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MARRICKVILLE VALLEY FLOOD STUDY

FINAL REPORT

STORM_CONSULTING

APRIL 2013

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FOREWORD

Marrickville Council is committed to sustainable water management and applies an integrated and collaborative approach to catchment planning that includes flood risk management.

The NSW State Government's Flood Prone Land Policy provides a framework to ensure the sustainable use of floodplain environments. The primary objective of the Policy is to reduce the impact of flooding and flood liability on floodplain users and to reduce private and public losses from floods, utilising ecologically positive methods wherever possible. The Policy is specifically structured to provide solutions to existing flooding problems in rural and urban areas. In addition, the Policy provides a means of ensuring that any new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

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

Marrickville Council implements the Flood Prone Lands Policy as part of the subcatchment planning program. In addition to water cycle, social and organisational studies, the program includes studies and actions as set out in the NSW Floodplain Development Manual under the guidance of the NSW Government and with the assistance of other agencies including the Department of Planning, the State Emergency Services and Sydney Water.

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

1. Flood Study

- Determine the nature and extent of the flood problem for the full range of flood events up to the Probable Maximum Flood (PMF).

2. Floodplain Risk Management

- Evaluates management options for the floodplain in respect of both existing and proposed development taking into consideration social, ecological and environmental factors related to flood risk.

3. Floodplain Risk Management Plan

- Involves formal adoption by Council of a plan of management for the floodplain after community consultation and within Marrickville LGA, integrated with the relevant subcatchment plan.

4. Implementation of the Plan

- Involves construction of flood mitigation works to protect existing development, implementation of community awareness programs to heighten flood awareness, improved evacuation arrangements to minimise flood damages and the risk to life, and the introduction of development control policies at various levels within the planning framework to ensure new development is constructed in a manner compatible with the flood hazard.

The Marrickville Valley Flood Study constitutes the first stage of the risk management process for the areas including and adjacent to Marrickville (including parts of or all of the suburbs of Petersham, Stanmore, Enmore, Newtown, St Peters, Tempe, Marrickville and Dulwich Hill) and has been prepared for Marrickville Council by WMAwater under the guidance of NSW Government staff and Council's Floodplain Risk Management Committee (FPMC). It provides the basis for the future management of flood liable lands within the study area.

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). **Note:** in the present study the ARI notation has been used to describe the smallest event modelled i.e. the 2Y ARI event, instead of the AEP notation. The reason for this is that in the case of design rainfall, intensity values are obtained by partial series analysis and so whilst for large events simple relationships exist between ARI and AEP (a 1% AEP event is the same as the 100Y ARI event for example), for small events such simple relationships do not exist. So for example, a 50% AEP event has an ARI of ~ 1.3Y. The theoretical relationship between partial and annual series becomes approximately equal for events greater than 5Y ARI or 20% AEP (refer to Reference 8). Therefore, the AEP notation has not been used to describe the smallest event modelled, rather it is referred to as the 2Y ARI event.

Australian Height Datum (AHD) — A common national surface level datum approximately corresponding to mean sea level.

Average Annual Damage (AAD) — Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.

Average Recurrence Interval (ARI) — The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.

caravan and moveable home parks — Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.

catchment — The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.

consent authority — The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.

development — Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act).

infill development: refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development.

new development: refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.

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

disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas

	have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the Aflood liable land@ concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL=s are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the Astandard flood event@ in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood

risk is simply the existence of its flood exposure.

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 Awater level@. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

EXECUTIVE SUMMARY

As part of its subcatchment planning process Marrickville Council engaged Storm Consulting, in conjunction with WMAwater, to carry out a flood study for the Marrickville Valley under the NSW State Government's Floodplain Risk Management Program.

The goals of the study are to:

- Define design flood behaviour particularly for the 1% AEP event; and
- Produce a modelling system that will be suitable for analysing floodplain management measures in subsequent stages of the Floodplain Risk Management Program.

To facilitate the flood study a variety of data has been collected from Council, the Bureau of Meteorology (BOM), Sydney Water and others. Additionally a community flood questionnaire has been distributed. The major finding is that besides the Marrickville Industrial Area (MIA) few residents report suffering from regular flooding/drainage issues (although there also seems to have been a lack of severe rainfall events in past decades). With little information/observations from the community to undertake a rigorous calibration/validation of the modelling process, verification was carried out instead. This has been achieved by the following means:

- Comparison of minor flood results to approximate expectations based on
    - Council's knowledge of drainage hot spots;
    - Community questionnaire results;
    - Comparison to estimates from previous studies.

Overall the verification work tends to confirm that the modelling process is producing reasonable results. However when future flood events occur, flood data should be collected and a more rigorous model calibration undertaken.

Design flood modelling has identified a number of areas which in a significant storm event are likely to experience flooding and in some cases widespread inundation. These include the following areas:

- Marrickville Industrial Area;
- Malakoff Street and generally the area downstream of Marrickville Oval and on the southern side of Sydenham Road (Livingstone Road);
- Marrickville Railway Station;
- Sydenham Railway Station.

Modelling results showed that the Eastern Channel (EC) has sufficient capacity to convey flows of up to the 1% AEP event magnitude. However the Central Channel, with practically negligible slope has insufficient capacity during all design events. Additionally, tailwater conditions in the Cooks River have an impact on its capacity. Whilst the Malakoff Tunnel (referred to as the Western Channel Amplification by Sydney Water) shows to be at almost full capacity during the

1% AEP event its inlet pits do not have capacity to convey all overland flows into the tunnel. The Western Channel only shows capacity to transfer flows in the 2Y ARI event along almost in its entire reach without overtopping.

The Marrickville Industrial Area is adversely affected during all design flood events. A critical factor for this is its relative low level and flat topography. In the 1% AEP event more than 50% of the streets are categorised as High Hazard areas.

Other locations such as Sydenham Road, Marrickville and Sydenham Railway Stations and Carrington Road are also categorised as High Hazard areas in the 1% AEP event.

During all modelled events, Sydney Water pumping stations DPS2 and SPS271 are operating 100% of the time and are unable to avoid surcharging of the pits. Pumping station DPS1 is in the Sydenham storage pit and operates at different rates depending upon the water level in the pit. In most design events it will reach full capacity within 30 minutes after the start of the rain.

Mackey Park does not become inundated in the 2Y ARI event. However, it becomes inundated in greater events. The Mackey Park levee, which protects the park from inundation from the Cooks River, is overtopped in the 1% AEP event.

A range of model parameters were tested to assess model sensitivity. The results showed a robust hydraulic model with peak level differences in the order of ± 0.1 m. Climate change scenarios as provided in the current NSW Government guidelines (August 2010) have been investigated for the 1% AEP event. The results indicate that a 0.4m sea level rise will increase the 1% AEP flood levels by a maximum of 0.1m and a 0.9m sea level rise by a maximum of 0.2m. These increases are confined to the lower parts of the catchment. The increase in the 1% AEP design rainfalls result in a more general increase in flood levels across the entire catchment. A 10% rainfall increase in design rainfalls results in approximately a 0.1m maximum increase in peak levels, a 20% rainfall increase a 0.2m maximum increase in peak levels and a 30% rainfall increase a 0.3m maximum increase in peak levels. The combinations of a rainfall increase and a sea level rise increase indicated the same as the addition of the individual rainfall/sea level rise scenario increases.

1. INTRODUCTION

1.1. Background

Marrickville Council has undertaken to address flood liability within the Marrickville Valley study area by engaging in the NSW Government's floodplain risk management process and this flood study is the first stage in the process. This Flood Study has been undertaken by WMAwater acting as a subcontractor to Storm Consulting.

Council now implements flood risk planning as part of its subcatchment planning program which is based on a collaborative multi stake holder approach at the local level.

The location of the Marrickville Valley is shown on Figure 1. As can be seen the study area includes parts or all of the suburbs of Petersham, Stanmore, Enmore, Newtown, St Peters, Tempe, Marrickville and Dulwich Hill. Bounding the study area to the west lies New Canterbury Road and to the east lies Alexandra Canal within the City of Sydney Local Government Area (LGA). The entire catchment area included in the study is approximately 8 km^2. The Marrickville Valley catchment drains southwards into the Cooks River, upstream of the main Illawarra/East Hills railway line crossing, via four drainage outfalls (Figure 2).

A number of locations within the study area are flood liable. This flood liability mainly relates to the nature of the topography within the study area as well as the current level of service provided by drainage assets. Generally low lying areas will accumulate runoff as trunk drainage elements (pit and pipe system as well as open channels) become inadequate to convey flow south to the Cooks River. Despite this likely high degree of flood liability it is noted that whilst a floor level survey has not been carried out it appears, based on site inspection, that most homes have some freeboard relative to ground elevation (0.3 to 0.5 m). This likely provides some degree of flood protection for all but the worst affected homes. Possibly for this reason there are few records of above floor inundation and consequently few enquiries to Council regarding the impacts of flooding.

Most of the larger drainage assets in the study area (open channels and larger sub-surface elements such as the trunk drainage system) are owned and maintained by Sydney Water (Reference 1) whilst pits and pipes are Marrickville Council's responsibility. References 1, 2 and 3 have investigated flooding in the catchment in the past.

Overall the study area is highly developed with a high proportion of impervious area. Land usage within the study area comprises of a mix of residential, light industrial and commercial developments together with relatively limited areas of open space.

The mean annual rainfall depth is 1100 mm per annum and the wettest months are January through to June (Reference 2). It is of note that in the past few decades the study area does not seem to have experienced significant rainfall events and as such the level of flood preparedness/knowledge within the valley is very low.

1.2. Objectives

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

Primarily, the study was developed in order to meet the objective of defining the flood behaviour (1%, 10%, 20% AEP and 2Y ARI (see explanation note in Glossary) events and the Probable Maximum Flood) in the Marrickville Valley catchment and:

- Define flood behavior in terms of flood levels, depths, velocities, flows and flood extents within the study area;
- Prepare provisional flood hazard and flood extent mapping (for all events modelled);
- To consider the potential effects of a climate change induced increase in design rainfall intensities and sea level rise in accordance with the current NSW Government climate change guidelines (August 2010).

1.3. History

The valley floor was originally a freshwater wetland known as Gumbramorra Swamp that drained in a southerly direction towards what is now known as the Cooks River. After European settlement the area was initially farmland but became developed as a working class residential area in the 1880s. There are records of major flooding in May 1889. Subsequently, the swamp was drained with the construction of the Eastern, Central and Western Channels and two associated pumping stations. In the 1940's a 100ML detention basin with associated pumping station was constructed in the Marrickville Industrial Area and is known as the Sydenham Storage Pit. There are no official or comprehensive records of water levels in the Sydenham Storage Pit but during extensive heavy rain in 1986 the basin was close to overtopping.

1.4. Study Area

1.4.1. Subcatchments

The Marrickville Valley study area (Figure 2) comprises a 7.9 km^2 catchment which ultimately drains via four outfalls into the Cooks River to the south, namely the Eastern, Central and Western Channels as well as the Malakoff Street Tunnel (referred to as the Western Channel Amplification by Sydney Water). The study area is heavily developed and consists primarily of high density residential and light industrial developments. The study area is bounded to the north, east, south and west (respectively) by New Canterbury Road, the Princes Highway, the Cooks River and Stanmore Road.

The study area is divided into nine subcatchments, each of which drains to one of the four outfalls (shown on Figure 2) and are described below:

- Eastern Channel North (**ECN**);
- Eastern Channel East (**ECE**) – this area has been studied in the 2010 Golder

Associates Flood Study (Reference 3). As such whilst flows out of the ECE have been incorporated into the current study no results are provided for this area in this present study and design flood results should be obtained from Reference 3;

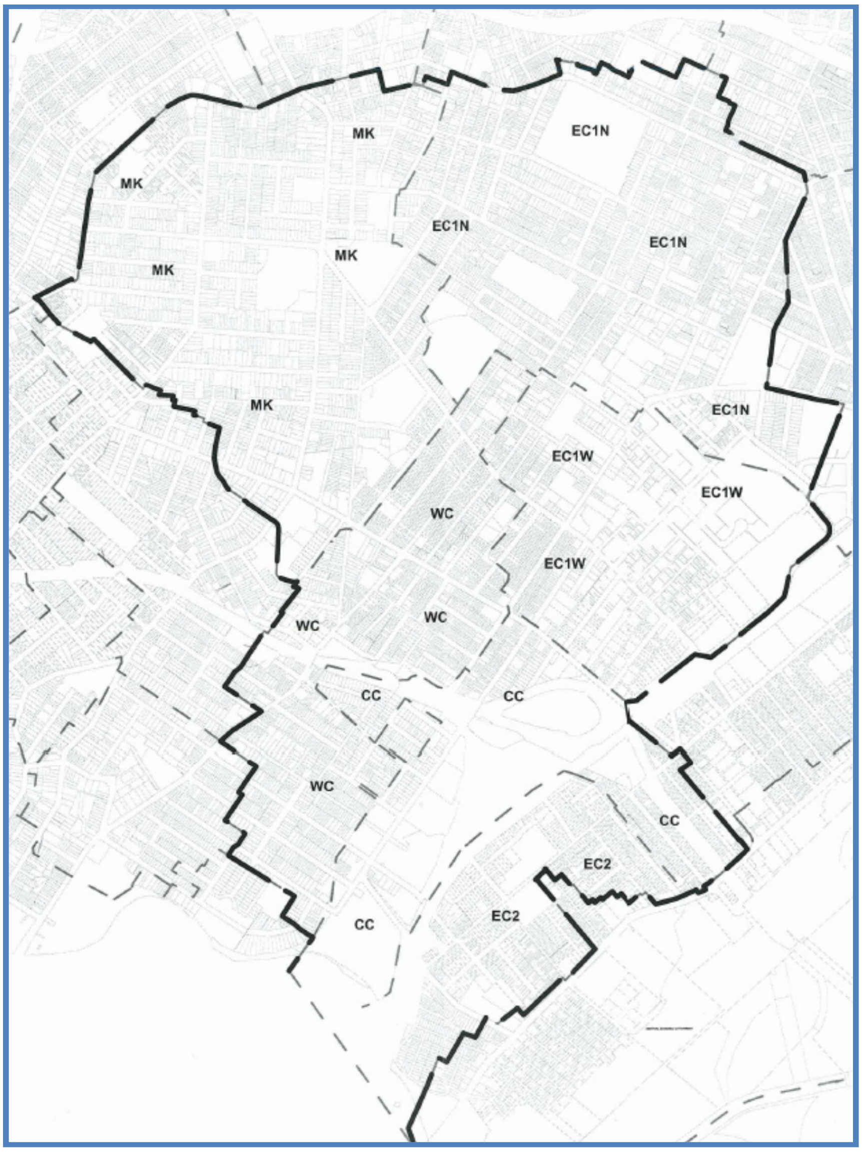
- Eastern Channel West (**ECW**) – this area corresponds to the Marrickville Industrial Area (MIA) and drains to the Sydenham storage pit from where it is later pumped into the Eastern Channel;
- Eastern Channel South (**ECS**);
- Eastern Channel 2 (**EC2**);
- Western Channel (**WC**);
- Central Channel (**CC**);
- Malakoff Street (**MK**) – This is the upstream area draining into the Malakoff Street Tunnel. In larger flood events which exceed the capacity of either the diversion into the Malakoff Tunnel or the Malakoff Tunnel itself, flow from this subcatchment will move into ECW and the MIA. It should be noted that Sydney Water refers to the "*Malakoff Tunnel*" as the "*Western Channel Amplification*" however the former terminology has been adopted in this report to make it consistent with other Marrickville Council documentation;
- Malakoff Tunnel (**MT**).

The catchment areas of the nine subcatchments are provided in Table 1.

Table 1: Subcatchments

Subcatchment Name	**Area (ha)**
Eastern Channel West (ECW)	75
Malakoff Street (MK)	141
Eastern Channel North (ECN)	136
Western Channel (WC)	81
Malakoff Tunnel (MT)	59
Eastern Channel 2 (EC2)	52
Central Channel (CC)	71
Eastern Channel South (ECS)	43
Eastern Channel East (ECE)	131
TOTAL	**790**

Subcatchments defined in this study are shown on the image below sourced from Reference 2, except for MK, ECS and ECE. The image below indicates a small "square" of catchment within the CC area, this represents an area where the piped drainage system drains to the WC and not to the CC.



