURE 19C
PROVISIONAL HYDRAULIC HAZARD
PMF EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
Strathfield
RAILWAY PDE
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
Study Area
Catchment Boundary
Provisional Hydraulic Hazard
Low Hazard
High Hazard
0
125
250
500
m
N
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure19c_Provisional_Hydraulic_Hazard_PMF.mxd

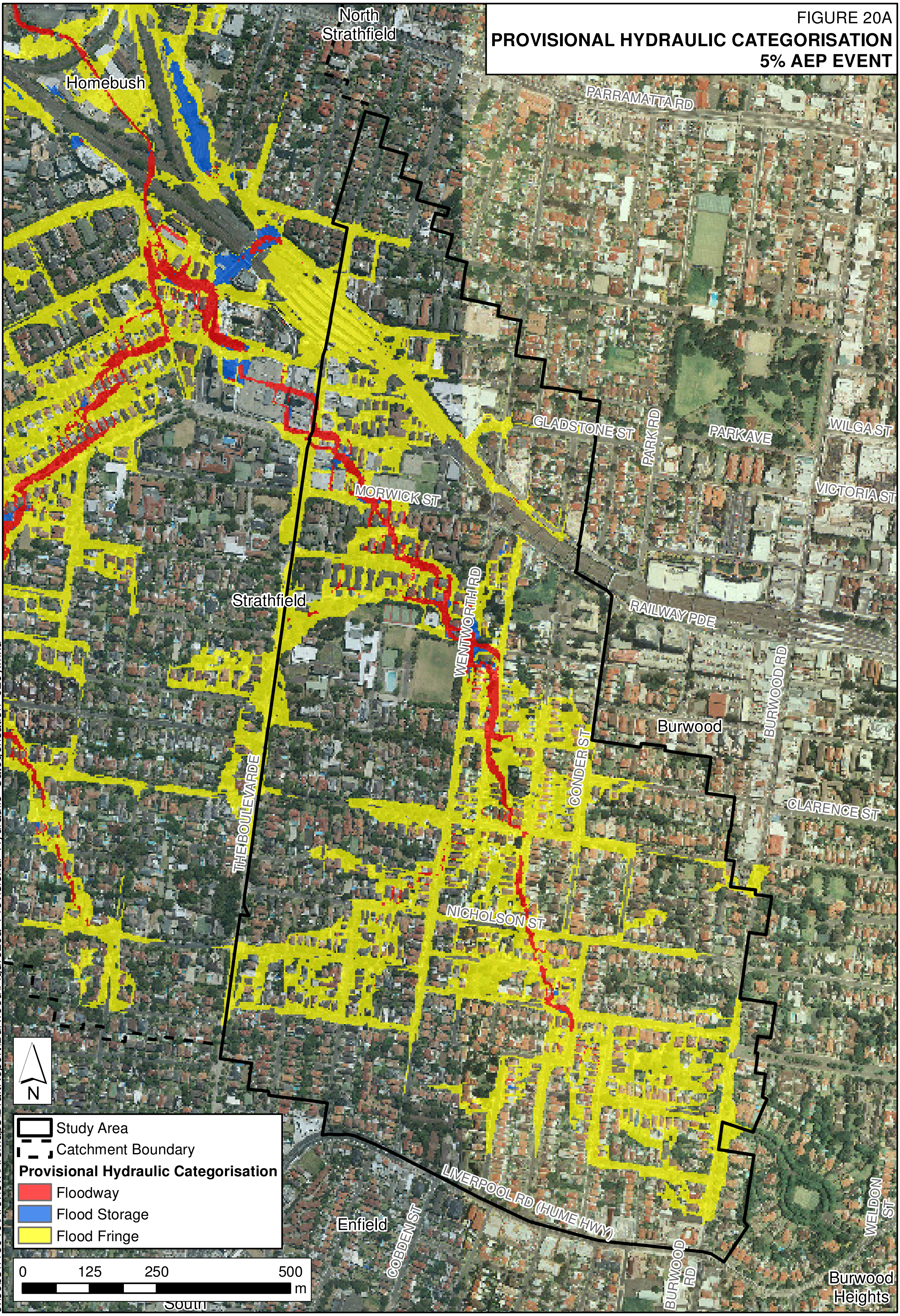


FIGURE 20A
PROVISIONAL HYDRAULIC CATEGORISATION
5% AEP EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
RAILWAY PDE
Strathfield
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
N
Study Area
Catchment Boundary
Provisional Hydraulic Categorisation
Floodway
Flood Storage
Flood Fringe
0
125
250
500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure20a_Provisional_Hydraulic_Categorisation_020v.mxd

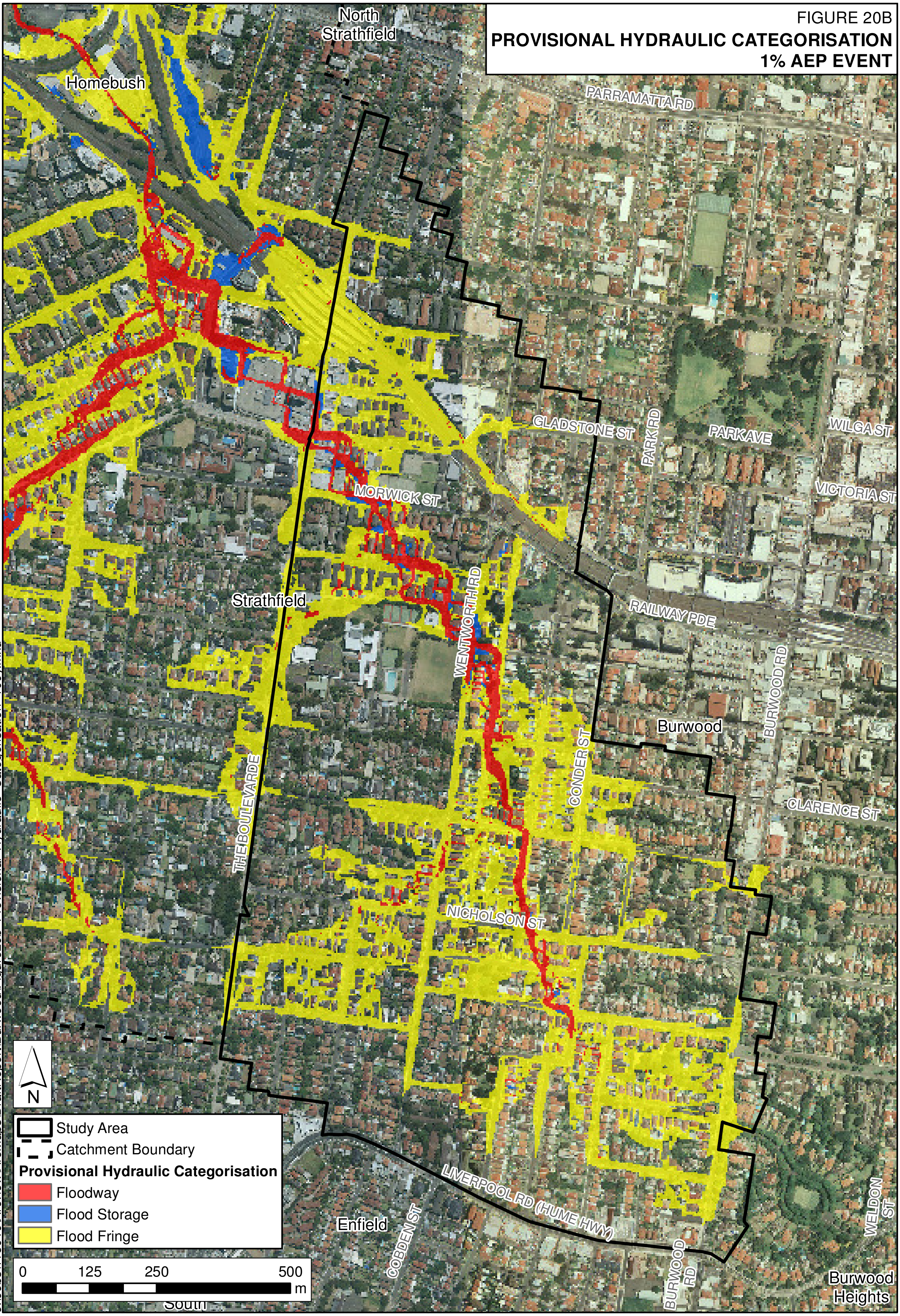
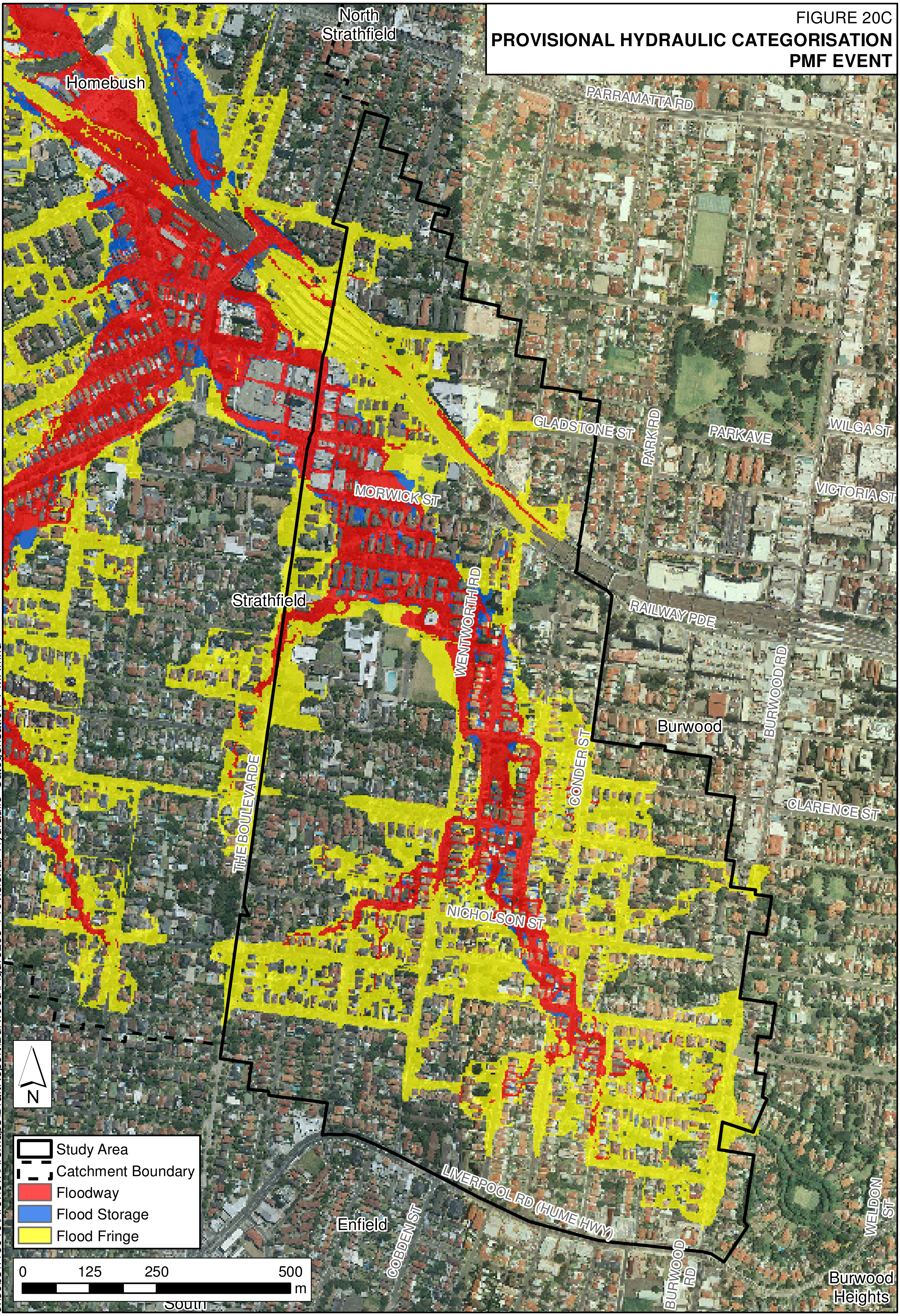