Four major trunk drainage lines discharge to the Cooks River (Figure 2), namely:

- Eastern Channel (EC);

  The Eastern Channel drains approximately 345 hectares or 44% of the Marrickville Valley. It also receives flow from the low lying areas and the Central Channel. The Eastern Channel cross section dimensions vary along its path. In the present flood study the Eastern Channel was modelled (as a 1D structure) downstream of Murray Street since the upstream section was modelled in Reference 3. Key features include:

  - Open concrete channel of varying dimensions and grade with an average grade of 0.17%;
  - Drains a significant portion of the MIA catchment;
  - Drains subcatchments EC2, EC1W and EC1N to the Cooks River;
  - Runoff from EC1W and some of EC1N conveyed by Sydney Water trunk drains to Sydenham storage pit basin located north of Sydenham railway station. Water is then pumped from the Sydenham pit to the Eastern Channel. EC2 enters the Eastern Channel downstream of the Sydenham pit;

- Central Channel (CC);

Two pumping stations are located within this subcatchment namely, DPS2 and SPS271. The channel alternates between an open concrete channel (refer to Figure 2) and closed box culvert and extends downstream of Marrickville Road to the Cooks River. The channel slope is negligible, which means that a significant water level difference is required between the upstream and downstream sections to ensure the maximum channel capacity.

- Western Channel (WC);

  The Western Channel alternates between an open concrete channel (Figure 2) and a concrete box culvert. In the upper areas it transfers flows to the Malakoff Tunnel. Downstream it crosses Marrickville Street and eventually discharges into the Cooks River as an open channel. At its junction with the Cooks River, the channel has an invert level of 0.8 mAHD. The average grade of the channel is approximately 0.07%.

- Malakoff Street Tunnel;
  - Closed concrete box culvert with average grade of 0.22%;
  - Extends all the way to Cooks River and picks up flow from MK catchment [note it appears Reference 2 missed out on some of the Malakoff Tunnel catchment although given the area of focus of the study was the MIA this is understandable].

The drainage assets owned and maintained by Council include pipes of dimensions 150 mm to 1200 mm in diameter.

The EC was constructed in 1900, the WC in 1901 and the rest of the system is relatively old, although build dates vary markedly. A lot of work has been undertaken on the system over time and as augmentations have been made this has necessitated changes. For example, CC originally conveyed runoff from subcatchment EC1W but this was blocked after the construction of EC and the WC once conveyed runoff from the Malakoff Street subcatchment but this was blocked after the tunnel was built in the 1970's.

1.4.2. Drainage System

The study area is serviced in the main by a major/minor drainage system. Property drainage is directed to the kerb/gutter system where it is then able to enter the piped system which then discharges to the trunk drainage system. Flows in excess of the pipe network capacity will either be stored in the road or conveyed by the roadway to other locations downstream. All runoff eventually drains to the Cooks River via the drainage system outfalls. Due to the local topography overland flow to the Cooks River across wide areas is not possible (true for all but the largest events).

Detention basins are utilised at Marrickville Oval and the Sydenham storage pit to provide for temporary storage of floodwaters. A particular noteworthy drainage asset is the Malakoff Street Tunnel which conveys flow for approximately two kilometres from the Malakoff Street area to the Cooks River. The Malakoff Street Tunnel, constructed in approximately 1974, is likely indicative of severe drainage issues experienced previously (prior to mid 1970's). There is little detail available on the reasons for construction of the tunnel and the preceding flood issues. The Tunnel is a 3.3m x 2.9 m concrete box culvert with an average grade of 0.22%. It extends from the junction of the Western Channel at Malakoff Street to the Cooks River (Figure 2) and conveys most of the stormwater runoff from the upstream catchments in small events. Once the flows exceed the hydraulic capacity of the Malakoff Tunnel excess flows are conveyed to the Western Channel.

A levee has been constructed in Mackey Park to prevent inundation from flooding in the Cooks River. However this levee also prevents runoff from within the study area from exiting by overland flow.

1.4.3. Marrickville Oval

In 1996 Marrickville Oval was formalised as a flood retarding basin after a combination of a concrete reinforced masonry wall and an earthen bund were constructed around the oval. The Marrickville Oval is the only retarding basin within the catchment where flows are not pumped out.

1.4.4. Stormwater Pump Stations

Stormwater runoff is conveyed to three retarding basins in order to reduce flooding in some areas within the study area. The runoff is subsequently pumped into downstream channels for ultimate discharge into the Cooks River. The three pumping stations found in the study area are operated by Sydney Water and are:

1. **Sydenham Storage Pit (DPS1)**: The stormwater storage capacity is approximately 100 ML and two pumps operate in the basin (3 pumps, 2 operate with one on standby) with a peak pumping capacity of 2.5 m^3/s. The first pump cuts in at minus 8.87 mAHD and the second pumps cuts in at minus 8.48 mAHD. The flows are pumped into the Eastern Channel and then conveyed into the Cooks River. At 2.5 mAHD the water level in the retarding basin spills into the EC.

2. **Mackey Park Pumping Station (DPS2)**: DPS2 is located at the north west corner of Mackey Park and drains low lying area along the southern end of Carrington Road in the CC catchment.. The well capacity is approximately 230 m^3 and the pumping capacity is 1.4 m^3/s. The runoff is pumped into the Cooks River via two 800 mm rising mains.

3. **SPS271**: Located in the northern end of Carrington Road and constructed so that it could provide relief drainage by pumping water from CC to EC. The storage capacity is approximately 200 m^3. The pumping cut in level is at 0.45 mAHD at a maximum rate of 1.8 m^3/s. SPS271 pumps runoff into the Eastern Channel via a 750 mm rising main.

2. DATA COLLECTION

2.1. Previous Studies

Listed below are all the relevant studies carried out previously. These reports are reviewed in Appendix A.

- Marrickville Valley SWC 66 Capacity Assessment, Sydney Water 1995 (Reference 1)
- Marrickville Industrial Area Drainage Study, GHD 2002 (Reference 2)
- Eastern Channel East (ECE) Subcatchment Management Plan, Volume 2 Flood Study, Golder 2010 (Reference 3)
- Sydenham Stormwater Storage-Pit Pollutant Trap Study Willing & Partners, 1993 (Reference 4)
- Marrickville Oval Detention Basin Study, SMEC, 2010 (Reference 5)
- Cooks River Flood Study, MWH Parsons Brinckerhoff, 2009 (Reference 7)

2.2. Data Collected

2.2.1. Introduction

To carry out a flood study, various forms of data are required. The data is either needed to build models, to check the accuracy of model function (calibration/validation) or to facilitate the presentation of model results. The following sections describe the data collected for use in the study.

2.2.2. Topographic

The Digital Elevation Model (DEM) is derived from Airborne Laser Survey (ALS) which captures data points at approximately one metre spacing. The DEM for the study area is shown in Figure 4. The data were supplied as a 1m RASTER with a vertical accuracy of +/- 0.15 m. These data have been used to:

- define subcatchment and catchment watershed boundaries;
- inform the 2D model used in the study; and
- inform 1D network details where necessary (pit inlet heights, extend cross sections).

A number of cross-sections were obtained from Reference 1 in order to define 1D drainage elements (mainly channels) and these are summarised in Appendix D.

2.2.3. Rainfall Gauges

There are no pluviometers (continuously collects rainfall data) or daily rainfall stations (collects only 24hour - daily rainfall) located within the study area. The closest pluviometer is located approximately 3.5 kilometres away at Kingsford Smith Airport (distance calculated from approximate study area centroid). The nearest available rainfall station locations are provided on Figure 3.

Rainfall events causing flooding in the Marrickville Valley catchment can be very localised and as such will only be accurately “registered” by a proximate gauge. Gauges sited even only a kilometre away in coastal catchments such as the Marrickville Valley can show very different intensities and total rainfall depths than those experienced in the catchment.

One recommendation arising from this study is for Council to install a pluviometer within the catchment area. This will ensure that future rainfall events are accurately captured and data can be used for calibration of the modelling systems.

Table 2 is a summary of the official rainfall gauges located closest to the study area (refer Figure 3 for locations). There may be other unofficial gauges but data from these gauges have not been collected. Whilst daily rainfall gauges have been included, these records are generally not suitable for calibration/validation of the modelling process as they are only 24 hour totals and thus do not define the short duration intensities (say less than 6 hours) that produce flooding in the Marrickville Valley.

Table 2: Rainfall Stations

Number	Name	Opened	Type	Source
66036	Marrickville Golf Course	May-1904	Daily	BOM
66000	Ashfield Bowling Club	Jan-1894	Daily	BOM
66037	Sydney Airport AMO	Sept-1929	Daily	BOM
66037	Sydney Airport AMO	Aug-1962	Pluviometer	BOM
66194	Canterbury Racecourse AWS	Nov-1995	Daily	BOM

Generally in these types of studies an analysis of the pluviometer data is undertaken for each historical event to place the magnitude of past storm events in some context. However as there are no accurate records of significant flooding in the catchment since construction of the Malakoff Tunnel this exercise was not undertaken. Rather the available records from Sydney Airport were analysed to identify the most intense rainfall events within the last 4 years (i.e could be remembered by the residents and Council staff as causing flooding). The identified events have been plotted and are shown in Figure 5. All records plotted are below the 1Y ARI and as such are very minor events. However, it should be noted the rainfall that fell on the study area may not equate to that recorded at Sydney Airport.

Reference 3 indicates that the largest event (duration less than 1 hour) recorded in the last 50 years occurred in March 1966 although the March 1974 event was of greater magnitude for the 2 hour and greater durations.

2.2.4. Pit and Pipe Network

Council provided a database of the pit and pipe network, a summary of which is shown on Tables 3 and 4. The physical details included:

- Coordinates of each pit;
- Linkage between pits;

- Pipe dimensions; and
- Pit details (type of pit, inlet type and dimensions and depth to invert).

Many of the larger (trunk) drainage assets are owned and maintained by Sydney Water (pipes above 1200 mm diameter). The details for these were obtained from Reference 1. Where the pit and pipe information was not available from Council's database, it was obtained from the Marrickville Industrial Area Drainage Study (Reference 2) DRAINS model, from StreetView in Google Maps or by site inspection. The surface levels of pits were obtained from the DEM in some cases.

Table 3: Pit Data Summary

Pit Type	**Number**
Assumed Outlet	7
Buried Pit	339
Channel Node	13
Converter	25
Double Grated Gully Pit	2
Extended Kerb Inlet	43
Gully Pit & Extended.Kerb Inlet	662
Headwall Inlet	24
Headwall Outlet	32
Intersection Converter Inlet	289
Intersection Converter Outlet	76
Junction Pit Grated	248
Junction Pit Sealed	415
No Structure Inlet	3
No Structure Outlet	19
Side Entry Pit	47
Total	2,244

Table 4: Pipe Network Summary

Pipe Diameter	**Number**	**Length (m)**
< 300 mm	194	2,011
300 - 450 mm	656	9,753
450 - 525 mm	240	5,461
525 - 600 mm	68	1,603
600 - 750 mm	123	3,538
750 - 900 mm	88	2,982
900 - 1050 mm	73	1,586
> 1050 mm	12	1,817
Box Culvert	625	10,134
Total	2,079	38,885

2.2.5. Details on Other Hydraulic Structures

Details on structures (bridges, culverts etc) were obtained primarily from Reference 1 and Reference 2. Council also contributed other relevant sources of information, including survey data for the proposed Marrickville Oval detention basin plan and the Mackey Park levee.

2.2.6. General GIS Layers

GIS layers obtained from Council were used to aid model schematisation for the hydrologic model DRAINS and the two-dimensional hydraulic model (TUFLOW). More generally, they were used as backgrounds for figures presenting results and other details.

GIS layers obtained from Council include:

- ALS,
- Cadastre,
- Subcatchment boundaries,
- Topographic contours,
- Aerial photographs, and
- Zoning classifications.

2.2.7. Design Rainfall

Design rainfalls and temporal patterns were obtained from Australian Rainfall and Runoff (Reference 8). Probable Maximum Flood (PMF) design rainfall depths were obtained from Reference 9. The Intensity-Frequency-Duration (IFD) data for Marrickville is provided on Table 5.

Table 5: IFD Data for Marrickville

Location: 33.900S 151.175E NEAR.. marrickville Issued: 24/2/2011

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

	Average Recurrence Interval						
Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	100	128	162	182	208	242	267
6Mins	93.6	120	152	170	195	227	251
10Mins	76.7	98.5	126	141	162	189	210
20Mins	56.2	72.6	94.1	107	123	145	161
30Mins	45.8	59.3	77.4	88.1	102	121	135
1Hr	30.9	40.2	53.0	60.6	70.4	83.5	93.5
2Hrs	20.1	26.1	34.5	39.4	45.9	54.4	60.9
3Hrs	15.4	20.0	26.4	30.1	35.0	41.5	46.5
6Hrs	9.76	12.6	16.6	18.9	21.9	25.9	29.0
12Hrs	6.24	8.08	10.5	12.0	13.9	16.4	18.3
24Hrs	4.06	5.25	6.85	7.78	9.01	10.6	11.9
48Hrs	2.61	3.38	4.41	5.02	5.80	6.85	7.65
72Hrs	1.95	2.52	3.28	3.73	4.31	5.08	5.67

(Raw data: 40.56, 8.06, 2.52, 84.98, 16.36, 5.09, skew=0.00, F2=4.29, F50=15.86)

The rainfall for the PMF event has been developed using the Generalised Short-Duration Method (Reference 9). This approach is suitable for small catchments, such as Marrickville Valley.

The maximum duration for which the method is applicable in Marrickville is 6 hours. The parameters used for estimating the PMP are:

- Terrain classification: smooth;
- Adjustment for catchment elevation (EAF): 1;
- Moisture Adjustment Factor (MAF): 0.7, and;
- Ellipses enclosing the catchment: A and B.

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

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

Storm Duration (hours)	Ellipse A (mm)	Ellipse B (mm)
0.25	160	140
0.50	240	210
0.75	300	260
1.00	350	310
1.50	400	350
2.00	440	400
2.50	470	420
3.00	500	440
4.00	540	490
5.00	580	530
6.00	620	560

2.2.8. Downstream Boundary Water Levels

The downstream boundary of the study area is the Cooks River. The Cooks River is tidal so natural variability of water level is expected at the Cooks River from both tidal and catchment flows. For design flood estimation a level in the Cooks River is required for calculation of water levels in the lower parts of the catchment. There is no definitive approach for determining the coincidence of flooding in the Marrickville Valley with a water level in the Cooks River. Flooding could occur on a low or high tide and be coincident with runoff from the Cooks River catchment or with no significant runoff event in the Cooks River.

Earlier modelling undertaken in Reference 2 adopted the Mean High Water Spring (MHWS) level of 0.625 mAHD as the downstream boundary water level, though the Cooks River Flood Study (Reference 7) adopted a level in Botany Bay of 1.1 mAHD. A level of 1.1 mAHD approximates the Highest Astronomical Tide (HAT) which occurs approximately twice per year whilst the MHWS occurs 40 times per year.

In this study a similar approach to Reference 2 (using the MHWS) was adopted with sensitivity analysis undertaken to asses the implications of a higher level.

2.2.9. Historical Flood Information

No stream gauges exist within the study area and as such no recorded water levels or flows are available for historical events. A data search was carried out to identify the dates and magnitudes of historical floods. The search concentrated on the period since approximately 1970 as rainfall data collected prior to this date would generally be of insufficient definition for model calibration. Unfortunately there are no stream height gauges in the catchment or any other means of reliably determining the magnitude of past flood events. The only source of flood height data was from Sydney Water which is shown on Table 7 and Figure 2 (the exact location of the levels is unknown).