FIGURE 20B
PROVISIONAL HYDRAULIC CATEGORISATION
1% AEP EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
WENTWORTH RD
RAILWAY PDE
Strathfield
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
COBDEN ST
BURWOOD RD
WELDON ST
Burwood Heights
South
N
Study Area
Catchment Boundary
Provisional Hydraulic Categorisation
Floodway
Flood Storage
Flood Fringe
0
125
250
500
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure20b_Provisional_Hydraulic_Categorisation_100y.mxd

FIGURE 20C
PROVISIONAL HYDRAULIC CATEGORISATION
PMF EVENT

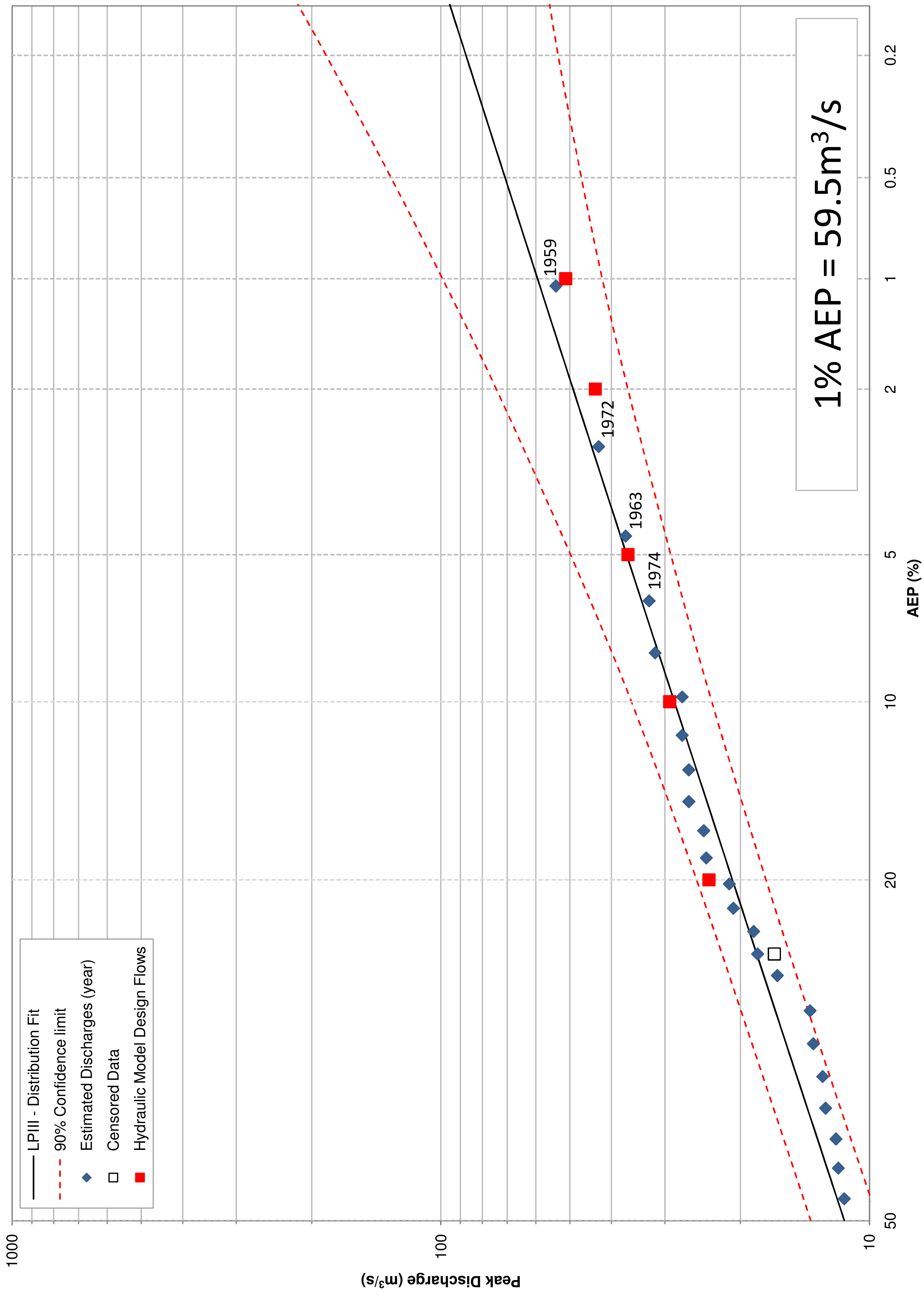


J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure20c_Provisional_Hydraulic_Categorisation_PMF.mxd

FIGURE 21

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

LP3 ANALYSIS - BAYESIAN



J:\Jobs\114021\FFA\Powells_FFA_LP3.xlsx

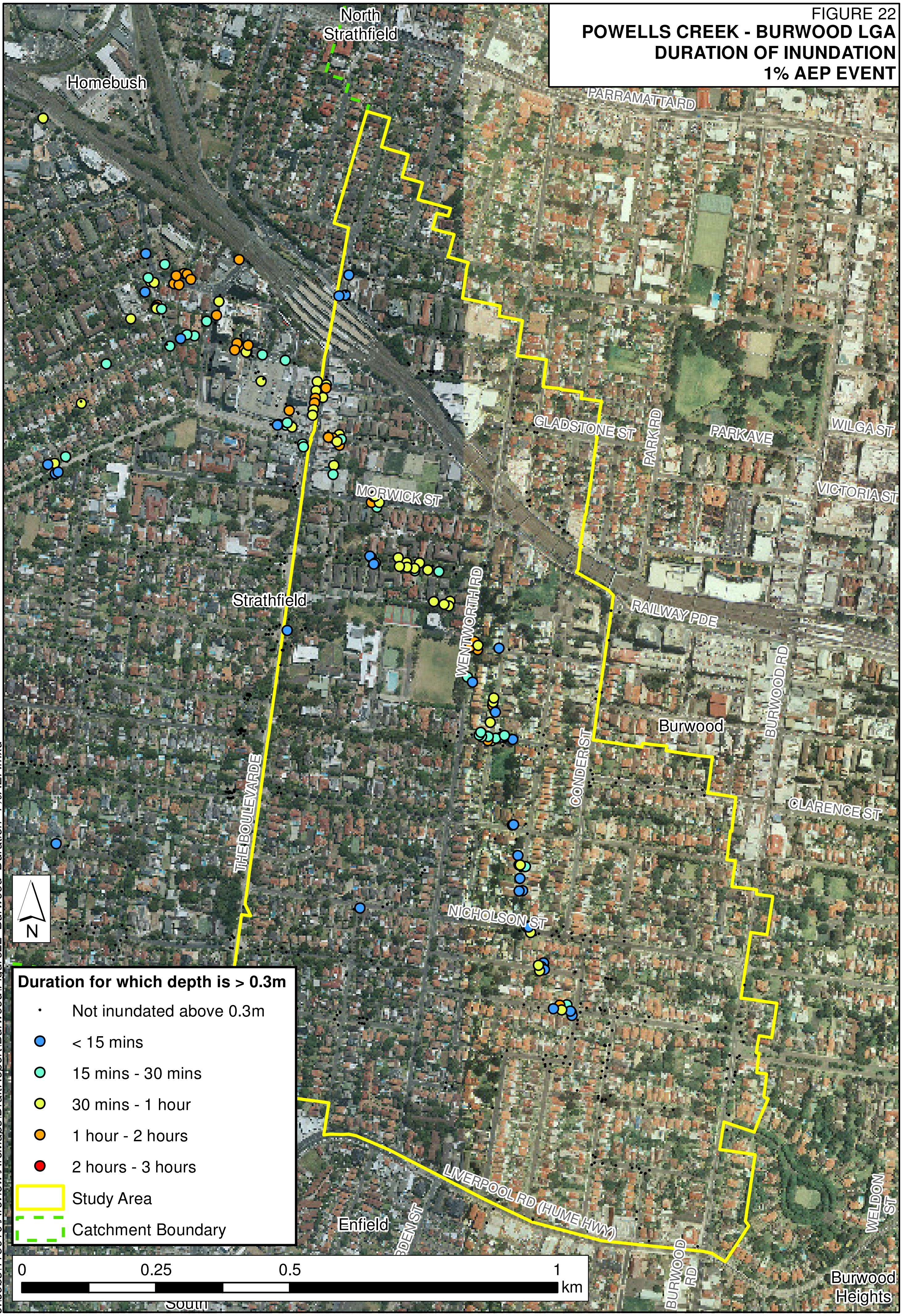
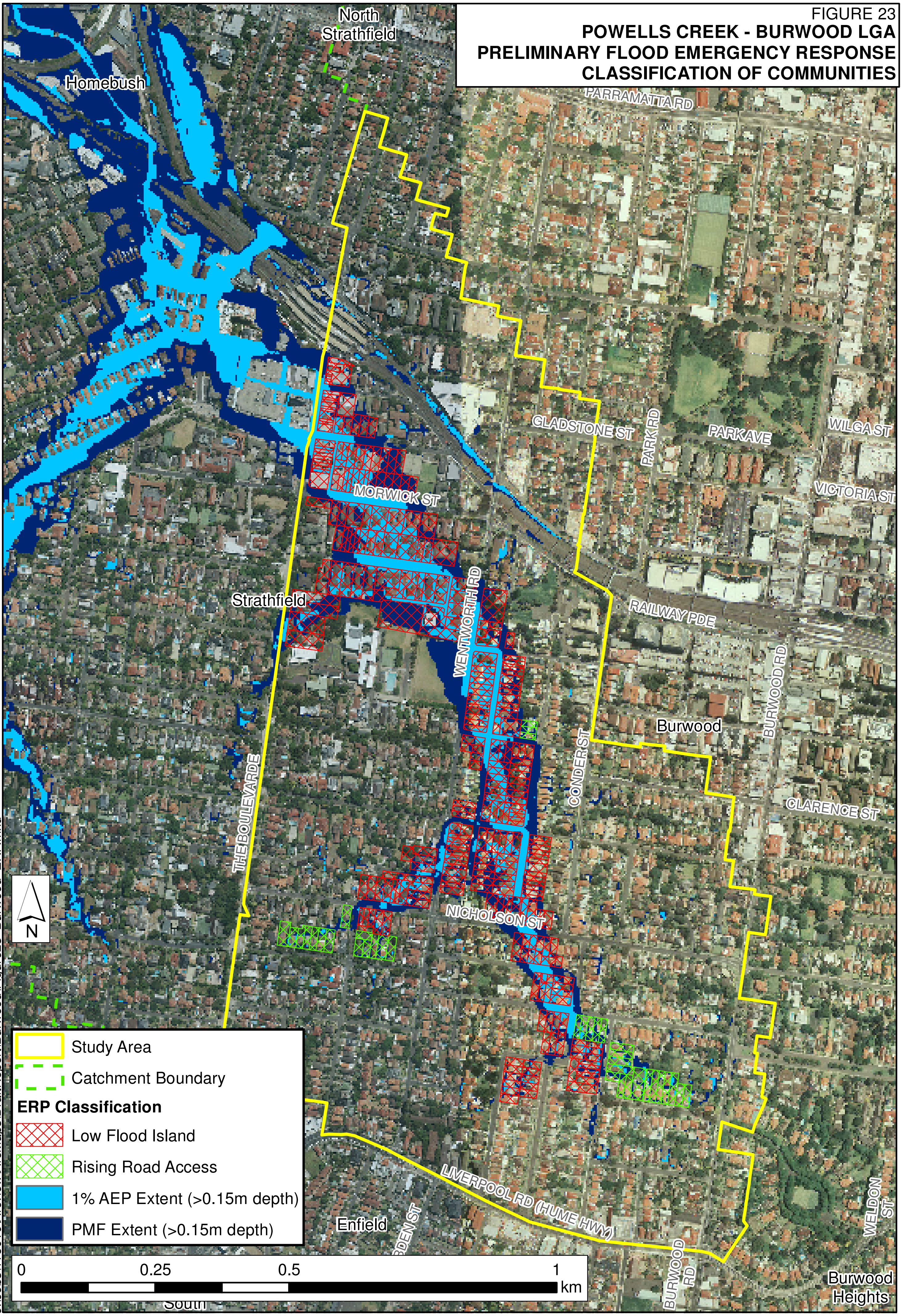


FIGURE 22
POWELLS CREEK - BURWOOD LGA
DURATION OF INUNDATION
1% AEP EVENT
North Strathfield
Homebush
PARRAMATTA RD
GLADSTONE ST
PARK RD
PARK AVE
WILGA ST
VICTORIA ST
MORWICK ST
RAILWAY PDE
Strathfield
WENTWORTH RD
BURWOOD RD
Burwood
CONDER ST
CLARENCE ST
THE BOULEVARDE
NICHOLSON ST
LIVERPOOL RD (HUME HWY)
Enfield
WELDON ST
Burwood Heights
South
N
Duration for which depth is > 0.3m
Not inundated above 0.3m
< 15 mins
15 mins - 30 mins
30 mins - 1 hour
1 hour - 2 hours
2 hours - 3 hours
Study Area
Catchment Boundary
0
0.25
0.5
1
km
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure22_Burwood_Duration_1%AEP.mxd