Table 7: Flood Records Obtained from Sydney Water

Point ID Figure 2	Branch No.	Location	Date	Depth (m)	Level above floor (m)	Level above coping (m)	Remarks	Data rating	Sydney Water source
1	66HA	Edgeware Rd, Marrickville	19/04/1950				Some flooding, not extensive	2B	A
2	66W	Fitzroy St, Marrickville	25/09/1951			0.84	Claims by owners of Fitzroy St	3	A
3	66W	Fitzroy St, Marrickville	25/09/1951			0.84	Flooded 2 factory premises, some damage	1	C
4	66W	Fitzroy St, Mville & Sth side of Smith St Marrickville	23/03/1952			0.56	Flooded some backyards Sth side of Smith St. Entered factory at Fitzroy St.	2B	C
5	66W	Fitzroy St, Marrickville	23/03/1952			0.56	Upstream of Fitzroy St. Flooded backyards on Sth side Smith St & entered Fitzroy St	1	A
6	66W	Fitzroy St, Marrickville	14/06/1952		0.03		Flooded Fitzroy St	1	D
7	66	Edinburgh Rd, Marrickville	27/01/1955				Flooded rear of Marrickville Margarine	1	D
8	66	Smith St, Marrickville	30/04/1955	4.71		3.68		1	D
9	66A	Carrington Rd, Marrickville	10/02/1956	0.62				2B	D
10	66	Fitzroy St,Marrickville	28-29/03/1957			0.38	Upstream side of Edinburgh Rd. No damage	2B	A
11	66	Smith St, Marrickville	28-29/03/1957		0.25		Considerable damage caused to houses on Sth Western side	1	D & E
12	66	Garden St, Marrickville	28-29/03/1957	1.32		0.33	Flowed thru Sydney Cotton Mills - damage to cotton & thru Sydney Water workskeds	1	C
13	66A	Carrington St, Marrickville	28-29/03/1957		0.25		Flooded goods store & lab of Duly & Hansford	1	C
14	66A/66B	Carrington Rd, Marrickville	28-29/03/1957		0.18		Flooded rear of cottage Nthn cnr Myrtle St & Carrington Rd	3	A
15	66B	Silver St, Marrickville	28-29/03/1957		0.05		Silver St damage to carpet & gardens. Silver St house flooded throughout	3	A
16	66B	Garners Ave, Marrickville	28-29/03/1957		0.05		Factory flooded	3	D
17	66H	Edgeware Rd, Marrickville (cnr Alice St)	28-29/03/1957				Water ponded only	1	D
18	66H	Victoria Rd, Marrickville (near Murray St)	28-29/03/1957			0.86	Upstream side of Victoria Rd. No damage	1	D
19	66P	Garden St, Marrickville	28-29/03/1957				Water ponded only	2B	A & E
20	66Q	Sydney St, Marrickville	28-29/03/1957	0.62			Water ponded only	1	A & E
21	66QA	Lane at rear of Sydney St, Marrickville	28-29/03/1957					2B	D & E
22	66QA & 66V	Buckley St, Sydney St & Garden St, Marrickville	28-29/03/1957				Water ponded only	2B	A
23	66T	Buckley St, Marrickville	28-29/03/1957				Water ponded only	1	C

24	66A	Renwick St, Marrickville	8/01/1958				Surcharge of gullies in Renwick St. No damage	2B	A & E
25	66A	Renwick St, Marrickville	10/02/1958	0.71			Factory (Duly & Hansford) flooded, some damage	1	D & E
26	66A	Renwick St, Marrickville	18-19/02/1959				Slight flooding adjacent to Duly & Hansford	3	A
27	66B	Silver St, Marrickville	18-19/02/1959				Guard fence on upstream side of culvert demolished	3	D & E
28	66B	SWC Corp. Land Myrtle St	18-19/02/1959			0.03		2B	C
29	66W	Smith St, Marrickville	18-19/02/1959			0.71	Flooded terrace houses in Smith St & Hatblocks Ltd in Fitzroy St	1	C
30	66Z	Sydenham Rd, Marrickville	18-19/02/1959			0.25	Surcharge of channel	2B	D
31	66G	Campbell St, St Peters	20/07/1959				No damage but rose to verandah height	1	D
32	66A	Renwick St, Marrickville	30/10/1959			0.25	At entrance to Duly & Hansford. No damage	2B	D
33	66G	Campbell St, Marrickville	30/10/1959		0.35		Cottages in May St & Campbell St & Hutchinson St flooded	3	B
34	66G	Unwins Bridge Rd, Marrickville	30/10/1959		0.77		Entered hotel at cnr Campbell St & Unwins Bridge Rd	2B	D
35	66W	Fitzroy St, Marrickville	30/10/1959		0.56	1.17	Flooded Aust. Ind Finishers P/L at rear & slightly less at front of bldg	1	D
36	66W	Fitzroy St, Marrickville	30/10/1959		0.23	1.22	McCann's Wollongong Transport also flooded. Paling fence at channel crossing collapsed.	1	D
37	66	Sydenham Stormwater Pit, Railway Pde, Sydenham	17-24/11/1961	5.13			Channel overflowed	1	D
38	66	Victoria Rd, Marrickville	17-24/11/1961	0.33			Above footpath on upstream side	1	D
39	66	Fitzroy St, Marrickville	17-24/11/1961		0.25	0.86	Above floor level of houses fronting Smith St. Also entered McCanns Transport	1	D
40	66	Railway Pde, Sydenham (DPS 1)	17-24/11/1961				Entrance of Rising Main from P.S. O/flowed into partly constructed amplifying channel	2B	D
41	66A	Railway Pde, Sydenham (DPS 1)	17-24/11/1961			0.25	Aggravated by O/flow from Western Ch	1	D
42	66B	Myrtle St, Marrickville	17-24/11/1961				Flooded Steel Merchants, CP Hopper P/L	2B	C
43	66G	Unwins Bridge Rd, Marrickville	17-24/11/1961		0.03		Entered hotel basement	1	C
44	66G	Brown St, Marrickville	17-24/11/1961		0.05		Enters from rear of shop on cnr and flows out into Campbell St	1	D
45	66G	Campbell St & Hutchinson St, Marrickville	17-24/11/1961	0.41			Backyards flooded only. Sewers in yards also surcharging	2B	D
46	66G	May St, Marrickville	17-24/11/1961				Flooded at rear, water entering some houses	2B	C
47	66H	Camden St, Marrickville	17-24/11/1961		0.18		Flooded Camden St & rear of several shops fronting Edgeware Rd	2B	C
48	66P	Garden Sts, Marrickville	17-24/11/1961				Flooded some premises in Shirlow St	2B	D
49	66G	Unwins Bridge Rd,Marrickville	28-29/08/1963	0.71			Flooded whole intersection, entered hotel basement & rose above ground floor	1	C
50	66H	Edgeware Rd, Marrickville	28-29/08/1963				Covered footpath, rose to gate step, did not enter property on Edgeware Rd	2B	C
51	66	Smith St, Marrickville	2/06/1975				Flooding caused by overland flow, inadequate Council drainage	3	A
52	66	Railway Pde, Marrickville (offtake to DPS 1)	17-20/03/1978	0.15			Flowed into P.S. via offtake	2B	C
53	66A	Marrickville Rd, Marrickville (Frazer Park)]	17-20/03/1978				Surface water covering park	1	A
54	66A	Railway Pde, Marrickville (offtake to	17-20/03/1978	0.45	0.15		Water flowed into P.S.	2A	C

		DPS 1)							
55	66A	Marrickville Rd, Marrickville (Frazer Park)	17-20/03/1978				Surface water covering part of park & debris	1	A
56	66A1	Unwins Bridge Rd, Marrickville (Bellevue Park?)	17-20/03/1978	0.15			Surcharging of M/S in centre of park	1	C
57	66A1	Unwins Bridge Rd, Marrickville (Bellevue Park?)	17-20/03/1978	0.15			Surface water covering part of park. M/H in centre of park was surcharging	1	C
58	66A2	Carrington Rd, Marrickville	17-20/03/1978				Parts still under water but was caused by blocked Council gullies	1	D
59	66A	Unwins Bridge Rd, Marrickville	21/03/1983				Localised flooding only adjacent to Tillman Park	1	D
60	66A1	Unwins Bridge Rd, Marrickville	21/03/1983					1	D
61	66B	Cary St, Marrickville	15/02/1990				Due to trash rack filled with rubbish during rain	1	D

DATA RATING: 1 - Well Defined Level and Location 2A - Level Well Defined, Location Unsure
2B - Level Unsure, Location Well Defined 3 - Approximate Only
SOURCE OF DATA: A - File No. 224152F9 B - File No. 225419F1 C - File No. 148839F1
D - Bunnerong SWC No. 11 General Flooding folder E - Flood Event folders

The data given in Table 7 indicates the following:

- a number of building floors have been inundated in the past, though generally the above floor depth is less than 0.3m. Possibly some of these buildings have now been re-developed at a higher level,
- the depth above the coping is generally up to 1m with a maximum of 3.68m (appears very high). This information is not surprising but cannot be used with confidence in model calibration due to the lack of a precise location and because in some places the coping may have been subsequently raised or lowered,
- there is little flood information since prior to construction of the Malakoff Tunnel in the mid 1970's. This suggests that either its construction has significantly reduced the flood problem, there have been no significant rainfall events or any flooding that has occurred has not been recorded. For this reason the value of the above information is limited for model calibration purposes.

In the absence of definitive historical peak flood levels suitable for model calibration the following sources were used:

- Reference 1 and 2 flow estimates;
- Community consultation results; and
- Comparison with previous studies such as Reference 3.

2.2.10. Community Consultation - Questionnaire

A Questionnaire (Appendix B) was sent to over 500 residents in July 2010 to areas that had experienced flooding problems within the study area in the past (as identified by Council). The questionnaire advised residents of the study and any flood information (photographs, description, peak levels etc.) thought relevant to the study.

The mail-out achieved a return rate of 5% (26 respondents in total) and as such the views expressed by this sample may not accurately reflect that of the total population. The returns consisted of both residential and non-residential property land owners/tenants who had experienced flooding dating back to December 2007. Of the 26 respondents only 11 reported flooding.

The locations of the returns are shown on Figure 6 with the respondents who reported flooding located on Figure 7 and a summary of the questionnaire results provided on Figure 8. In addition a set of photographs received from the community responses to the questionnaire has been provided to Marrickville Council.

In summary the reported events from the questionnaire are too small to be of value for model calibration. It would be worthwhile to undertake a similar questionnaire distribution immediately following the next large rainfall event.

3. HYDROLOGIC MODEL BUILD

3.1. Introduction

DRAINS (Reference 10) is a widely used hydrologic/hydraulic model and was adopted as the hydrologic model for this study.

DRAINS is a hydrologic/pipe hydraulics model that can simulate the full storm hydrograph and is capable of describing the flow behaviour of a catchment and pipe system for both observed and design events.

The DRAINS model is broadly characterised by the following features:

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

DRAINS generates a full hydrograph of surface flows arriving at each pit and routes these through the pipe network or overland, combining them where appropriate. Used in conjunction with a 2D hydraulic model, the benefit of DRAINS is that it produces a flow hydrograph at all modelled pits that can then be input into the 2D model.

3.2. Input Data

An extensive amount of data is required to establish the DRAINS model. The majority of the required input parameters and details for DRAINS were taken from the GHD study (Reference 2). Such parameters include the following;

- Pit location and type;
- Pit surface levels;
- Grate and inlet details;
- Pit naming convention;
- Pipe size, location and depth to invert;
- Pit connectivity;
- Time to concentration;
- Catchment areas;
- Overland flow paths; and
- Ku loss factor.

The GHD Drainage study (Reference 2) modelled 7 out of the 9 catchments within the current study area. The GHD Drains model was then used as a base and the two excluded catchments, the Malakoff Tunnel and Eastern Channel South subcatchments added. The data entered into the model was then cross checked against the Council survey data.

Sub catchment areas were obtained based on the ALS survey and assuming that properties drain to the street and flow in the street is along the gutters and in one direction only (i.e does not sub-divide at an intersection). This approach is a not strictly correct but is the generally accepted method for input into DRAINS (cannot accurately allow for flow splits at intersections). The resulting sub catchment areas are shown on Figure 9.

Only the pits which met the criteria of containing the information of Junction Pit Grated, Side Entry Pit, Assumed Outlet, Double Grated Gully Pit, Extended Kerb Inlet and Gully Pit + Extended Kerb Inlet were considered as inlet pits and therefore were the only structures capable of surcharging in the hydraulic models. The final pit and pipe layout is shown on Figure 10.

For each subcatchment area the proportion of pervious (grassed and landscaped), impervious (paved), supplementary area (paved area not directly connected to pipe system) were determined from field and aerial photographic inspections (shown in Appendix C) and summarised in Table 8. The adopted % imperviousness values are shown on Figure 11. For residential areas (includes roads) a relatively high value was adopted to reflect the likely low infiltration capacity of suburban yards and open space areas.

Table 8: Summary of Catchment Imperviousness values used in DRAINS

Area	Area (ha)	%
Paved Area	650	82
Grassed Area	140	18
Supplementary	0	0
TOTAL	**790**	100

3.3. DRAINS Model Build

The DRAINS model established for the study area includes 1,185 subcatchments. The drainage system defined by the model is made up of:

- runoff entry points representing surface inlet pits;
- bends, junctions or inspections locations which are termed pits with no inlet (i.e. the lid is sealed); and
- underground conduits (circular pipe or box) or open channel lengths between pits, called reaches.

A number of consecutive reaches is called a branch. The pipe system "tree" structure is defined by nominating the pits where two or more branches join. The length, slope, shape and dimension of each reach are specified, as well as representative inflow characteristics (surface inlet capacity) for each inlet pit.

A range of storm durations were modelled using the 1% AEP design to define the duration for the catchment. The adopted temporal pattern was the 2 hour storm since it produced the highest peak flows at the downstream end of each trunk drain. In the case of the PMF event, the 1 hour duration storm was adopted.

3.4. Adopted Model Parameters

Losses from a paved or impervious area are considered to comprise only an initial loss (an amount sufficient to wet the pavement and fill minor surface depressions). Losses from grassed areas are comprised of an initial loss and a continuing loss. The continuing loss was calculated from an infiltration equation curve incorporated into DRAINS and is based on the estimated representative soil type and antecedent moisture condition. It was assumed that the soil in the catchment has a slow infiltration rate potential and the antecedent moisture condition was assumed saturated. The latter was justified by the fact that the peak rainfall burst can typically occur within a longer event that has a duration lasting days. The adopted parameters are summarised in Table 9.

Table 9: Adopted 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 CONDITIONS	3
Description	Rather wet
Total Rainfall in 5 Days Preceding the Storm	12.5 to 25mm

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

The approach adopted for this study has been to assume 50% blockage at all culverts and pipes, with the exception of the trunk drainage structures such as the Eastern Channel, Central Channel, Western Channel and the Malakoff Tunnel. This approach has been adopted to take into account blockage caused by debris (cars, fencing, vegetation) being swept into drainage structures. In this study blockage has been assumed to occur at the culvert/pipe level instead of at pit inlets that was assumed in Reference 2. Reference 2 assumed blockage in sag pits (50%) and on-grade pits (20%) instead of pipe/culvert blockage.

4. HYDRAULIC MODEL BUILD

4.1. Introduction

TUFLOW (Reference 11) is a 1Dimensional/2Dimensional hydraulic model that has been adopted for use in this the present study. The TUFLOW modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water flow equations in two dimensions. TUFLOW has been widely used for a range of similar projects (including Reference 3). The model is capable of dynamically simulating complex overland flow regimes. It is especially applicable to the hydraulic analysis of flooding in urban areas characterised by critical short duration events and a combination of supercritical and subcritical flow behaviour.

For the hydraulic analysis of overland flow paths, a two-dimensional (2D) model such as TUFLOW provides several key advantages when compared to a traditional one-dimensional (1D) model. For example, in comparison to a 1D approach, a 2D model 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,
- 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 in detail across the model extent. This information can then be easily integrated into a GIS based environment enabling the outcomes to be incorporated into Council's planning activities.

4.2. TUFLOW Model Build

4.2.1. General

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

The hydraulic model encompasses the area between the Cooks River, the Princes Highway, the Inner West train line and the freight train line (Figure 12). Whilst the East Channel East subcatchment (ECE) flows are included in the present study, the East Channel East subcatchment has been investigated in a separate Flood Study (Reference 3). For a definition of design flood behavior in ECE, Reference 3 should be consulted.

4.2.2. Model Domain and Grid

A 3m by 3m 2D grid was generated from the ALS data based on the aerial photography available at the time of the study. A computational time step of 0.5 seconds was adopted. Buildings have been excluded from the model as it is assumed that there is very little flow through the structures and minimal temporary floodplain storage.

The earthen levee built at Mackey Park, which protects the lower area of the valley from flooding from the Cooks River, was included based on contour data detailing the levee crest obtained from Council.

4.2.3. Roughness Co-efficient

The Manning's "n" values for each grid cell were estimated from engineering experience and applied to the 2D overland area based on the terrain shown in Table 10.