FIGURE 23
POWELLS CREEK - BURWOOD LGA
PRELIMINARY FLOOD EMERGENCY RESPONSE
CLASSIFICATION OF COMMUNITIES



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure23_Burwood_ERP.mxd

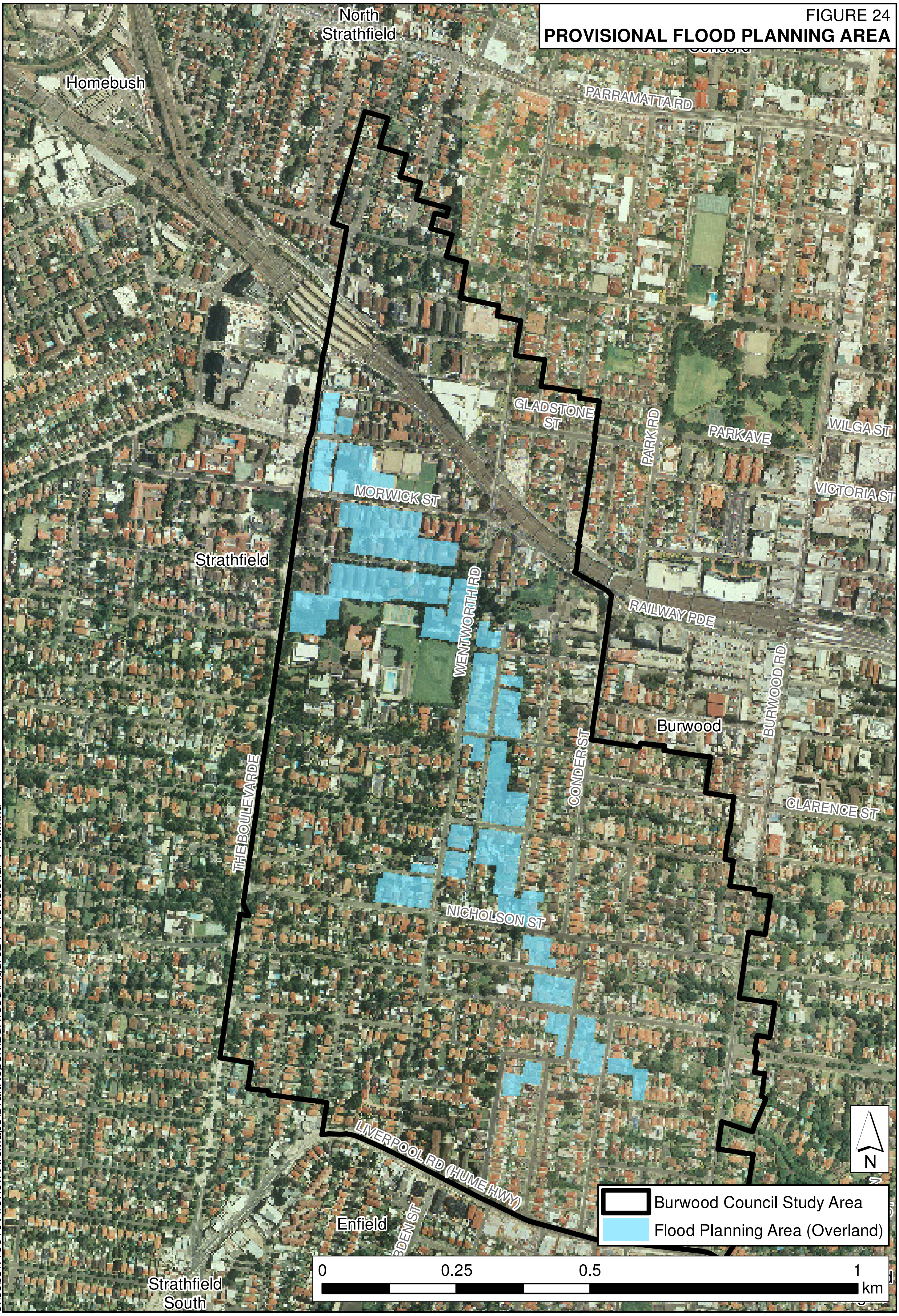


FIGURE 24
PROVISIONAL FLOOD PLANNING AREA

J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure24_Provisional_FPA.mxd

Appendix A

APPENDIX A: GLOSSARY of TERMS

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 diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.

flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the “flood liable land” concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL’s are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the “standard flood event” in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the

	Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: • the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or • water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or • major overland flow paths through developed areas outside of defined drainage reserves; and/or • the potential to affect a number of buildings along the major flow path.
mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State's rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock

	and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to "water level". Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

FIGURE B1

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #1

LP3 ANALYSIS - BAYESIAN

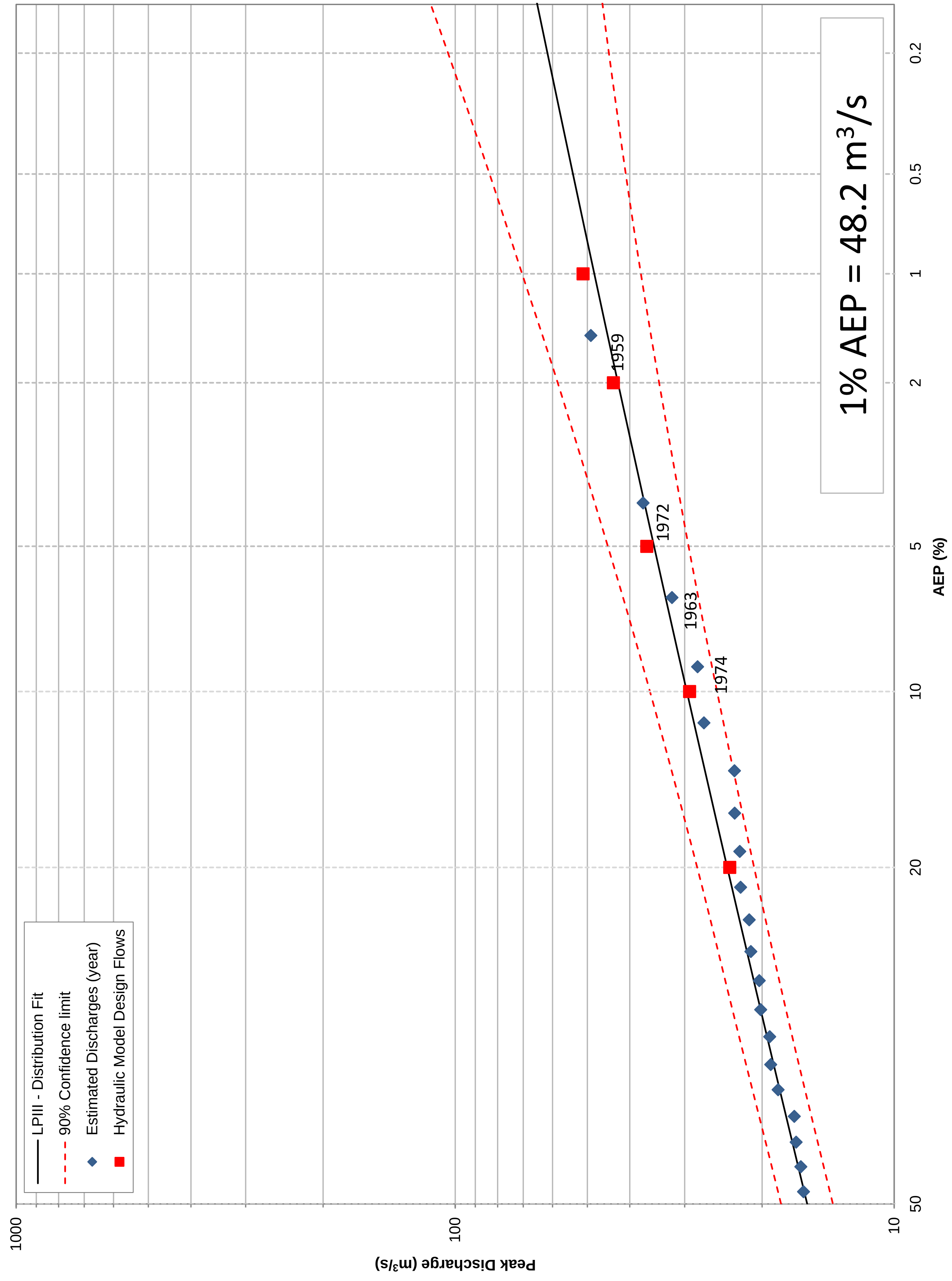


FIGURE B2

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #2

LP3 ANALYSIS - BAYESIAN

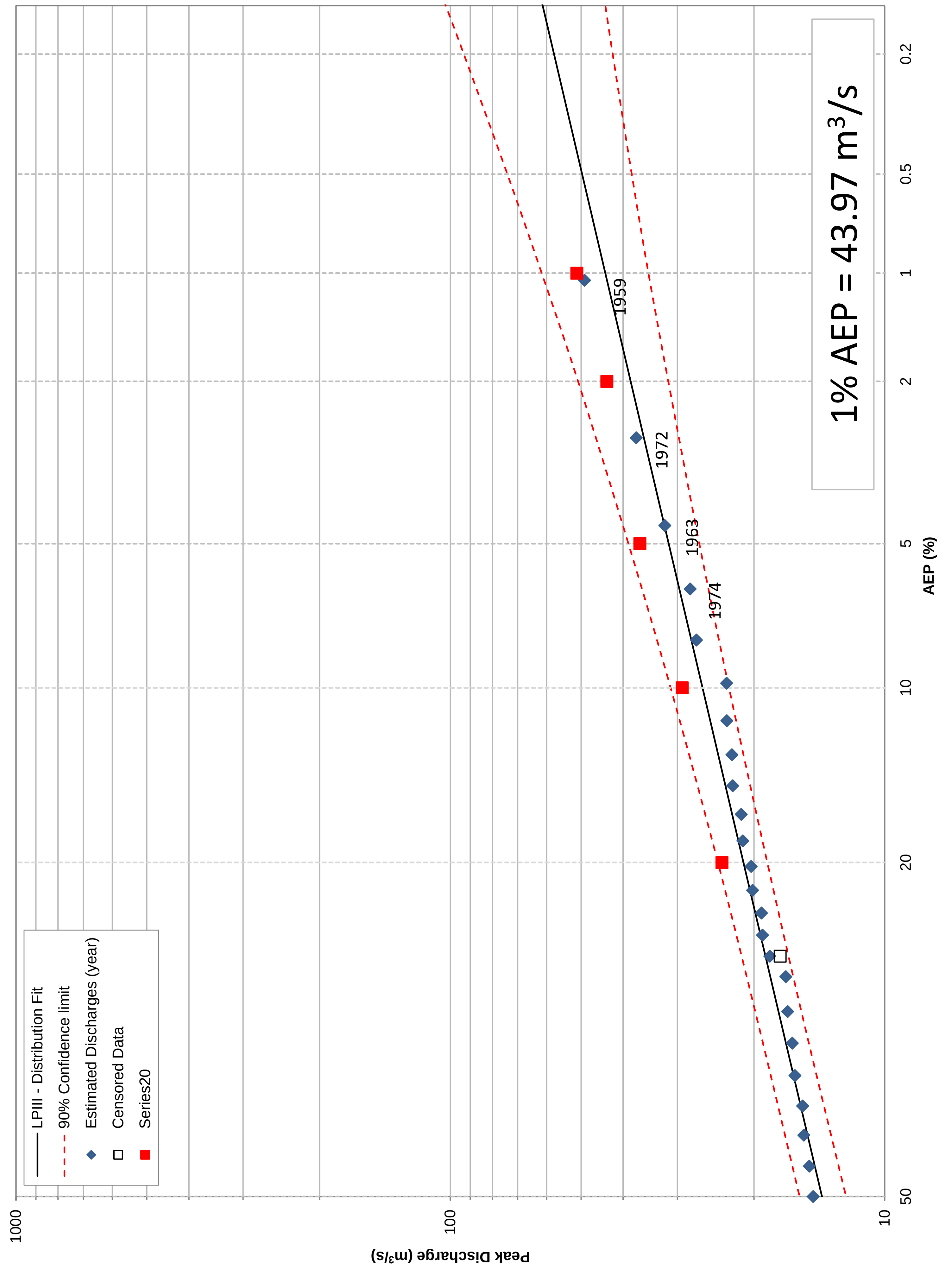


FIGURE B3

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #3

LP3 ANALYSIS - BAYESIAN

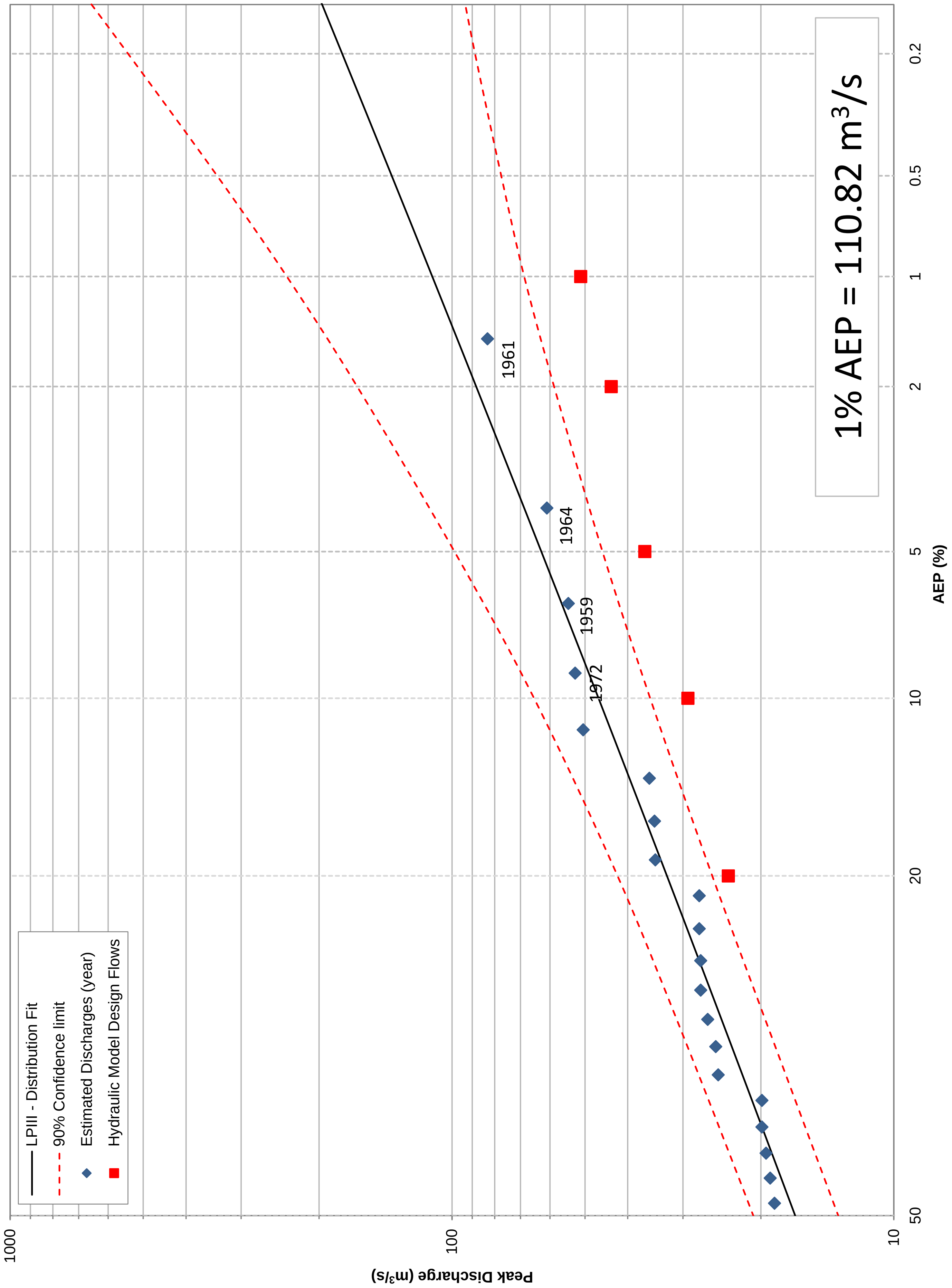


FIGURE B4

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #4

LP3 ANALYSIS - BAYESIAN

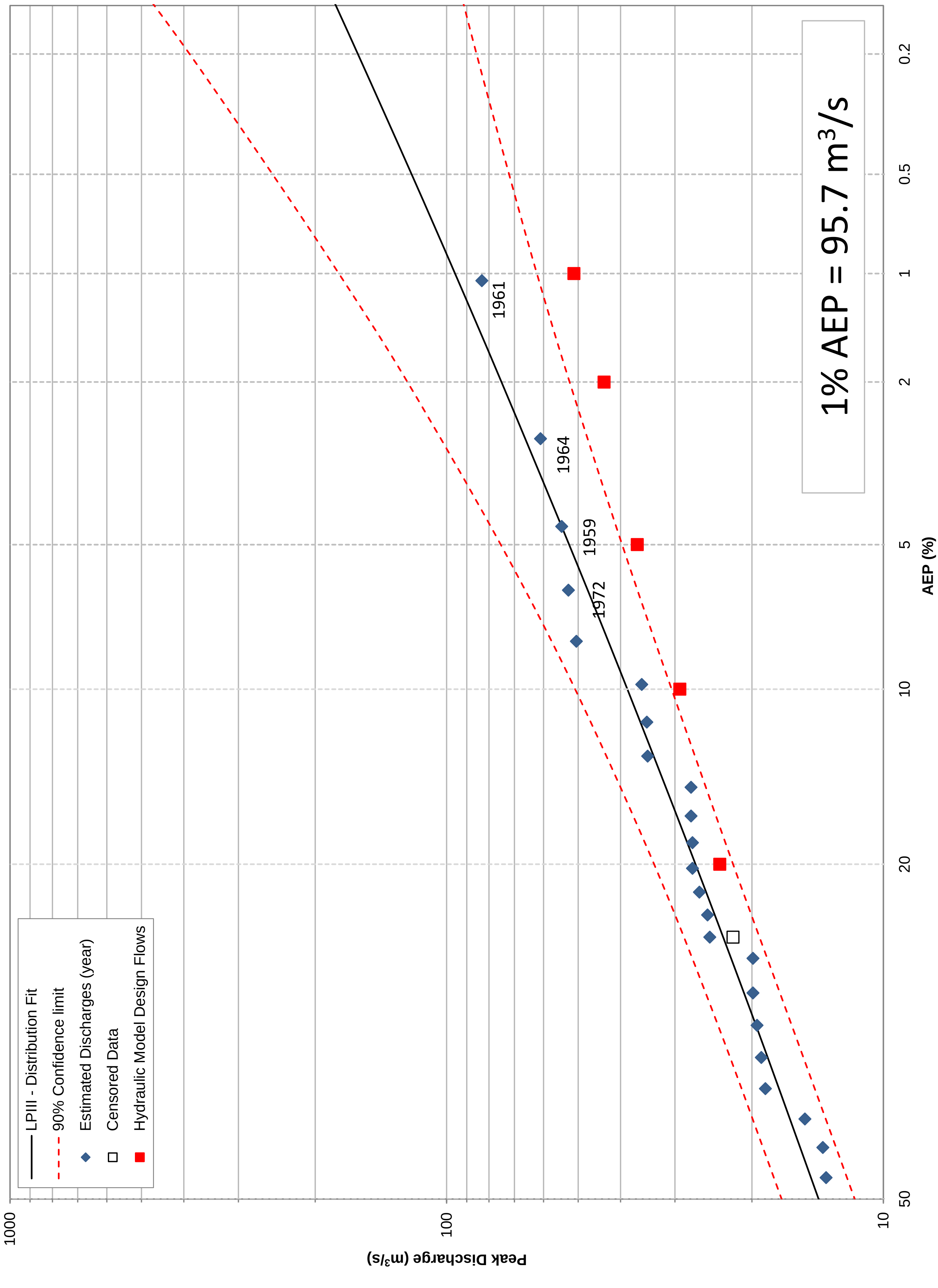


FIGURE B5

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #5