Table 10: Manning's "n" values adopted in TUFLOW

Category	Manning's "n"	Description
1	0.04	Parks and Reservations
2	0.02	Bitumen road reserve and some car-parks
3	0.03	Railway Lines
4	0.12	Dense Vegetation

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

For this study it is considered that properties adjacent to the overland flow-path boundary would not be part of the effective flow path due to the presence of fences and buildings. This was achieved by extruding the building outlines in these areas (i.e restricts the flowpath).

Fences around the vicinity of train stations and in areas next to drainage paths were included in the TUFLOW model as they were considered to provide significant flow restrictions. However in other areas fences were not specifically included in the TUFLOW model.

The "loss" of temporary floodplain storage by extruding the building outlines is a slightly conservative assumption as in reality some floodwaters will enter these buildings. However this approach was adopted as it was considered that the impact of such an assumption would be small given the considerable amount of temporary floodplain storage elsewhere.

4.2.4. Pit and Pipe Network

Pit and pipe information incorporated in the DRAINS model was used to create a 1D drainage network in TUFLOW. Pipes of diameter smaller than 300mm were not included in the TUFLOW model as it was assumed that pipes of diameter smaller than 300mm would suffer from blockage during storms due to leaves and debris. Temporarily blockage may also occur during a storm as the pit entry may be restricted by a vehicle parking over the grate or leaves/silt/branches filling the inlet. However all pits and pipes were included in the DRAINS model.

The Malakoff Tunnel was modelled as a 1D structure in TUFLOW.

4.3. Boundary Conditions

Design flows from the DRAINS model (a hydrograph for each of the 1030 subcatchments) are applied at pit locations and used as inflows into the 2D model.

Stormwater runoff from the study area is conveyed to the Cooks River via a pipe/trunk system. For the purpose of this study a constant tailwater level of 0.625 mAHD was adopted in the Cooks River (refer Section 2 for further details).

4.4. Model Verification

In order to assess the performance of the TUFLOW hydraulic model, in the absence of suitable calibration events, modelled peak flows for various locations were compared against results from the Marrickville Industrial Area Drainage Study (Reference 2) and the Eastern Channel East Subcatctcment Management Plan (Reference 3) for the 20% AEP design event. The results are shown in Table 11 and 12. For the comparison with Reference 3 a unit catchment area flow was used.

Table 11: Comparison of Modelled Peak Flows with Reference 2 (values in m^3/s)

Locations	Reference 2	Present Study
Open channel u/s of Malakoff Tunnel	13.6	12.3
Western Channel d/s Bankstown Train Line	13.6	16.4
Flow arriving at DPS1 from MIA	13.8	12.6

Table 12: Comparison of Modelled Peak Flow with Reference 3

Locations	Reference 3	Present Study
Catchment	Eastern Channel East	Philcott Street (u/s Addison Road)
Area (ha)	131	18.7
Peak flow (m^3/s)	9.25	1.18
Unit area flow (m^3/s / ha)	0.071	0.063

This comparison shows minor differences in the peak flows at the selected locations. The differences in peak flows are attributed to the routing process involved in each model. DRAINS is a 1D hydraulic model where overland flowpaths, such as roads, must be estimated according to surface conditions above the pit inlets. On the other hand, TUFLOW uses ALS information to define overland flowpath and is fully 2D in the floodplain overland area.

The results generally indicate that the model system developed herein is performing in a comparable way to the previous studies.

5. DESIGN MODELLING RESULTS

5.1. Overview

Local topography restricts the amount of runoff that can flow south and directly into the Cooks River. Apart from the Eastern Channel the trunk drainage system has a limited capacity, relative to the 1% AEP event at least. Thus there is a significant amount of ponding of floodwaters within low lying areas such as within the Marrickville Industrial Area.

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

- Peak flood profiles for the main drainage channels on Figure 14 to Figure 17,
- Peak flood depths on Figures 18 to 22,
- Provisional flood hazard on Figures 23 to 27,
- Peak flood velocities on Figures 28 to 32,
- Comparison of peak depths and velocities in Table 16 for the locations shown on Figure 13,
- Comparison of provisional flood hazard categorisation in Table 17 for the locations shown on Figure 13.

5.2. Peak Outflows from Subcatchments

Table 13 indicates the peak outflows to the Cooks River from the four trunk drainage systems (Eastern, Central and Western Channels as well as the Malakoff Tunnel). It is noted that generally there is little difference between the peak flows in the 2Y ARI event and the 1% AEP event. For example, the Eastern Channel increases from 23.3 to 32 m^3/s and the Western Channel from 9.1 to 10.2 m^3/s. This indicates a lack of capacity in the channels for larger events as typically 1% AEP flows should be approximately twice as large as 2Y ARI flows.

Table 13: Peak Outflows to the Cooks River (m^3/s)

Channel	2Y ARI	20% AEP	10% AEP	1% AEP	PMF
Eastern Channel	23.3	27.2	29.2	32.0	54.7
Central Channel	1.5	1.8	2.0	3.6	4.0
Western Channel	9.1	9.4	9.6	10.2	17.5
Malakoff Tunnel	10.4	11.7	13.3	19.6	26.6

5.3. Properties Inundated

A similar situation to that described above is shown in Table 14 which indicates that the percentage of each subcatchment inundated (to a depth exceeding 150 mm) does not change markedly from the 2Y ARI event to the 1% AEP event. However, these results indicate that significant flooding issues exist within the study area in even the smallest events modelled.

Table 14: Percentage of Catchments with Peak Flood Depth greater than 150 mm

Subcatchment	2Y ARI	20% AEP	10% AEP	1% AEP	PMF
Eastern Channel West	20	25	27	34	42
Eastern Channel North	17	19	20	21	27
Eastern Channel South	12	14	15	16	20
Eastern Channel 2	9	13	13	15	20
Central Channel	16	22	25	34	44
Western Channel	13	15	16	18	23
Malakoff Street	25	27	27	29	33
Malakoff Tunnel	12	13	13	14	18

Table 15 indicates the number of properties inundated in the various design events with depths greater than 150mm. This indicates that a substantial number of properties are inundated in the smaller design events but the increase from say the 2Y ARI to the 1% AEP is only approximately 25%.

Table 15: Number of Properties Inundated (peak depth greater than 150mm)

Design event	Number of properties Inundated
2Y ARI	3151
20% AEP	3485
10% AEP	3638
1% AEP	3989
PMF	4828

5.4. Results at Key Locations

The results at key locations for peak flood depth, velocity and provisional hazard are shown on Tables 16 and 17 (refer Figure 13 for locations).

One feature of flooding within the study area (as noted above) is that due to the relatively large area of low lying land there is not a great difference between the peak flood depths in the more frequent events and the rarer events. This is shown in Table 16 which indicates that the 2Y ARI flood depth at the corner of Livingstone Road and Marrickville Road is 0.5 m, in the 1% AEP it is 0.6m and in the PMF 0.9m. This is a relatively small flood range but is typical of many similar urban catchments in Sydney. Along river systems the difference is generally much larger. One positive aspect of this limited “flood range” is that whilst residences and businesses within the study area are liable to regular over ground inundation the risk to life associated with the flooding is relatively low (i.e. relatively low hazard flooding occurs as shown on Table 17).

Table 16 also indicates that generally velocities are low and as such high hazard flows will generally be the result of the high depth criteria rather than the velocity depth product exceeding 1 (refer Reference 12 for flood hazard criteria).

Table 16: Peak Flood Depth and Velocity at Key Locations (refer Figure 13 for locations)

I.D. #	Location	2Y ARI		20% AEP		10% AEP		1% AEP		PMF	
		Depth (m)	Vel (m/s)	Depth (m)	Vel (m/s)	Depth (m)	Vel (m/s)	Depth (m)	Vel (m/s)	Depth (m)	Vel (m/s)
1	Marrickville Ave cnr Livingstone Rd	0.5	0.3	0.6	0.3	0.6	0.3	0.6	0.4	0.9	1.0
2	Greenbank St cnr Moyes St	0.4	1.4	0.4	1.5	0.5	1.6	0.5	1.6	1.0	1.4
3	Bankstown Line near Marrickville Train Station	1.1	0.3	1.2	0.3	1.3	0.3	1.5	0.4	2.4	0.7
4	Arthur St (lowest point, 130 m west of Ann St)	0.5	0.4	0.5	0.3	0.6	0.3	0.6	0.3	0.9	1.3
5	O'Hara St cnr Byrnes St	1.5	0.3	1.6	0.6	1.8	0.7	2.2	1.0	2.9	2.2
6	Livingstone Rd (near Tennis courts next to Marrickville Oval)	0.4	0.5	0.4	0.7	0.5	0.8	0.7	1.0	1.7	0.9
7	Petersham Rd cnr Boland Ln	0.4	0.4	0.5	0.4	0.5	0.5	0.7	0.6	1.7	0.7
8	Northcote St cnr Western Channel	0.3	0.1	0.3	0.2	0.3	0.6	0.5	1.3	1.9	0.9
9	Malakoff St (Western Channel and Malakoff Tunnel Junction)	0.5	0.4	0.6	0.4	0.7	0.4	0.9	0.5	2.3	1.0
10	Sydenham Rd cnr Illawara Rd	0.1	0.9	0.2	1.1	0.3	1.2	0.4	1.5	1.5	2.8
11	Hogan Ave cnr Burrows Ave	0.3	0.4	0.3	0.4	0.3	0.4	0.4	0.5	1.4	1.4
12	Unwins Bridge Rd cnr Terry St	0.6	0.6	0.7	0.6	0.8	0.6	1.0	0.7	1.6	1.4
13	Garden St cnr Shirlow St	0.1	0.0	0.2	0.1	0.3	0.5	0.7	1.1	2.3	2.1
14	Saywell St cnr Sloane St	0.0	0.2	0.2	0.1	0.4	0.3	0.8	0.9	2.2	1.6
15	Sydenham Rd cnr Fitzroy St	0.2	0.3	0.3	0.4	0.4	0.4	0.8	0.4	2.1	1.0
16	Victoria Rd cnr Rich St	0.4	2.0	0.5	1.4	0.5	1.4	0.7	2.0	1.4	2.3
17	Addison Rd cnr Philpott St	0.3	0.5	0.3	0.6	0.4	0.7	0.5	0.8	1.1	1.6
18	Illawara Rd 120m south of Addison Rd	0.4	0.4	0.6	0.4	0.6	0.4	0.9	0.5	1.7	0.7
19	Mackey Park	Not flooded		0.1	0.0	0.3	0.1	0.9	0.1	1.6	0.5

Table 17: Provisional Flood Hazard Categories at Key Locations (refer Figure 13 for locations)

I.D. #	Location	2Y ARI	20% AEP	10% AEP	1% AEP	PMF
1	Marrickville Ave cnr Livingstone Rd	L	L	L	H	H
2	Greenbank St cnr Moyes St	L	L	L	H	H
3	Bankstown Line near Marrickville Train Station	H	H	H	H	H
4	Arthur St (lowest point, 130 m west of Ann St)	L	L	L	L	H
5	O'Hara St cnr Byrnes St	H	H	H	H	H
6	Livingstone Rd (near Tennis courts next to Marrickville Oval)	L	L	L	L	H
7	Petersham Rd cnr Boland Ln	L	L	L	H	H
8	Northcote St cnr Western Channel	L	L	L	H	H
9	Malakoff St (Western Channel and Malakoff Tunnel Junction)	L	L	L	H	H
10	Sydenham Rd cnr Illawara Rd	L	L	L	H	H
11	Hogan Ave cnr Burrows Ave	L	L	L	L	H
12	Unwins Bridge Rd cnr Terry St	L	H	H	H	H
13	Garden St cnr Shirlow St	L	L	L	H	H
14	Saywell St cnr Sloane St	L	L	L	H	H
15	Sydenham Rd cnr Fitzroy St	L	L	L	H	H
16	Victoria Rd cnr Rich St	H	H	H	H	H
17	Addison Rd cnr Philpott St	L	L	L	L	H
18	Illawara Rd 120m south of Addison Rd	L	L	L	H	H
19	Mackey Park	Not flooded	L	L	H	H

(Note: L refers to Low Hazard and H refers to High Hazard categories)

5.5. Sensitivity Analysis

A sensitivity analysis was carried out in order to assess the effect of varying model parameter values on the 1% AEP modelled results. The following scenarios were modelled:

- An increase in rainfall losses (soil type 2 adopted in hydrological model);
- A decrease in rainfall losses (soil type 4 adopted in hydrological model);
- An increase in routing lag of 20%;
- A decrease in routing lag of 20%;
- An increase in bed resistance (Manning's 'n') of 20%;
- A decrease in bed resistance (Manning's 'n') of 20%;
- Pipe/culvert blockage increase of 50%, and;
- Pipe/culvert blockage decrease of 50%.

A summary of the results obtained are shown in Table 18. Overall results were shown to be relatively insensitive to; rainfall losses, routing, roughness and blockage with results tending to be +/- 0.1 m which can generally be accommodated within the 0.5 m freeboard applied to 1% AEP results to determine the Flood Planning Levels (FPLs).

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

Table 18: Results of Sensitivity Analyses – 1% AEP event

I.D. #	Location	1% AEP Peak Flood Level (mAHD)	Increase in rainfall losses	Decrease in rainfall losses(1)	Routing lag increase by 20%(2)	Routing lag decrease by 20%	Bed resistance increase by 20%	Bed resistance decrease by 20%	Pipe/Culvert Blockage increase by 50%	Pipe/Culvert Blockage decrease by 50%
			Difference with 1% AEP base case (m)							
1	Marrickville Ave cnr Livingstone Rd	17.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	Greenbank St cnr Moyes St	13.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	Bankstown Line near Marrickville Train Station	9.5	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	Arthur St (lowest point, 130 m west of Ann St)	14.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	O'Hara St cnr Byrnes St	6.5	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	Livingstone Rd (near Tennis courts next to Marrickville Oval)	13.4	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	Petersham Rd cnr Boland Ln	11.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	Northcote St cnr Western Channel	8.0	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	Malakoff St (Western Channel and Malakoff Tunnel Junction)	7.8	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	Sydenham Rd cnr Illawara Rd	6.6	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	Hogan Ave cnr Burrows Ave	3.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	Unwins Bridge Rd cnr Terry St	4.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	Garden St cnr Shirlow St	2.0	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.0
14	Saywell St cnr Sloane St	2.3	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	-0.1
15	Sydenham Rd cnr Fitzroy St	2.4	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	-0.1
16	Victoria Rd cnr Rich St	5.4	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.0
17	Addison Rd cnr Philpott St	9.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	Illawara Rd 120m south of Addison Rd	10.0	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	-0.1
19	Mackey Park	2.3	-0.2	0.0	0.0	0.0	0.0	0.0	-0.1	0.0

1 Soil type 2 (Moderate infiltration rate), grassed area depression storage: 10 mm, paved area depression storage 2 mm
2 Soil type 4 (High runoff potential), grassed area depression storage: 2.5 mm, paved area depression storage 0.5 mm

5.6. Detailed Description of Results

5.6.1. Eastern Channel

The EC conveys stormwater runoff from approximately 437 ha (55% of the total catchment area). The contributing catchment to the EC (refer to Figure 2) is located north and northeast of the MIA and most of the urban area located east of the Illawarra Railway Line. Additionally, the EC receives flows from the Sydenham Stormwater Pit (DPS1) and SPS271. Design peak flood profiles are shown on Figure 14.

Along its entire reach the channel has the hydraulic capacity to convey stormwater flows for the 1% AEP and smaller events. The modelled flood depth reaches 1.2 m just upstream of the DPS1 intake in the 2Y ARI event and 1.4 m in the 1% AEP. Where the channel flows below Richardson Crescent the modelled flood depth is 1.6 m in the 2Y ARI event and 1.9 m in the 1% AEP event.

Before reaching the Cooks River, modelled peak discharges in the channel range from 23.3 m^3/s in the 2Y ARI to 31.8 m^3/s in the 1% AEP event. Modelled peak velocities are 3.0 m/s and 3.6 m/s in the 2Y ARI and 1% AEP events respectively.

Sydenham Railway Station

Sydenham Railway Station appears to be inundated by stormwater flows originating on the southern side of the line with inundation in all events modelled (2Y ARI to PMF). Overland stormwater flow originates from surcharged pits on Hogan Avenue and Unwins Bridge Road. A significant overland flow occurs through a car park located between Unwins Bridge Road and the end of Bolton Street.

Bolton Street is inundated by up to 1.2 m in the 2Y ARI and 1.4 m in the 1% AEP event. No stormwater from Bolton Street crosses the railway tracks in the 2Y ARI, instead flows are conveyed through the gutter towards the train station. Stormwater does flow down Hogan Avenue and it is this flow which eventually flows into the station.

Stormwater flowing from Bolton Street crosses over the railway tracks towards the EC in the 1% AEP event.

Sydenham Storage Pit (DPS1)

Pumping activity and stage levels for the all design events at DSP1 are shown in Figure 33. Table 19 indicates the stormwater volumes from the MIA pipe network and the EC into the basin.

Table 19: Total Volume of Flood Waters entering DPS1 and Peak Levels

Design Event	Volume (m^3)	Peak level (mAHD)
2Y ARI	40,886	-7.5
20% AEP	61,054	-6.6
10% AEP	72,407	-5.3
1% AEP	90,694	0.1

As observed in Table 19 the maximum water level modelled in the basin reaches 0.1 mAHD in the 1% AEP event. It should be noted that the peak levels in Table 19 assume that the basin is empty at the start of the storm. Additionally, pipes and culverts conveying stormwater to the basin have been modelled adopting a 50% blockage, therefore reducing the capacity to convey flow. In reality this may not necessarily be the case and thus the peak levels in the basin should not be adopted for design purposes.

Also it is likely that a longer duration event with greater volume may result in higher flood levels for a given AEP than shown in Table 19. If required this should be evaluated using some form of “embedded storm approach”.

In the 1% AEP event 2 hour event the basin still has capacity to store an additional 10ML. In larger events the water level (or in a smaller event if there was some water in the basin at the start of the design storm and pipes and/or culverts have less less blockage) will start “spilling” first into Garden Street upon reaching 1.5 mAHD and into the EC at 2.5 mAHD.

If pumping failure was to occur in DPS1 stormwater would start “spilling” into Garden Street exacerbating flood levels in the MIA. This would happen if the catchment receives an effective rainfall of 133 mm. Rainfall events of this magnitude are greater than a 1% AEP 3 hour or a 10% AEP 12 hour event. Generally though, pump failure will not impact on peak flood levels but will impact on the duration of inundation.

5.6.2. Central Channel

The CC conveys stormwater flows from the urban area located on the eastern side of the Illawarra Railway Line in the vicinity of Tempe and Sydenham. Design peak profiles for the channel are provided on Figure 15.

The bank of the channel is exceeded by 0.45 m in the 2Y ARI event at the upstream end and in the 1% AEP the peak depth is almost 1.2 m above the channel bank at the same location.

The hydraulic slope is approximately zero in the channel. Modelled peak velocities in the channel are on average 0.4 m/s in the 2Y ARI to 0.7 m/s in the 1% AEP. The relatively high tailwater level in the Cooks River is another factor which compromises the hydraulic capacity of this channel (assumed tailwater is 0.625 mAHD).

Upstream of the channel, where Unwins Bridge Road passes underneath the freight train line there is a trapped low point. Stormwater ponds at this location due to the limited capacity of the pit inlets. The peak flood depth in the 2Y ARI is 0.9 m and in the 1% AEP is 1.3 m.

A 0.9 m circular pipe takes stormwater flows from Unwins Bridge Road underneath Tillman Park towards the Eastern Channel. This pipe is at capacity in all design events modelled. Tillman Park is largely flood free in the 2Y ARI event, however, the 1% AEP event inundates the park with modelled peak depths averaging 0.7 m and a maximum of 2.5 m at its lowest point (near the Illawarra Railway Line).

SPS271

During all modelled scenarios the pumping station is assumed to be operating 100% of the time. Due to the relatively low capacity and short nature of the design critical durations storms the pumps are unable to prevent surcharging of SPS271. Flood levels in the vicinity of this location might be adversely affected if pumping failure was to occur in small events (approximately a 2Y ARI). In greater events the impact on flood levels would be negligible.

Carrington Road

Carrington Road lies between the Western and Central channels. Stormwater pipes underneath Carrington Road connect the CC to Mackey Park Pumping Station (DPS2) and the Cooks River.

At its northern end, overland stormwater runs in two directions. A portion flows towards a 1800mm x 1600mm box culvert connected to the CC. The other flows southwards towards Mackey Park. At the northern end of Carrington Road the peak flood depths vary from a maximum of 0.7 m in the 2Y ARI to 1.5 m in the 1% AEP. In the 1% AEP event Carrington Road appears completely inundated. The average peak depth is approximately 0.3 m and 0.8 m for the 2Y ARI and 1% AEP events respectively. No stormwater pipe network was identified between Warren Road and Myrtle Street and this will likely exacerbate the extent and magnitude of inundation in the vicinity.

Mackey Park

A levee embankment (crest at 2.75 mAHD) exists at the southern end of Mackey Park. It was built to prevent inundation from the Cooks River. The park is not inundated during the 2Y ARI event but gradually becomes inundated in larger events. Most of the runoff that inundates Mackey Park comes from Carrington Road. Peak flood depths within the park reach 1.0 m. The levee is overtopped by less than 0.1 m depth in the 1% AEP event.

During all modelled scenarios the pumping station (DPS2) is assumed to be operating 100% of the time. Due to the relatively low capacity and short nature of the design critical duration storms the pumps are unable to prevent surcharging of DPS2. Flood levels in the vicinity of this location might be adversely affected if pumping failure were to occur in small events (approximately 2Y ARI). In greater events the impact on flood levels would be negligible.

5.6.3. Western Channel

The WC conveys flows from the suburb of Marrickville, east of the junction with the Malakoff Tunnel, and some low lying areas close to the Cooks River. Design peak profiles for the WC are shown on Figure 16.

The channel is crossed by 13 roads from Petersham Road to Garners Avenue where the channel changes to a rectangular box culvert (3000 mm x 1700 mm). At Malakoff Street stormwater runoff is transferred to the Malakoff Tunnel. The flood behaviour is also affected by stormwater flowing down from Sydenham Road towards the pit inlets that discharge into the

WC. Pit inlets located from Convent Lane down to Illawarra Road have low inlet capacity to convey stormwater to the WC in the 2Y ARI and larger events. Flood depths over Garners Avenue range from 0.7 m in the 2Y ARI up to 1.0 m in the 1% AEP event.

Downstream of Garners Avenue the main flowpath becomes a rectangular box culvert that discharges into an open channel south of the Bankstown Railway Line. This culvert has the capacity to convey the 1% AEP and smaller events until reaching the open channel. The peak flow at this location is 2.4 m^3/s in the 2Y ARI event and 3.8 m^3/s in the 1% AEP event.

A low point is noted where Victoria Road crosses the Bankstown Railway Line. The peak flood depth reaches 0.3 m in the 2Y ARI and 0.7 m in the 1% AEP impacting on road traffic at this location. Downstream stormwater flows along the rear of properties located between Warren Road and Cary Street and towards the channel. This inundation occurs due to stormwater runoff travelling on these streets not being conveyed to the channel due to the low pit inlet capacity. Peak flows range from 9.4 m^3/s in the 2Y ARI up to 11.2 m^3/s in the 1% AEP event. Peak velocities are between 1.5 and 1.7 m/s in the 2Y ARI and 1% AEP respectively.

Marrickville Railway Station

Marrickville Railway Station lies within the Western Channel subcatchment and is located west of the crossing between the Western Channel and the Bankstown Railway Line. The station is inundated by runoff from local subcatchments in all design events modelled. The main culvert conveying the flows underneath the railway tracks is a 1500 mm x 900 mm box culvert. Stormwater runoff from Moyer Street and Greenback Street that does not enter into the underground network traverses McNeilly Park and flows onto the railway line and then towards the station. The existing pit inlet located 60 m west of the station does not provide sufficient capacity to convey flows into the main box culvert.

Peak flood depths underneath the Illawarra Road bridge range from 0.4 m in the 2Y ARI up to 0.9 m in the 1% AEP. Stormwater over the railway tracks flows eastwards towards an open channel that runs parallel to Byrnes Street. Pit inlets located on the corner of O'Hara Street and Byrnes Street do not provide enough capacity and stormwater flows towards the previously mentioned open channel. At this corner, peak flood depths can reach 1.3 m in the 2Y ARI event and 2.1 m in the 1% AEP event.

It should be noted that railway ballast may provide considerable temporary floodplain storage as well as infiltration capacity and this has not been simulated in the modelling.

Marrickville Avenue

Marrickville Avenue is located west of Marrickville Railway Station. Stormwater runoff which does not enter the pipe network flows overland towards the Bankstown Railway Line before the intersection with Livingstone Road. Runoff in this area then flows towards Marrickville Station. At the nearby railway tracks the modelled peak flood depth in the 2Y ARI event is 0.3 m and in the 1% AEP is 0.4 m. At the lowest point of Marrickville Avenue the peak flood depths are 0.5 m in the 2Y ARI and 0.6m in the 1% AEP.

Arthur Street

On Arthur Street, between Ann Street and Livingstone Road, there is a trapped low point. A rectangular grated pit of 2000 mm x 1000 mm conveys local runoff into a pipe system that discharges into the Malakoff Tunnel. This is a critical location since the topography of the contributing catchment consists of steep roads that convey stormwater runoff to this location.

Modelled peak flood depths range from 0.5 m in the 2Y ARI event to 0.6 m in the 1% AEP.

Malakoff Tunnel

The Malakoff Tunnel extends from the intersection of the Western Channel with Malakoff Street to the Cooks River (Figure 2). Modelled peak discharge ranges from 10.4 m^3/s in the 2Y ARI to 19.6 m^3/s in the 1% AEP in the tunnel. Modelled peak velocities range from 3.2 m/s to 4.0 m/s in the 2Y ARI and 1% AEP event respectively.

Marrickville Industrial Area (MIA)

Due to its relatively flat topography and large upstream catchment size this area is one of the most critical locations in the study area. Stormwater exits into the Sydenham storage pit, in which flows are later pumped into the Eastern Channel.

The 2Y ARI event only causes some localised flooding in the MIA and these are described below.

- **Corner of Sydenham Road and Victoria Road**: Stormwater flows over Sydenham Road in the direction of the Sydenham pit. The peak flood depth is 0.3 m in the 2Y ARI and 0.5 m in the 1% AEP event. With the inlet pit capacity at this location being exceeded by excessive overland flow, ponding occurs with associated low velocities. The 900 mm circular pipe flows at full capacity for both the 2Y ARI and 1% AEP events.

- **Sydney Street**: The pit inlet capacity is exceed at the lowest point of the road with ponding from 0.5 m in the 2Y ARI up to 1.2 m in the 1% AEP. Pipe flows are routed to the main pipe at Sydenham Road.

- **Fitzroy Street**: The area between Sydenham Road and Saywell Street is impacted by stormwater runoff. Flows originating from surcharged pits slowly move towards Saywell Street and Sydenham Road. Modelled peak flood depths vary from 0.4 m in the 2Y ARI to 0.9 m in the 1% AEP.

In 1% AEP event almost every road in the MIA becomes inundated. Stormwater runoff flows over Shirlow Street and Saywell Street directly discharging into the Sydenham pit without entering the pipe network. The most critical areas in the 1% AEP design event are Barclay Street, where the modelled peak flood depth reaches 1.6 m, Sloane Lane with 1.5 m depth and Buckley Street with 1.2 m depth.

5.7. Climate Change Analysis

5.7.1. Background

The 2005 Floodplain Development Manual (Reference 12) requires that Flood Studies and Floodplain Risk Management Studies consider the impacts of climate change on flood behaviour.

Since 2000, current best practice for considering the impacts of climate change (ocean level rise and rainfall increase) have been evolving rapidly. Key developments in the last few years have included:

- release of the Fourth Assessment Report by the Inter-governmental Panel on Climate Change (IPCC) in February 2007 (Reference 13), which updated the Third IPCC Assessment Report of 2001 (Reference 14);
- preparation of Climate Change Adaptation Actions for Local Government by SMEC Australia for the Australian Greenhouse Office in mid 2007 (Reference 15);
- preparation of Climate Change in Australia by CSIRO in late 2007 (Reference 16), which provides an Australian focus on Reference 15;
- release of the Floodplain Risk Management Guideline Practical Consideration of Climate Change by the NSW Department of Environment and Climate Change in October 2007 (Reference 17 - referred to as the DECC Guideline 2007);

In October 2009 the NSW Government issued its Policy Statement on Sea Level Rise (Reference 18) which states: "*Over the period 1870-2001, global sea levels rose by 20 cm, with a current global average rate of increase approximately twice the historical average. Sea levels are expected to continue rising throughout the twenty-first century and there is no scientific evidence to suggest that sea levels will stop rising beyond 2100 or that the current trends will be reversed.*

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

In August 2010 the NSW State Government Department of Environment, Climate Change and Water issued the Flood Risk Management Guide (Reference 19): Incorporating sea level rise benchmarks in flood risk assessments.

As a result of the information provided in the above and other documents, and to keep up-to-date with current best practice, this study incorporates an assessment of climate change. It should be noted that the estimated rise in ocean/sea level along the NSW coast varies between the above reports and at this time there is no absolute value that has been adopted by all experts.

The climate change scenarios in the DECC Guideline 2007 (Reference 17) suggested for undertaking sensitivity analysis in flood studies are indicated below.

ocean/sea level rise:

low level ocean rise	=	0.18 m,
medium level ocean rise	=	0.55 m,
high level ocean rise	=	0.91 m.

increase in peak rainfall and storm volume:

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

A high level rainfall increase of up to 30% is recommended for consideration due to the uncertainties associated with this aspect of climate change and to apply the "precautionary principle". It is generally acknowledged that a 30% rainfall increase is probably overly conservative and that a timeframe for the provision of definitive predictions of the actual increase is unknown. The DECC Guideline 2007 (Reference 17) is currently the only reference providing guidelines for rainfall increases due to climate change.

The most recent guideline (Reference 19) indicates a 0.9m ocean level rise by the year 2100 and a 0.4m rise by the year 2050 and thus supersedes those sea level rise guidelines provided in the DECC Guideline 2007. However it should be noted that climate change (man made or due to natural processes) will still occur beyond 2100.

5.7.2. Approach

As part of the study the following climate change scenarios have been analysed for the 1% AEP event:

- 0.4m rise in tailwater level in the Cooks River,
- 0.9m rise in tailwater level in the Cooks River,
- 10% increase in design rainfall intensity,
- 20% increase in design rainfall intensity,
- 30% increase in design rainfall intensity,
- 10% increase in design rainfall intensity PLUS a 0.4m rise in tailwater level in the Cooks River,
- 10% increase in design rainfall intensity PLUS a 0.9m rise in tailwater level in the Cooks River,
- 30% increase in design rainfall intensity PLUS a 0.4m rise in tailwater level in the Cooks River.

The results are shown in Table 20 (refer Figure 13 for locations) and Table 21 (refer Figure 35 for locations) and Figures 34 and 35.