LP3 ANALYSIS - BAYESIAN

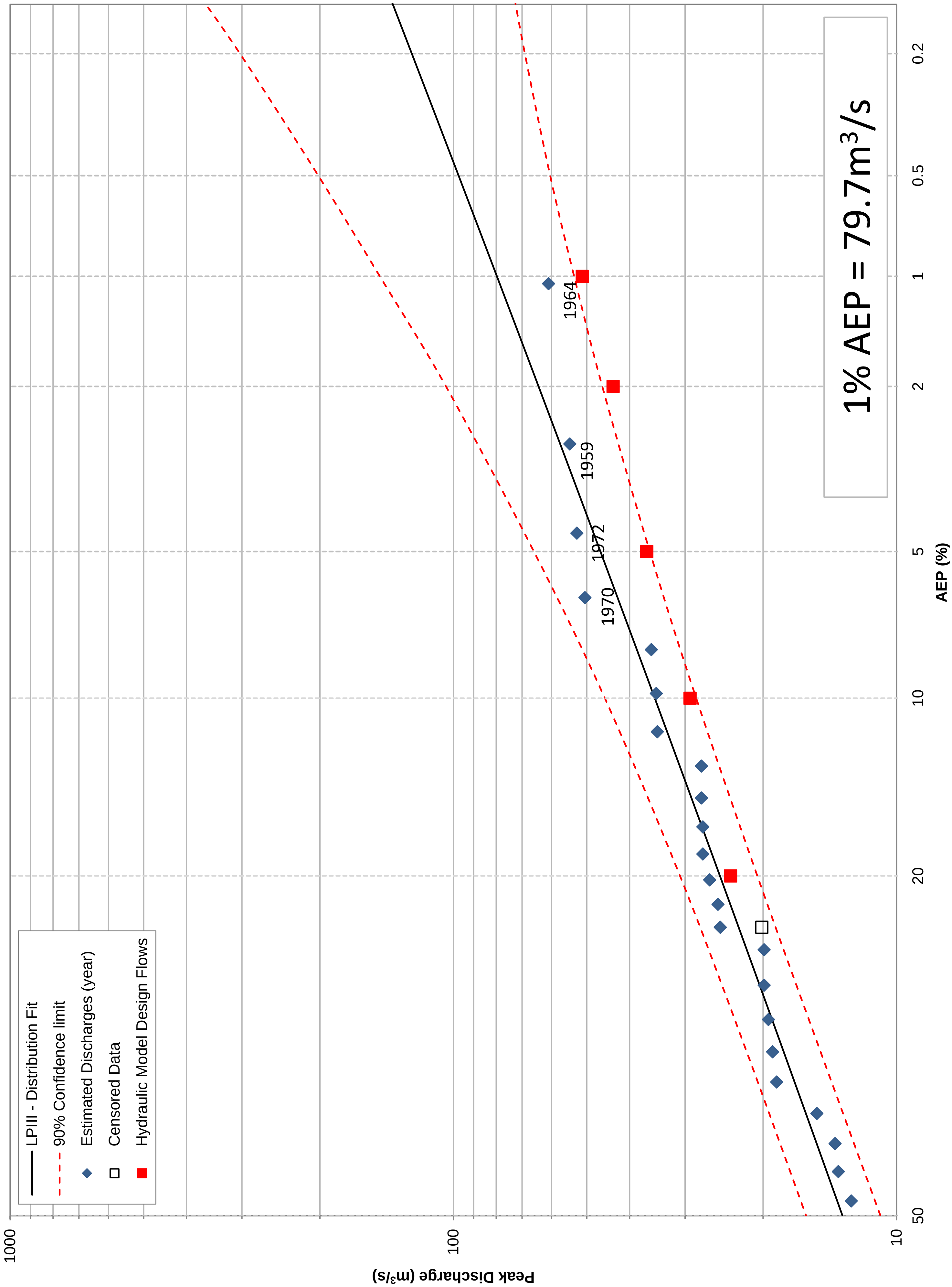


FIGURE B6

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #6

LP3 ANALYSIS - BAYESIAN

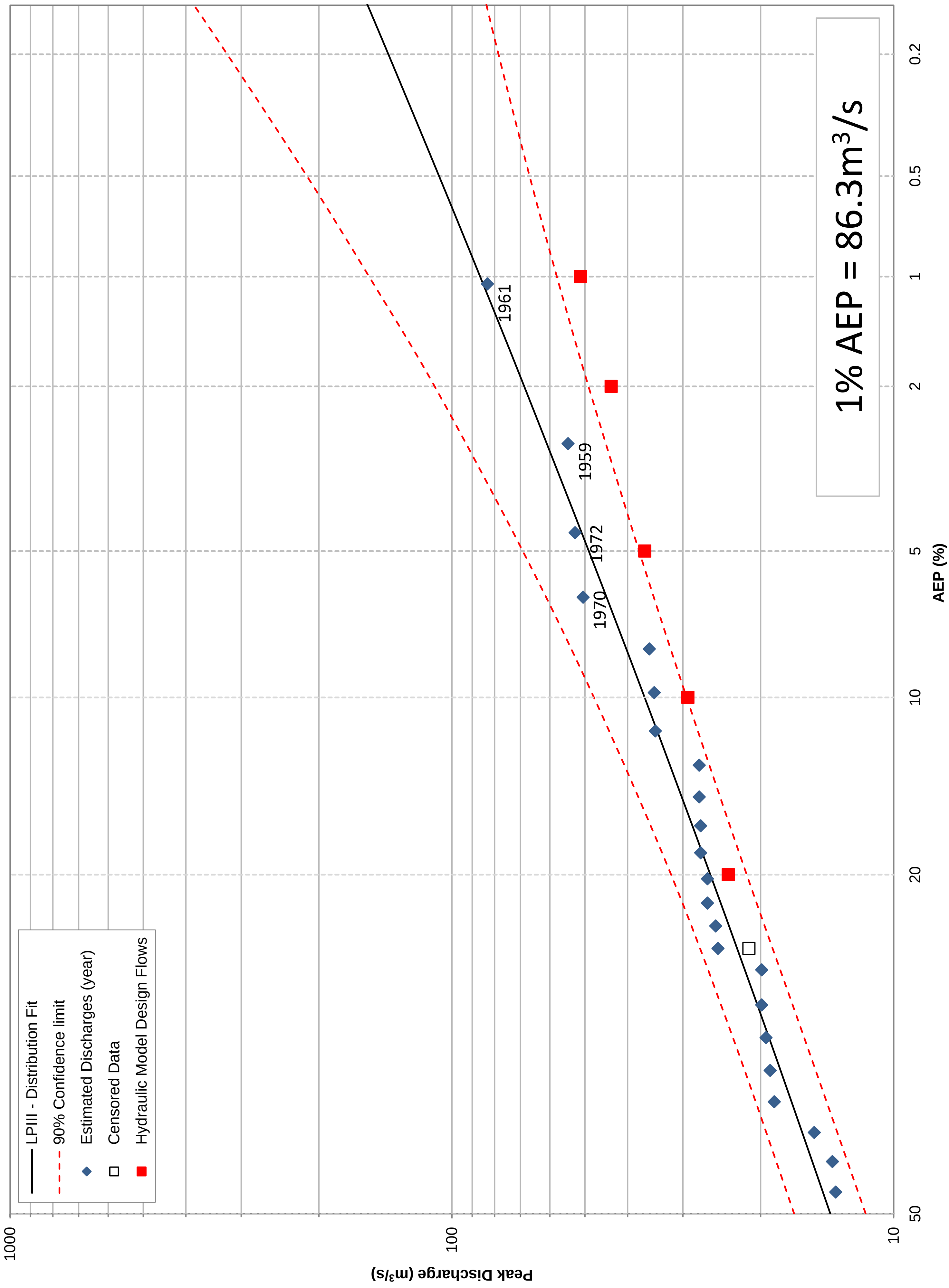


FIGURE B7

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #7

LP3 ANALYSIS - BAYESIAN

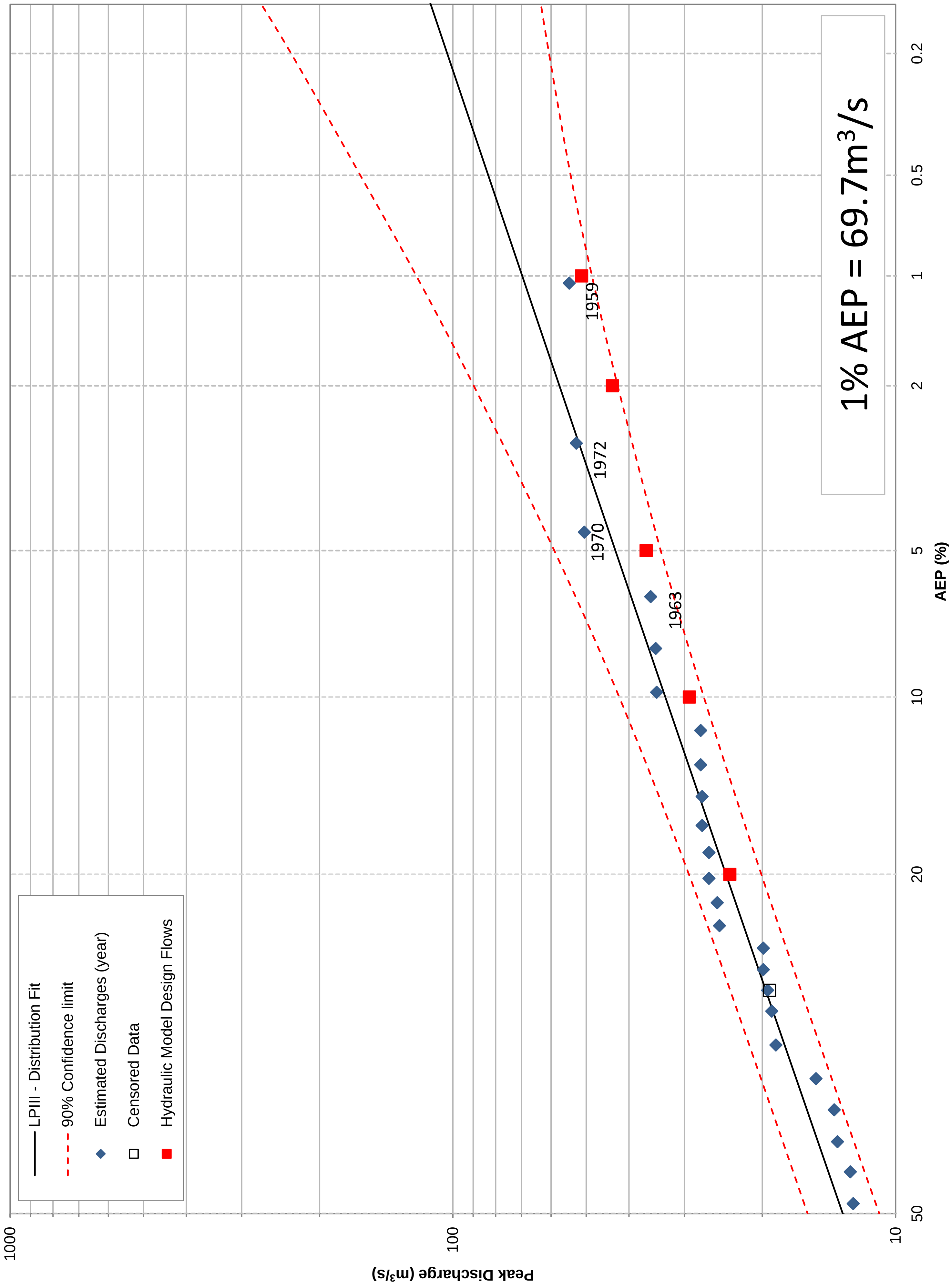
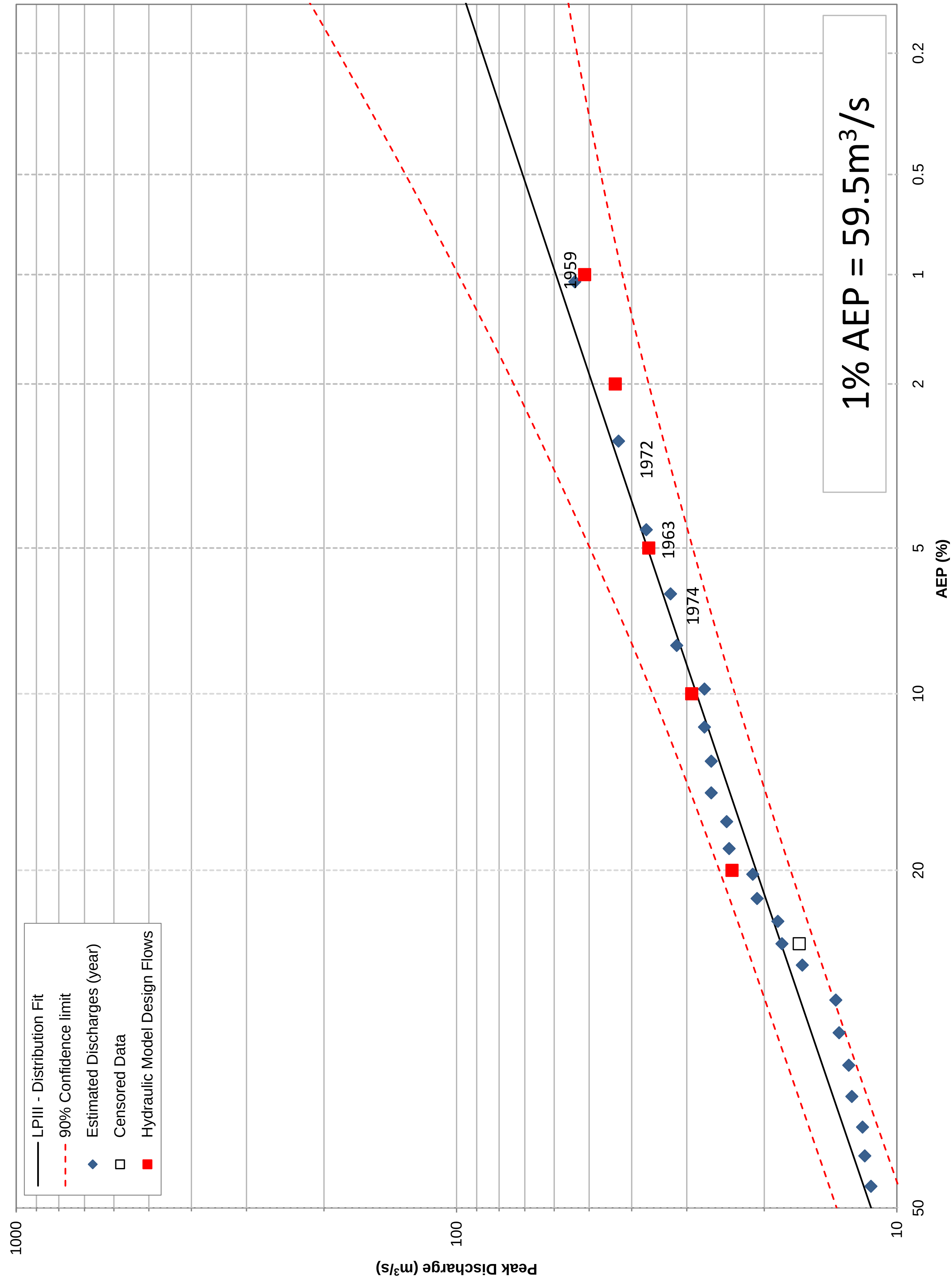


FIGURE B8

POWELLS CREEK - FLOOD FREQUENCY ANALYSIS

DATA SET #8

LP3 ANALYSIS - BAYESIAN



Appendix C

FIGURE C1

5% AEP EVENT PEAK FLOOD DEPTHS MINNA STREET TO NORWOOD STREET