Table 20: Climate Change Results

I.D. #	Location	1% AEP Peak Flood Level (mAHD)	10% Increase in Design Rainfall Intensity	20% Increase in Design Rainfall Intensity	30% Increase in Design Rainfall Intensity	0.4 m rise in tailwater level in the Cooks River	0.9 m rise in tailwater level in the Cooks River	10% Increase in Design Rainfall Intensity + 0.4 m rise in tailwater level in the Cooks River	10% Increase in Design Rainfall Intensity + 0.9 m rise in tailwater level in the Cooks River	30% Increase in Design Rainfall Intensity + 0.4 m rise in tailwater level in the Cooks River
			Difference with 1% AEP base case (m)							
1	Marrickville Ave cnr Livingstone Rd	17.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	Greenbank St cnr Moyes St	13.9	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.1
3	Bankstown Line near Marrickville Train Station	9.5	0.1	0.1	0.2	0.0	0.0	0.1	0.1	0.2
4	Arthur St (lowest point, 130 m west of Ann St)	14.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	O'Hara St cnr Byrnes St	6.5	0.1	0.1	0.2	0.0	0.0	0.1	0.1	0.2
6	Livingstone Rd (near Tennis courts next to Marrickville Oval)	13.4	0.1	0.2	0.3	0.0	0.0	0.1	0.1	0.3
7	Petersham Rd cnr Boland Ln	11.1	0.1	0.2	0.2	0.0	0.0	0.1	0.1	0.2
8	Northcote St cnr Western Channel	8.0	0.1	0.2	0.3	0.0	0.0	0.1	0.1	0.3
9	Malakoff St (Western Channel and Malakoff Tunnel Junction)	7.8	0.1	0.1	0.2	0.0	0.0	0.1	0.1	0.2
10	Sydenham Rd cnr Illawara Rd	6.6	0.1	0.2	0.2	0.0	0.0	0.1	0.1	0.2
11	Hogan Ave cnr Burrows Ave	3.8	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.1
12	Unwins Bridge Rd cnr Terry St	4.1	0.1	0.1	0.2	0.0	0.0	0.0	0.1	0.2
13	Garden St cnr Shirlow St	2.0	0.1	0.1	0.3	0.0	0.0	0.1	0.1	0.4
14	Saywell St cnr Sloane St	2.3	0.1	0.2	0.3	0.0	0.0	0.1	0.1	0.3
15	Sydenham Rd cnr Fitzroy St	2.4	0.1	0.2	0.3	0.0	0.0	0.1	0.1	0.3
16	Victoria Rd cnr Rich St	5.4	0.1	0.1	0.2	0.0	0.0	0.1	0.1	0.2
17	Addison Rd cnr Philpott St	9.2	0.0	0.1	0.1	0.0	0.0	0.0	0.0	0.1
18	Illawara Rd 120m south of Addison Rd	10.0	0.1	0.1	0.2	0.0	0.0	0.1	0.1	0.2
19	Mackey Park	2.3	0.1	0.2	0.3	0.0	0.1	0.2	0.2	0.4

Table 21: Sea Level Rise Results

Label (refer Figure 35)	Location	Existing	Sea Level Rise 0.4m	Sea Level Rise 0.9m
1	**Western Channel**	2.36	2.36	2.36
2	**Mackey Park**	2.22	2.22	2.32
3	**Eastern Channel**	1.87	1.90	1.99
4	**Carrington Rd**	2.30	2.35	2.38
5	**Central Channel**	2.57	2.69	2.74

The results indicate that a 0.4m sea level rise will increase the 1% AEP flood levels by a maximum of 0.1m and a 0.9m sea level rise by a maximum of 0.2m. These increases are confined to the lower parts of the catchment (refer Figure 35).

The increase in the 1% AEP design rainfalls result in a more general increase in flood levels across the entire catchment. A 10% rainfall increase in design rainfalls results in approximately a 0.1m maximum increase in peak levels, a 20% rainfall increase a 0.2m maximum increase in peak levels and a 30% rainfall increase a 0.3m maximum increase in peak levels.

The combinations of a rainfall increase and a sea level rise increase indicated the same as the addition of the individual rainfall/sea level rise scenario increases.

6. CONCLUSIONS

The work carried out for this study cannot be verified by calibration to historical data (due to the lack of available data) but is based on best practice and does produce results for inundation and flows which are in line with previous investigations and expectations (References 1 to 6) and the limited information provided from resident surveys.

There is an extensive flood liability throughout the lower areas of the study area as a result of extensive development (filling of the floodplain and blocking of flow paths) in conjunction with pervious surfaces converted to impervious surfaces. The restricted overland flow paths to the south and into the Cooks River, as well as the possibility of local flows being backwatered by an elevated river level in the Cooks River exacerbate the flood liability of the area. In addition the minor and major drainage systems are limited in capacity.

The minor system, which consists of the pit/pipe network and trunk drainage elements, does not achieve a consistent standard across the study area and ranges between the 1Y and 10Y ARI event. Generally the minor drainage systems in urban areas are designed to have a minimum capacity of the order of the 20% AEP event.

The major drainage system (road easements and trunk drainage elements) vary greatly in capacity. The Eastern Channel can contain the 1% AEP event whilst the Malakoff Tunnel is unable to prevent widespread inundation in Malakoff Street and surrounds for events greater than the 20% AEP.

Based on our review of available rainfall and flood records Marrickville has not experienced a significant (> 2Y ARI) flood event for a long period of time, probably since the mid 1970's. The largest event in the last 50 years appears to have occurred in 1966. Future events will alert the population of Marrickville to their flood liability, particularly those residents in some of the most flood liable land near Sydenham Road, Malakoff Street and those businesses within the MIA.

Future improvements to the level of inundation will come primarily from mitigation works. Policy is unlikely to mitigate current hazard/risk although it could be effective in ensuring that current hazard/risk levels are not exacerbated. Suitable mitigation works will:

- enhance catchment storage; and/or
- enhance the conveyance of both the minor and major drainage systems.

Given the general lack of open space on which communal retarding basins (enhancing catchment storage) could be located, conveyance solutions will be the main priority. Where possible opportunities to incorporate basins into parklands should nevertheless be pursued.

Possible conveyance enhancements that could be considered at the next stage of the floodplain risk management process (Floodplain Management Study), are as follows:

- Identifying and then replacing structures which limit the flow capacity of the channels;
- Examine the possibility of updating the drainage system (major and minor) such that 10% AEP flooding at least is contained within defined flow paths. Updating could

proceed as asset replacement programmes and incidental failures occur if a program of works had been identified;

- Converting parks, as appropriate into retarding basins. It is noteworthy that the main constraints will be the available volume in the basins relative to design rainfall given the need to limit danger in the event of dam failure and also to avoid backwatering into upstream properties. Marrickville Oval provides an excellent example of what can be done with a playing field. However it is noted that for its limited upstream catchment (52 ha) and relatively large capacity, the Oval achieves significant flood protection for downstream residents only up to the 10% AEP event. The cost to construct basins is large, there are significant maintenance issues and construction of a basin may increase the flood risk to downstream residents in the event of a wall failure or overtopping; and
- Examining the opportunities for widening channels/conduits and ensuring future zoning decisions do not exacerbate the problem or restrict the means of mitigating flooding.

7. PUBLIC EXHIBITION

Marrickville Council resolved to place the Draft Marrickville Valley Flood Study on public exhibition at their February 2012 meeting. The flood study was placed on public exhibition during May and June 2012. There were also two workshops held where the community was encouraged to attend and ask questions. Council received three written submissions that were generally supportive of the flood study. The submissions expressed concern over flooding in the vicinity of Livingstone Road and Brereton Avenue, and also in the vicinity of Malakoff Street and Northcote Street. Both of these areas will be considered as part of the Marrickville Valley Floodplain Risk Management Study and Plan. The Marrickville Valley Flood Study was subsequently adopted by Council in April 2013.

8. ACKNOWLEDGEMENTS

WMAwater wish to acknowledge the assistance of Marrickville Council staff in carrying out this study as well as the NSW Government, Sydney Water and the residents of the Marrickville Valley. This study was jointly funded by Council, the NSW Government and Sydney Water.

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REVIEW OF PREVIOUS STUDIES

APPENDIX A

A1: MARRICKVILLE VALLEY SWC 66 CAPACITY ASSESSMENT, SYDNEY WATER 1995 (REFERENCE 1)

This report undertook an audit of the drainage infrastructure within the Marrickville Valley. A range of manual calculations was used to assess the flow capacity of various drainage elements and this was presented via both tabular and graphical means. Overall the capacity of most drainage elements was found to be relatively low, within a range between the 2Y ARI event and the 10Y ARI event, but averaging around the 5Y ARI event. The usefulness of the capacity assessment is limited in that it is not based on modelling but on Manning's equation calculations. This excludes the significant backwatering effect (downstream levels impact on upstream levels) likely in such a system and thus actual flow rates for trunk drainage elements may vary significantly from estimates provided in the report. No model calibration was possible.

This study provides excellent detail on trunk drainage elements (adopted for use in this study and shown in Appendix D) including location, alignment, typical sections, culvert details, channel dimensions and slopes. Trunk drainage elements were also provided via GIS files although the origin of these is unknown.

A2: MARRICKVILLE INDUSTRIAL AREA DRAINAGE STUDY, GHD 2002 (REFERENCE 2)

Introduction

This drainage study used the DRAINS hydrologic model in order to examine the existing drainage system capacity and to assess possible flood mitigation works. A considerable amount of data was obtained from both Council and Sydney Water and this is described further in Appendix A. The identified drainage issues were

- Prior to this study Council surveyed 31 trapped low points within the MIA catchment. The survey included levelling of pits, kerbs, road centrelines, property and floor levels. Hydraulic modelling was carried out and this identified that some of the surveyed low points did not result in flooding of property whilst in other areas new trapped low points were identified. A list of additional trapped low points is provided in Table 2.3 of report;
- Council have in some cases augmented/upgraded drainage in order to deal with known issues. A list of these is given in the report but they typically involve pipe sizes of 600 mm diameter or smaller.

Subcatchments areas for the DRAINS hydrologic model were described for all pits based on 2 metre contours along with assumptions that roads are two way cross-fall and that house fences are impermeable barriers. A value of 3 was used to define antecedent moisture condition and soil type. This implies that 12.5 mm of rainfall would typically have fallen in the 5 days prior to the storms being modelled (the large impervious fraction in most catchments largely negates the importance of this factor). 1 mm and 5 mm depression storage was adopted for paved and grassed areas respectively. Further details of the DRAINS model are provided in Appendix A.

The Cooks River tailwater level adopted was 0.625 mAHD (Mean High Water Spring (MHWS) which occurs approximately 40 times a year). No calibration data was available, therefore the Probabilistic Rational Method (PRM) was adopted for verification purposes [it is noteworthy that the PRM is not suitable for such urbanised catchments]. The 20% AEP event was modelled for the catchment upstream of the Malakoff Tunnel entrance and a comparison of results downstream of Marrickville Oval was satisfactory.

Data Supplied By Council

- Council surveyed the drainage system prior to this study;
- Sydney Water and Council assets were identified;
- Survey of pit location, type, inlet dimension, depth to invert of incoming and outgoing pipe and dimensions of incoming and outgoing pipes; and
- Council supplied the following– pit data sheets, drawings (in dwg file format) of trapped low points, land use data, electronic copy of Sydney Water assets, drawing files of Council and Sydney Water drainage assets, base GIS data including street names and CCTV inspection reports.

Data Supplied by Sydney Water (Reference 1)

- Marrickville Valley SWC 66 Compendium of Data;
- Contingency Plan DPS1 1999;
- Contingency Plan DPS2 1999;
- MV SWC 66 Capacity Assessment 1995;
- DPS1 Risk of Pump Failure – Flood Investigation – July 1997; and
- DPS2 Risk of Pump Failure – Flood Investigation – June 1998.

During the study all pits were surveyed in order to determine levels and also further pits outside of the original Council Database were sought and many identified and then surveyed. The report includes works as executed drawings from Sydney Water which cover approximately 90% of Sydney Water assets in the study area

Hydrological Modelling

DRAINS V2002.02 was used. There were multiple problems with DRAINS during the course of the study the most significant being:

- DRAINS would not export box culvert data to Mapinfo (GIS software);
- DRAINS corrupted co-ordinates imported from DWG files;
- DRAINS has not merged correctly where subcatchment CC joins onto subcatchment EC1W (graphical issue only); and
- A problem with catchment symbols when merging data bases.

Mapinfo was used to derive catchment characteristics to inform DRAINS and a naming convention was used (discussed on page 16 of the report). In this convention Council pit numbers are used in conjunction with subcatchment abbreviations. For example for subcatchment EC2 and pit 12 the following apply:

- Pipe d/s of EC2 12 named pEC2 12;
- Catchment flowing to pit 12 in EC2 named cEC2 12;
- Overflows from pit 12 in EC2 named oEC2 12;
- Drainage path to pit 12 named dEC2 12; and
- All channels d/s of a node were named eg. chEC2 12. For extra pits used (dummy presumably) a letter suffix has been used.

Comprehensive hydrological modelling was carried out for the majority of the study area. The hydrological assumptions are:

- Paved and grassed constant times of, respectively, 3 and 5 minutes, were assumed for pits draining directly to the kerb;
- Additional 3 minutes added if the property must drain via another property;
- No allowance was made for ponding at individual properties;
- Gutter system used to convey flow along roads;
- Gutter flow factor of 10.8 used representing a typical RTA, SA kerb with 3% road cross fall; and
- The residential area percentage was 70% paved, commercial 90%, industrial 95%, open space 5% and special use 40%.

Six GIS layers were used to present and hence review the DRAINS results:

1. Existing data input and DRAINS results,
2. Option 1,
3. Option 1 and 2,
4. Options 1, 2, 3,
5. As above plus pipe, pit and overflow mitigation works,
6. Contextualising GIS layers.

More information was provided on the GIS files in Appendix A of the report. The general DRAINS modelling assumptions are listed in Appendix C of the report.

Several different pit types were used such as, 500 mm lintel type, Hornsby grated inlet/lintel, grated inlet only. Sag pits were assumed as 50% blocked and on-grade 20% blocked.

Trapped low points were initially modelled using DRAINS but this caused stability issues. Ultimately they were modelled as detention basins with stage-storage relationships based on available survey.

A3: EASTERN CHANNEL EAST (ECE) SUBCATCHMENT MANAGEMENT PLAN, VOLUME 2 FLOOD STUDY, GOLDER 2010 (REFERENCE 3)

This study focused on the ECE subcatchment [north-eastern corner of the Marrickville Valley, see Figure 2 of this report]. It documents the flood study process that has been undertaken for the ECE subcatchment which includes the collection of data, community consultation, model build and calibration, design flood modelling and hazard and hydraulic categorisation.

The ECE subcatchment has an area of approximately 131 ha (95 ha or 75% of the catchment area is assessed as being impervious) and land use is distributed in industrial, commercial and residential areas. The catchment was divided into 3 subcatchments; namely, northern, southern and eastern.

Notable rainfall events are mentioned in the report and the event of 23 March 1966 is the most significant (approximately 1% AEP intensity but the storm duration was not reported). Five other storms are highlighted with an approximate ARI of 2Y or lower. Over the last 15 years no significant rainfall events (higher than 2Y ARI) have been observed.

The DRAINS package was used for hydrological modelling and the TUFLOW package for hydraulic modelling purposes. Further details are provided in Appendix A.

Community consultation carried out during the course of the study identified a number of drainage "hotspots" and these are also described in Appendix A.

Table A1 indicates the number of land parcels inundated for each design flood event:

Table A1: ECE - Number of Land Parcels Inundated – Reference 3

	2Y ARI	20% AEP	10% AEP	1% AEP
Number of land Parcels Inundated	321	418	603	925

Hydrologic Modelling

In order to convert the rainfall event into stormwater runoff, DRAINS, using the Extended Rational Method was used. Flows were obtained for 198 pits within the catchment. Each pit was assumed to be the outlet of each subcatchment. Other DRAINS modelling details include the following:

- Runoff coefficients utilised in the model were 0.58 and 0.9 for pervious and impervious areas respectively;
- Several assumptions were made using ALS data due to missing information on pit and pipe invert levels;
- Calibration of the hydrologic model was achieved indirectly by incorporating the TUFLOW hydraulic model;
- The DRAINS model was run for a variety of storm durations and the critical duration determined; and
- The modelling was undertaken with and without pit blockages to assess the impact on peak flows. The results of the hydrologic modelling were compared to the SWC66 Capacity Assessment (Reference 1) and the results considered satisfactory.

Hydraulic Modelling

A TUFLOW 1D/2D hydraulic model was set up in order to model 2D overland flow as well as channelised flow and structures. A model cell size of 2 m was utilised (i.e. 2 m x 2 m) resulting in a model grid of 840 by 775 cells. The computational time step of 0.1 seconds was adopted. Other details of the hydraulic modelling include:

- The pit and pipe network was modelled in 1D whilst detention storage and overland flow was modelled in 2D;
- Discharges from DRAINS were applied into TUFLOW at the pit locations;
- An assumption of normal depth along the Eastern Channel and available flow paths was used as the downstream boundary of the model;
- Design flood behaviour was carried out for the 2Y ARI, 20%, 10% and 1% AEP events;
- Critical storm durations were found to be the 45 minute and 3 hour durations; and
- Varying pit loss coefficients, culvert losses and contraction coefficients were included in the model.

No stream gauging stations exist within or downstream of the catchment. However model calibration data were derived from community observations of two rainfall events, namely 13th February and 14 March 2009.

Community consultation carried out during the course of the study identified a number of drainage “hotspots” and these are described below.