FIGURE C2

1% AEP EVENT PEAK FLOOD DEPTHS MINNA STREET TO NORWOOD STREET

J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure_C02_Depths_100y_Minna.mxd

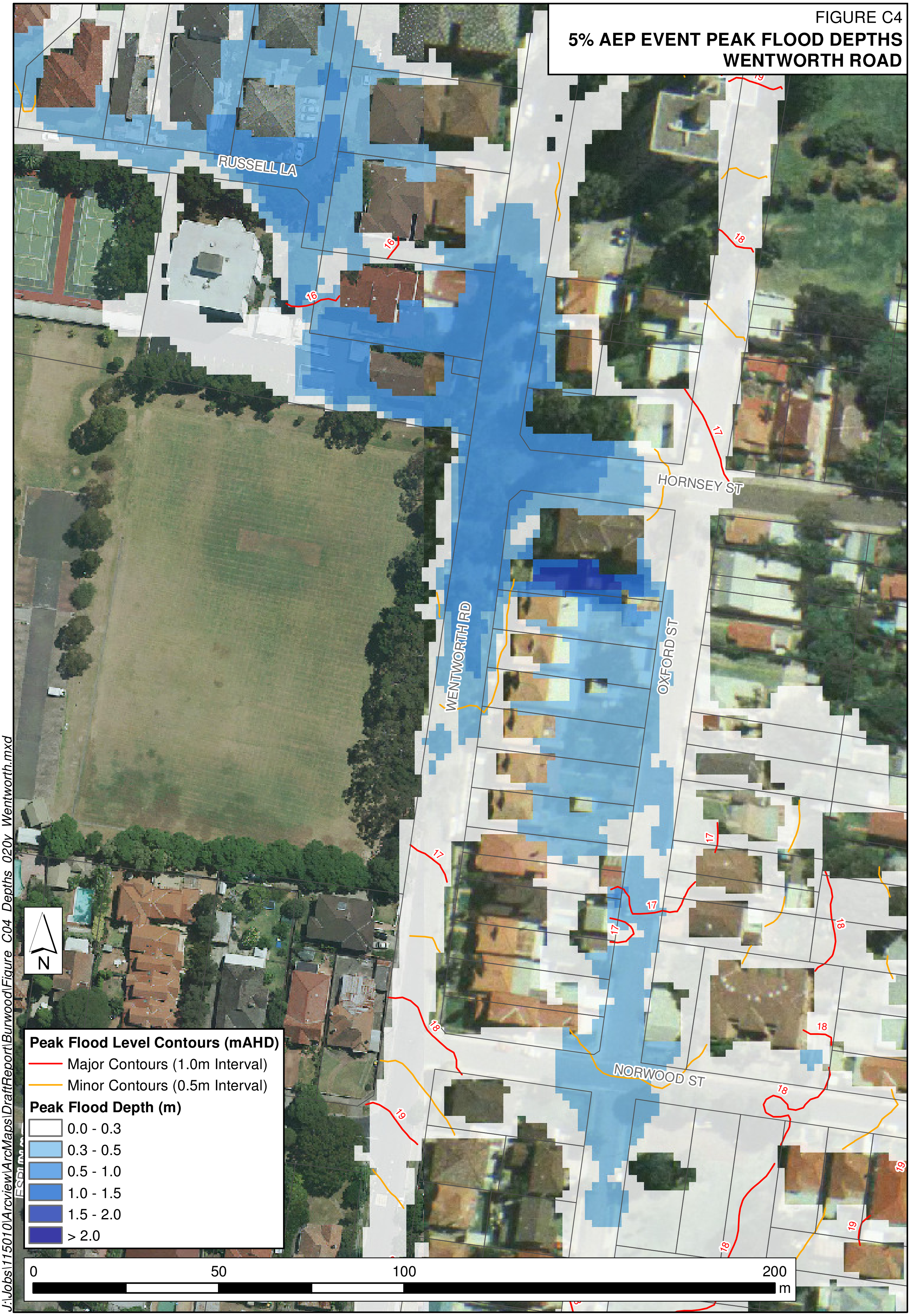


FIGURE C4
5% AEP EVENT PEAK FLOOD DEPTHS
WENTWORTH ROAD
RUSSELL LA
HORNSEY ST
WENTWORTH RD
OXFORD ST
NORWOOD ST
N
Peak Flood Level Contours (mAHD)
Major Contours (1.0m Interval)
Minor Contours (0.5m Interval)
Peak Flood Depth (m)
0.0 - 0.3
0.3 - 0.5
0.5 - 1.0
1.0 - 1.5
1.5 - 2.0
> 2.0
0
50
100
200
m
J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure C04 Depths 020y Wentworth.mxd