Northern subcatchment:

- Corner of Edgeware Road and Camden Street (along TAFE car park);
- Sarah Street between Marian Street and Simmons Street;
- Margaret Street;
- Corner of Edgeware Road and Alice Street;
- Between 12 and 14 Llewellyn Street (stormwater channel);
- From 228 Edgeware Road to corner of Alice Street; and
- Corner of Edinburgh and Murray Streets.

Eastern catchment:

- Well Street, Darley Street and Lord Street;
- Corner of John Street and Lord Street; and
- 108 Wells Street.

Southern catchment:

- Campbell Street from Church Street to Bedwin Road and lower end of Brown Street, Florence Street and Hutchinson Street; and
- Camdenville Oval at end of May Street.

Some of the key results in specific areas are summarised below:

- **Goodsell Street and May Lane**: No formal stormwater drainage exists in this area. The contributing catchment to this area is not significant, although runoff from the Princes Highway is conveyed to the top of Goodsell Street. Flood depths obtained range between 20 cm and 30 cm for the 2Y ARI and 1% AEP respectively. Velocities range from 1.2 to 1.5 m/s.
- **John Street and Lord Street**: The modelled flood depth at the corner of John Street and Lord Street ranges between 20 cm and 40 cm in the 2Y ARI and 1% AEP respectively. The provisional flood hazard in this area is low up to the 1% AEP.
- **Lower Lord Street, Darley Street and Wells Street**: There have been reported issues of the stormwater drainage capacity being exceeded within Lord Street. The spacing of pit inlets along Lord Street is more than 60 m which limits the amount of stormwater runoff that can enter the drainage system. The modelled flood depth ranges between 25 cm to 40 cm and overland flow velocities can reach 2 m/s in the 1% AEP. The flood hazard categorisation is low for the 2Y ARI and 20% AEP but high for the 10% and 1% AEP events.
- **Corner of Railway Parade and Edgeware Road**: This area is reported to experience regular issues with respect to flooding. The modelled flood depth underneath Bedwin Road ranges from 60 cm in the 2Y ARI event to 85 cm in the 1% AEP event. Overland velocities are low implying the stormwater ponds at this location.
- **Intersection of Campbell Street, Hutchinson Street, Brown Street and May Street**:

No drainage infrastructure exists at this natural low point. A series of pits and inlets capture the overland stormwater flow and conveys it via a 1200 mm pipe underlying Bedwin Road railway bridge. The modelled flood depth ranges from 40 cm for the 2Y ARI event to 90 cm for the 1% AEP event.

- **Overland flow Upper Brown Street and Florence Street**: No drainage infrastructure exists in Brown Street or Florence Street. Runoff is conveyed by roads and footpaths. At Brown Street the modelled flood depth ranges from 5 to 10 cm in the 2Y ARI event to 10 to 15 cm in the 1% AEP. On Florence Street the modelled flood depth ranges from 10 cm in the 2Y ARI event to 20 cm in the 1% AEP event.
- **Edgeware Road (alongside TAFE carpark)**: Overland flow along this location has been regularly reported as a drainage problem. The modelled flood depth ranges from 5 cm in the 2Y ARI to 15 cm in the 1% AEP.
- **Camden Street (adjacent TAFE carpark)**: The modelled flood depth ranges from 35 cm in the 2Y ARI event to 80 cm in the 1% AEP event.
- **Corner of Edgeware Road and Alice Street**: This location is a reported hotspot with respect to ponded stormwater. Analysis suggests that pit capacity is the main issue since the main culvert does not flow at full capacity. The modelled flood depth at this intersection ranges from 30 cm in the 2Y ARI event to 60 cm in the 1% AEP event.
- **Alice Street**: Modelled results suggest that in the 1% AEP event the flood depth might be sufficient enough to enter the car park at the Alice Street driveway.
- **Marrickville Metro**: The modelled flood depth ranges from 10 cm in the 2Y ARI to 40 cm in the 1% AEP resulting in the dock area, which is below street level, possibly becoming inundated.
- **Edinburgh and Murray Streets**: The modelled flood depth ranges from 40 cm in the 2Y ARI event to 1.1 m in the 1% AEP event. The flood hazard is low in the 2Y ARI and 20% AEP events, intermediate hazard for the 10% AEP event and high hazard for the 1% AEP event.
- **Pemell Street**: The modelled flood depth ranges from 45 cm in the 2Y ARI event to 1.1 m in the 1% AEP. The flood hazard is low for the 2Y ARI, 20% and 10% AEP events but high hazard for the 1% AEP.
- **Fulham Street**: The modelled flood depth ranges from 25 cm in the 2Y ARI event to 1.5 m in the 1% AEP event. The flood hazard is low for the 2Y ARI, 20% and 10% AEP events but high hazard for the 1% AEP.

A4: SYDENHAM STORMWATER STORAGE-PIT POLLUTANT TRAP STUDY WILLING & PARTNERS, 1993 (REFERENCE 4)

The objective of the study was to assess the potential to improve the stormwater pollutant trap potential in the pit whilst maintaining the same level of flood control in the area.

Hydrological/Hydraulic modelling

A hydrological model (RAFTS-XP) was established in order to estimate inflow hydrographs for the Eastern Channel and the MIA arriving into the pit. Rainfall losses were estimated based on previous modelling work carried out elsewhere in the Sydney region. Flow hydrographs from 1Y ARI to 1% AEP design events were estimated using a 2 hour temporal pattern (assumed as the critical storm duration). Hydraulic modelling was carried out using EXTRAN-XP software.

The report found that improvement of flood control could be maximised by:

- Increasing the pit storage capacity (excavating floor level by a depth of 1.8m);
- Commencing pumping earlier, and;
- Increasing the maximum pumping rate to allow the EC to be at full capacity during the storm event.

Water Quality modelling

The objectives of the water quality modelling were to estimate the stormwater load pollutant arriving from the EC into the Cooks River. The water quality modelling was carried out using the AQUALM-XP modelling package. The standards to achieve these objectives were based in accordance to the environmental planning authority recommendations together with the target pollutant levels for the lower Cooks River zone of the Cooks River Total Catchment Management Action Plan.

A limited amount of water quality data was available (volume of sediment cleaned and aqueous samples from the pit taken in 1992).

Pit operation for optimal flood control and water quality

A number of options were examined to increase the capturing of debris and water pollutants in the pit as well as for the EC.

The recommended options for the MIA catchment comprised:

- Litter cages at the main drainage outlet to the pit, and;
- A 9.8 ML sedimentation pond excavated into the floor of the pit, with a daily exchange volume of 4.5 ML.

Chemical dosing was also recommended. If the chemical dosing was rejected as an aid to sedimentation the recommended actions would consist of:

- Diversion of up to 3 ML/day for the EC into the pit;
- Installation of a major GPT with a frangible trash rack upstream in the EC, and;
- Installation of a litter cage on the outlet to the EC diversion.

No increase in pumping capacity was adopted due to the small benefits that would arise from this approach. Monitoring during a period of 12 months was also recommended. This would assess sediment particle distribution and pollutant load concentrations.

A5: MARRICKVILLE OVAL DETENTION BASIN STUDY, SMEC, 2010 (REFERENCE 5)

Marrickville Oval was formalised as a retarding basin in approximately 1996. The construction works comprise an earthen bund and a concrete reinforced masonry wall which surrounds the oval. The earthen bund has a spillway at 14.75 mAHD. Overflows from the spillway are conveyed towards the lowest point on Livingstone Road and then to the stormwater network. 25 m of the masonry wall have a crest level of 15.43 mAHD, whilst the remaining crest wall is at 15.6 mAHD. The capacity at the spillway crest is approximately 10 ML (approximately a 1Y ARI rainfall event). The total area draining into the basin is 52 hectares.

A DRAINS model was established to estimate hydrograph inflows into the trunk drainage system which was modelled using the MIKE-11 hydraulic model. The 1% AEP and PMF design events were computed for all storm durations. The critical storm durations were 1 hour in the 1% AEP event and in 20 minutes the PMF event.

A summary of the results obtained is presented in the image below (sourced from Reference 5).

Location	Section ID / Chainage (m)[b]	Existing overland flow without dam break 1% AEP (m^3/s)	PMF (m^3/s)
Marrickville Oval	Main 80.0	16.0	74.7
Livingstone Road[a]	Main 248.0	11.6	80.3
Brereton Avenue	Main 379.7	13.4	78.1
Petersham Road	Main 443.9	16.7	97.9
Centennial Street	Main 649.5	19.5	103.4
Malakoff Street	Main 955.6	31.9	128.0

[a] Allows for an estimated 50% flow split at corner of Livingstone Road and Sydenham Road.
[b] Refer to Appendix D for plots of cross sections presented in this table. Dam wall is located at Chainage 100.

The results also showed a limited capacity of the drainage under the oval. In the 1% AEP the peak flow is 3.6 m^3/s and 3.94 m^3/s in the PMF. Results indicated that the brick drain channel downstream of the oval has a capacity of between 4.0 and 9.2 m^3/s in the 1% AEP and 4.8 to 11.6 m^3/s in the PMF.

Dam Break modelling

The dam break scenario was developed in accordance to the worst case Flood Consequence Category (FCC) using the MIKE-11 hydraulic model. ALS data was included as input into the model to develop 1D cross sections extending from the retarding basin outlet to Victoria Street. Two events were modelled namely, the 1% AEP and the PMF.

In the case of the 1% AEP event, the breach level was set at the spillway level to indicate a conservative worst-case scenario for dam failure. The breach in the dam occurs at a time just before the inflow hydrograph peak arrives in the dam. In the PMF event, the basin water level at which the breach starts was assumed to be 15.428 mAHD, which is the level at which water starts overtopping the lowest level of the wall. The study assumed a breach width of 10 m and 1:1 side slopes in the wall. A summary of the results is shown in Table A2 (sourced from Reference 5).

Table A2: Flows and Water Levels at Marrickville Oval

Flood Event	Peak Basin Inflow (m^3/s)	Peak Basin Outflow (m^3/s)	Max Water Surface Level (mAHD)	Level Description	Time to Basin Inflow Peak (minutes)
1% AEP - No dam break	16.0	1.4	15.08	0.33 m above spillway level[1]	30.5
1% AEP - Dam break	16.0	23.3	15.03	0.28 m above spillway level[1]	30.5
PMF – No dam break	74.7	24.7	15.85	0.42 m above dam wall[2]	16.0
PMF – Dam break	74.7	63.9	15.80	0.37 m above dam wall[2]	16.0

[1] Spillway level is at RL 14.75 mAHD

[2] Lowest level of masonry wall is at RL 15.43 mAHD

The report concluded that the impact of dam failure would be classified as a category “High C” (according to flood consequence categorisation) in the 1% AEP and PMF events.

This report refers to the Malakoff Street Drainage Study (Reference 6) which provides an analysis of the former drainage system prior to upgrading in 1996 and construction of the Marrickville Oval retarding basin. In Reference 6, an ILSAX model was used and the critical storm duration was found to be 60 minutes. The study examined mitigation schemes involving Marrickville Oval and Marrickville High School. It is noted that extended works as recommended in the study were not implemented. The study states that:

> “*The majority of minor flooding problems within the catchment are due to the lack of hydraulic capacity in the underground pipeline, box culvert and open channel sections together with a kerb inlet pits, particularly in the Livingstone Road and Malakoff Street areas. Major flooding of residential properties between Livingstone Road and Malakoff Street occurs on a relatively regular basis due to lack of designated overland flowpath to cater for water surcharging from the underground pipeline system”. (Reference 6)*

Reference 6 was not reviewed as part of the present study.

A6: COOKS RIVER FLOOD STUDY, MWH PARSONS BRINCKERHOFF, 2009 (REFERENCE 7)

The Cooks River has a catchment area of approximately 102 km^2, covers portions of thirteen local government areas and discharges to Botany Bay underneath General Holmes Drive at Tempe. In the study a WBNM hydrologic model was used in conjunction with a TUFLOW hydraulic model to determine design flood levels.

The adopted hydrologic loss parameters in this study were 10 mm and 1.5 mm of initial loss and 2.5 mm/h and 0.0 mm/h for continuing loss in pervious and impervious areas respectively.

Climate change scenarios were modelled for the 1% AEP for an increase in rainfall intensity of 10%, 20% and 30% and sea level rise values of +0.18m, +0.55m and +0.91m.

Floodplain roughness values adopted for the TUFLOW model are provided in Table A3.

Table A3: Bed Resistance Values used in the Cooks River Flood Study (Reference 7)

Land use	Manning’s ‘n’
Roads	0.015
Airport	0.020
Open space / Parkland	0.040
Dense vegetation	0.070
Creek / Waterway	0.030
Botany Bay	0.020
Residential	0.300
Inundation	0.300

A downstream boundary condition of 1.1 mAHD was adopted as a tailwater level condition in conjunction with design flood events in the Cooks River. In addition the 1% AEP design level in Botany Bay (1.45m AHD) was modelled jointly with a 2Y ARI event.

The results of this study indicate that upstream of the Princes Highway water levels are dominated by rainfall induced flooding, whereas downstream of the Princes Highway water levels are dominated by ocean levels in Botany Bay. The peak flood levels determined at the Princes Highway are provided in Table A4.

Table A4: Peak Flood Levels at Princes Highway (Reference 7)

Design Event	Level (mAHD)
2Y ARI	1.49
5% AEP	1.89
1% AEP	2.16
1% AEP Climate Change	2.55
PMF	3.46

COMMUNITY CONSULTATION APPENDIX B

MARRICKVILLE VALLEY FLOOD STUDY
COMMUNITY CONSULTATION

A community consultation programme was implemented with Council's assistance. The programme involved a number of steps and these were:

- A media release advising Marrickville residents that a Flood Study was to be carried out, what the goals of the Flood Study were and indicating that those with any interest/information might contact the consultant and/or Council in order to communicate information; and
- A questionnaire was then issued to certain residences that were, based on Council experience as well as preliminary modelling carried out by WMAwater, likely to be impacted by drainage issues. A total of 500 questionnaires were mailed out to individual residences. Of the 500 issued 5% have been returned. Those questionnaires returned have been compiled into a database so that the information contained within them can be better utilised in reporting as well as model calibration/validation exercises.

Please note that for completeness's sake a copy of the media release and questionnaire are attached. Photos received from residents are included catalogued on the basis of event. Figure 1 indicates the location of residents who responded to the mail out. Charts summarising various features of the responses received from the community are shown in Figure 2.

Figure 3 indicates the respondents who have experienced flooding. The awareness of historical flooding events amongst respondents was low with only a few residents identifying December 2007 and June 2008 events. 14 of the 26 respondents claimed to have never been impacted by any flooding issues, with a majority of those responses originating from the Marrickville Industrial Area.

Major findings from the community consultation campaign are as follows:

- The previously identified storm events (December 2007, June 2008, June 2010 and July 2010) were remembered by a number of residents. 4 of the 26 respondents experienced flooding in one form or another;
- Inundation of properties and damage to properties in Marrickville is a major issue for those residents who responded, also identified there was an issue with the drainage system being unable to cope;
- 35% of the 14 respondents (who have experienced flooding) and only 1% of the total group surveyed) have actually had flood waters enter their home (water through the

house etc);

- Rainfall events which precipitate drainage issues (i.e. inundation of private property) occur relatively often (once every two years at least);

Community responses were accompanied by a number of photographs (see attached) and in some cases observed flood depths at known points. All of this information has been collated into a GIS layer (see Figure 3) so that it can be:

- Utilised by Council as needs be;
- Presented as a product of this study; and
- Utilised for calibration/validation purposes.