FIGURE C5
1% AEP EVENT PEAK FLOOD DEPTHS
WENTWORTH ROAD

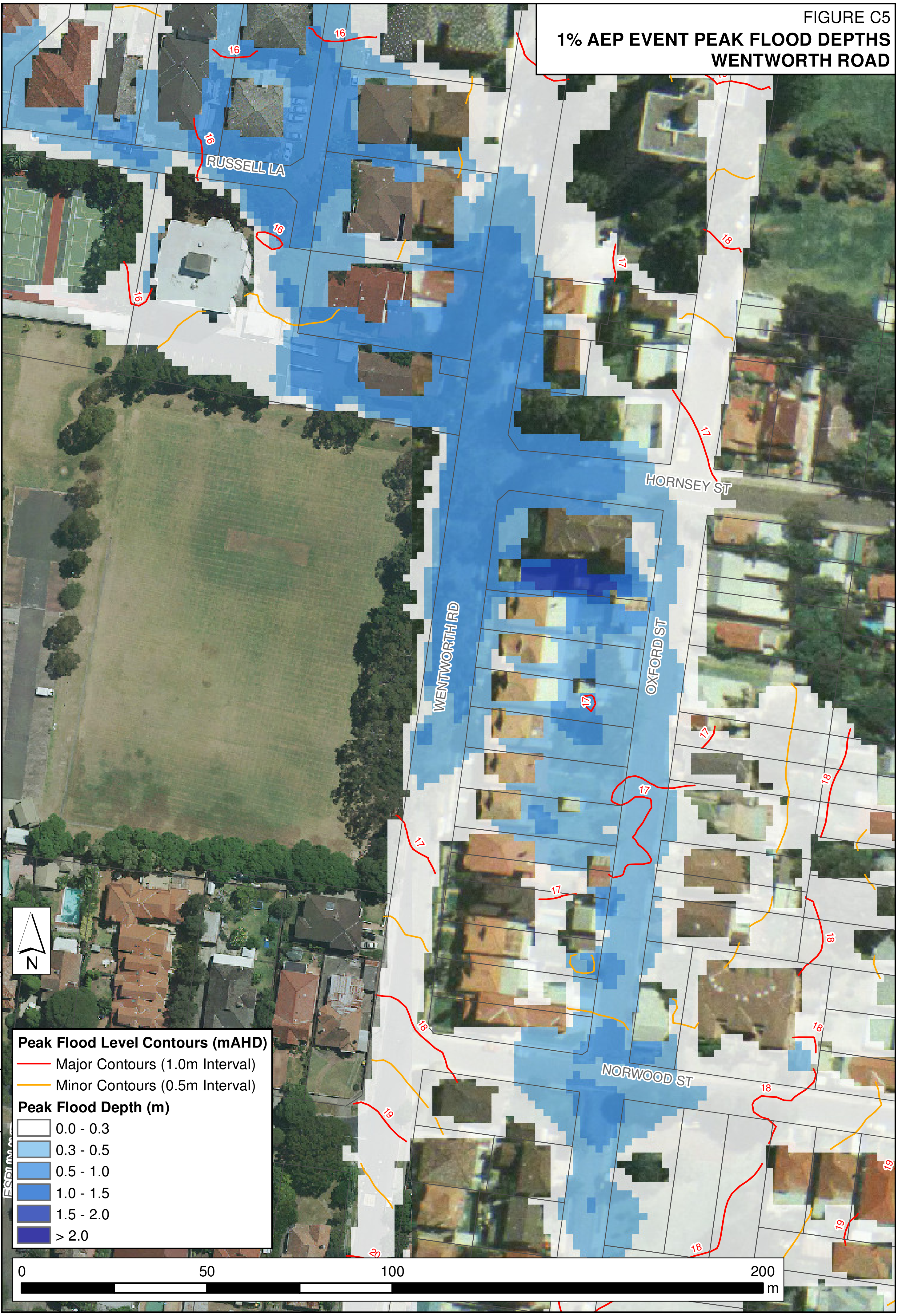


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FIGURE C7
5% AEP EVENT PEAK FLOOD DEPTHS RUSSELL STREET

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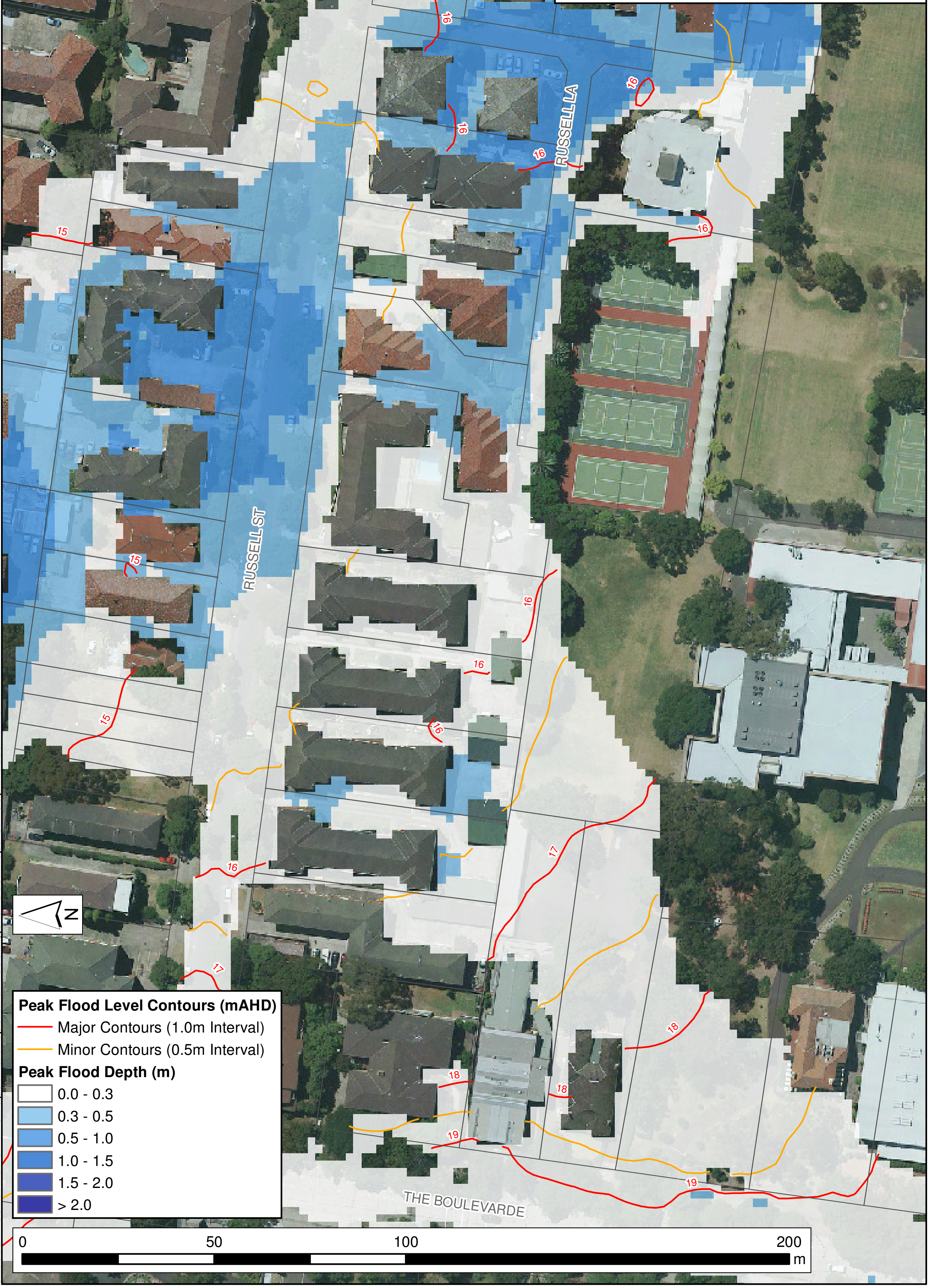
FIGURE C8
1% AEP EVENT PEAK FLOOD DEPTHS
RUSSELL STREET



J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure C08 Depths 100y Russell.mxd

FIGURE C10

5% AEP EVENT PEAK FLOOD DEPTHS MORWICK STREET TO LYONS STREET

J:\Jobs\115010\Arcview\ArcMaps\DraftReport\Burwood\Figure C10 Depths 020y Morwick.mxd

FIGURE C11

1% AEP EVENT PEAK FLOOD DEPTHS MORWICK STREET TO LYONS STREET

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