MARRICKVILLE VALLEY FLOOD STUDY

QUESTIONNAIRE

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

WMAwater is carrying out the study for Council and would like information about your experiences of flooding. Please return the completed questionnaire before 31/07/2010 by:

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

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

Your Name: ______________________________________ Tel No: ____________________
Property Address: ______________________________________ E-Mail: ____________________

☐ Residential Property ☐ Non-Residential Property

Flood Information

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

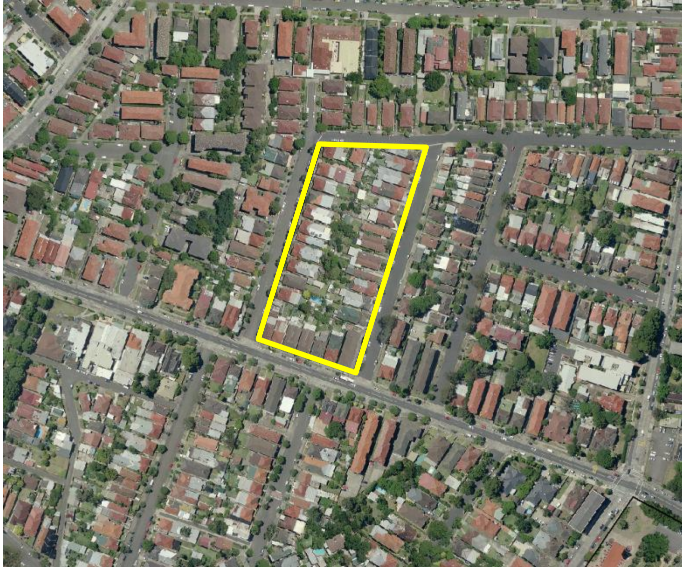
Have you experienced any of the following flood events?

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

What other floods have you experienced? ____

REVIEW OF IMPERVIOUSNESS APPENDIX C

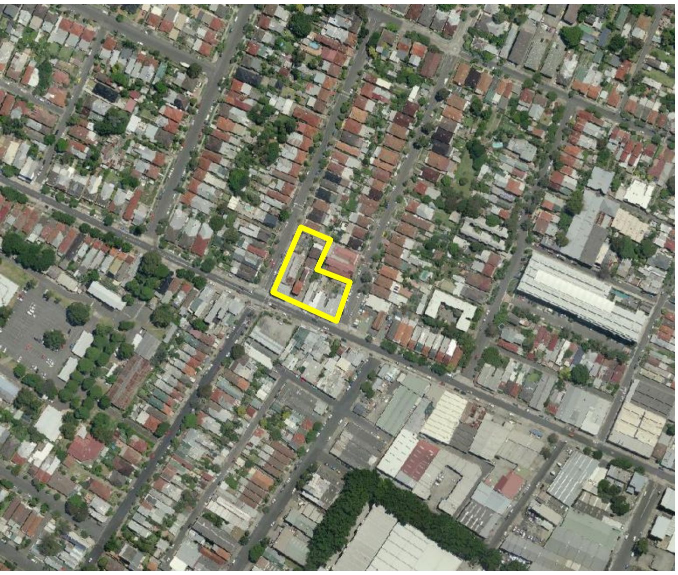
1. 2a - Residential



2. 2b - Residential

3. 2c - Residential

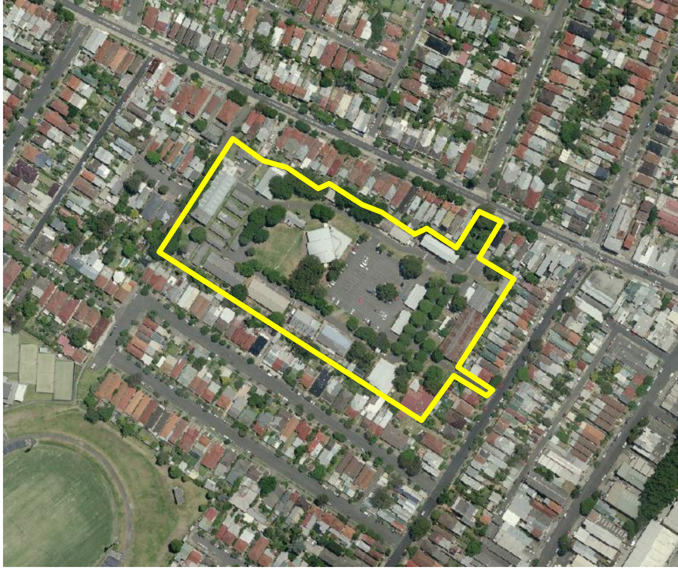
4. 3a – General Business



5. 3b – Neighbourhood Business

6. 4a – General Industrial

7. 4a – Light Industrial



8. 5a - Special Uses – Theatre – 80%

9. 5a - Special Uses – Retarding Basin - 100%

10. 5a - Special Uses – School - 60%

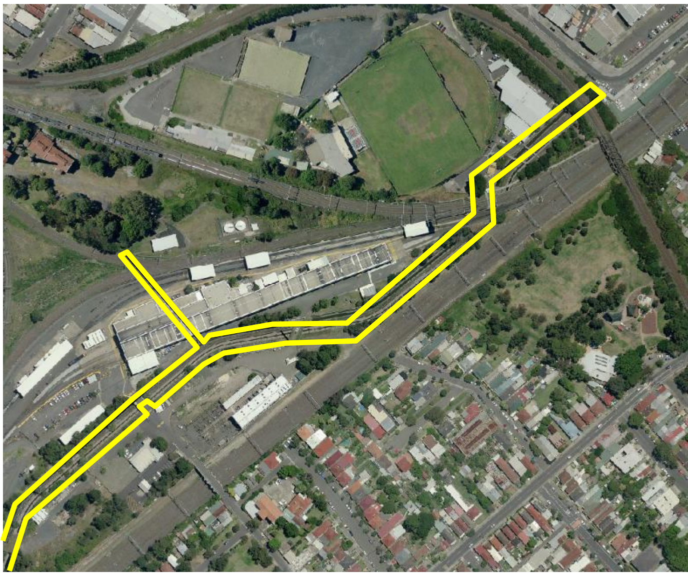
11. 5a - Special Uses – Pumping Station – 20%

12. 5b –Special Uses(Railways)

13. 6a – Open Space

14. 6b – Private Open Space

15. 9a – Reservation (Local Open Space)



16. 9b – Reservation (Special Uses)

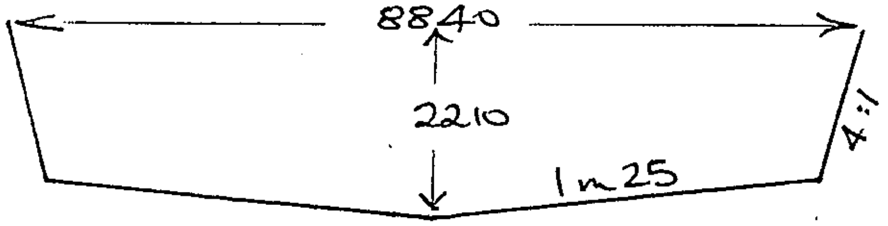
17. 9c – Reservation (Arterial Road and Arterial Road Widening)

18. 9d – Reservation (Local Road and Local Road Widening)

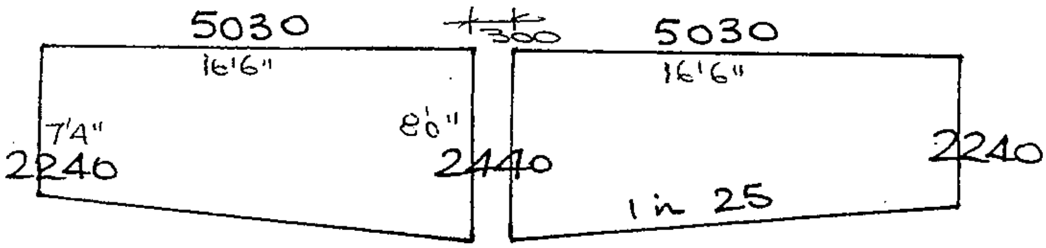
CHANNEL CROSS SECTIONS

APPENDIX D

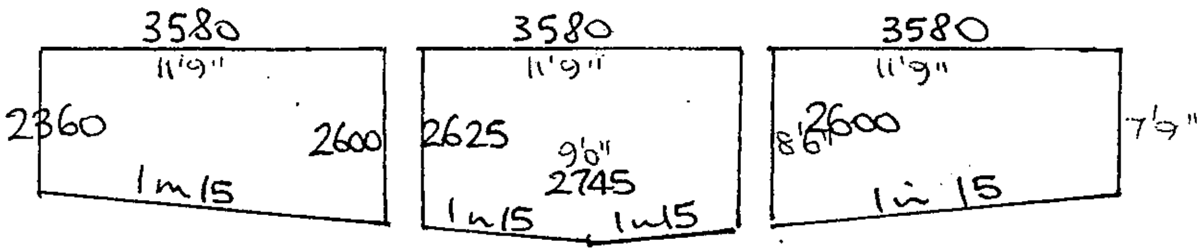
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



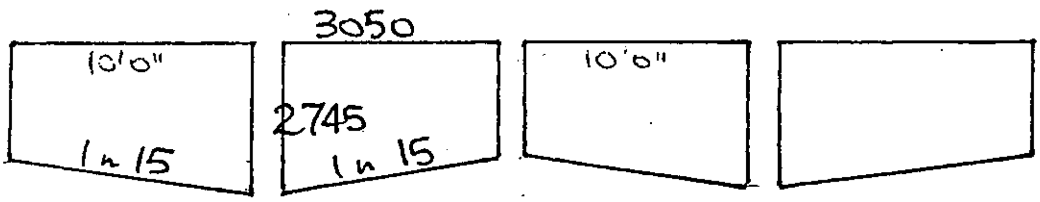
YZ-XZA, UZA-TZA, RZC-RZA



XZ-VZ



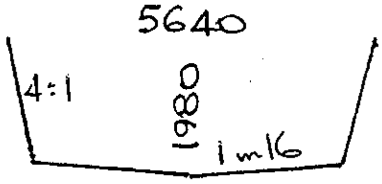
TZ-SZ

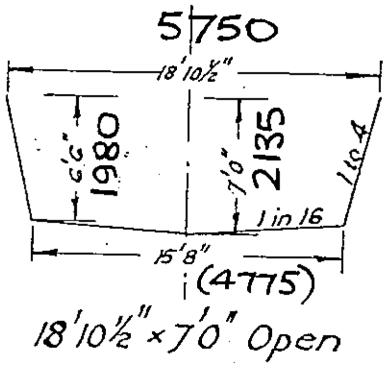


RZ-QZB

MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

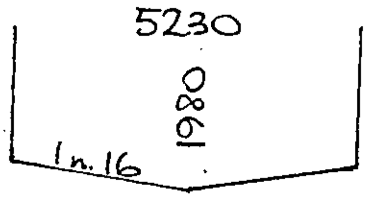
(Looking downstream)

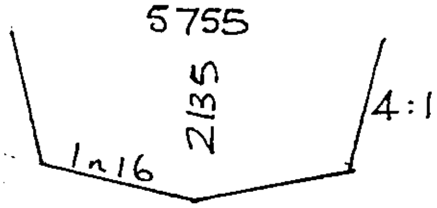




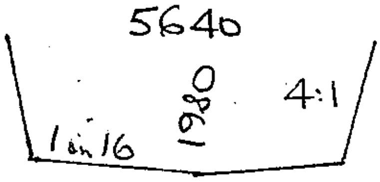
QZ-PZA

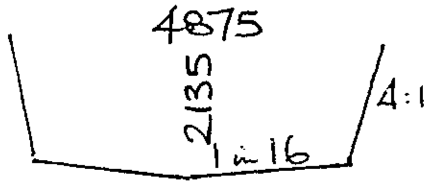
NZ-MZ





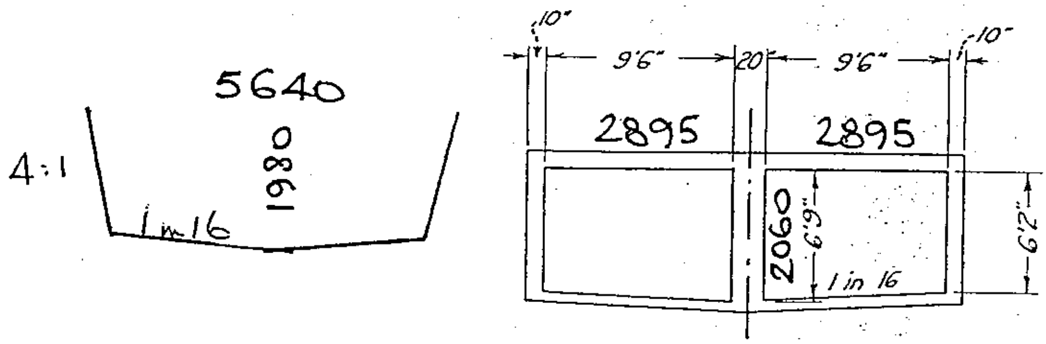
PZ-NZA



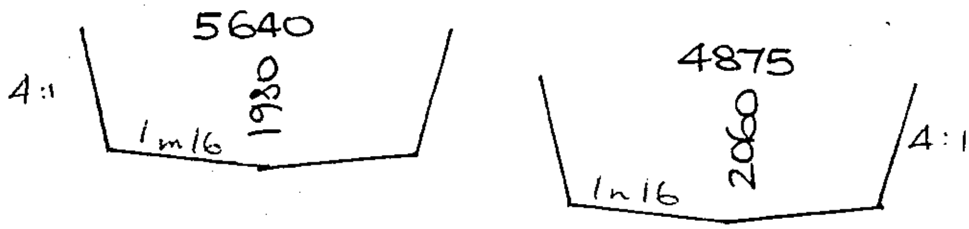


MZ-KZ

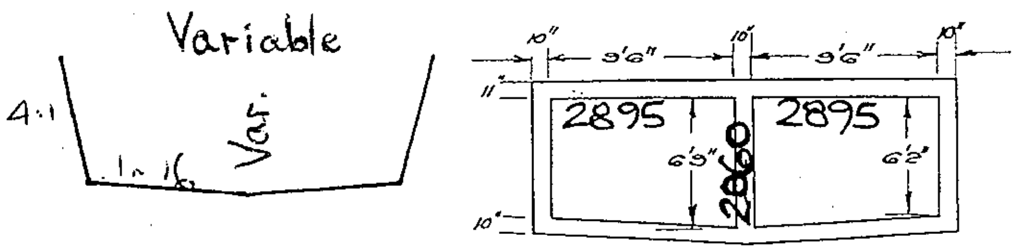
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



KZ-JZ
HZ-GZ

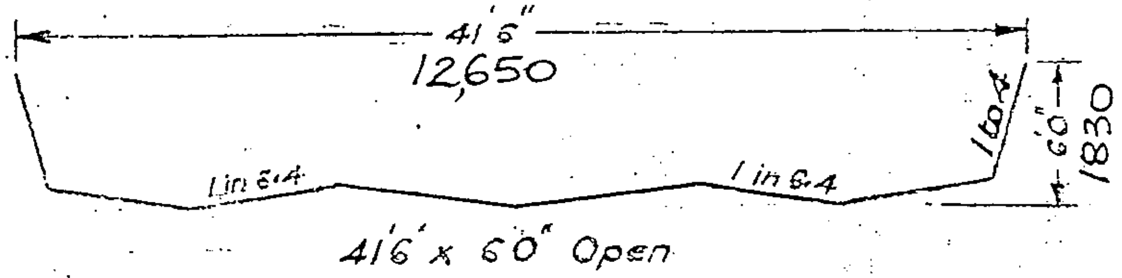


JZ-HZ

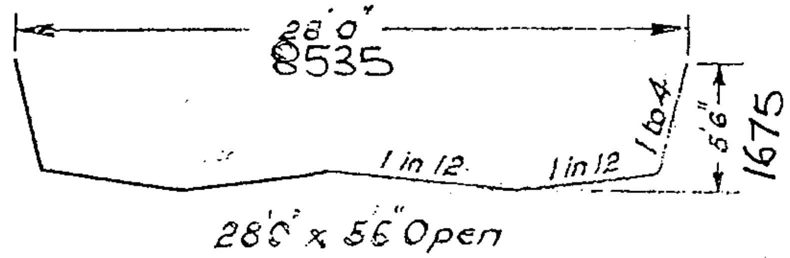


GZ-A2

MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

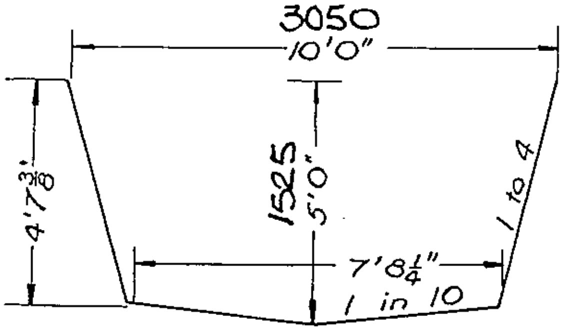


AA2-BA

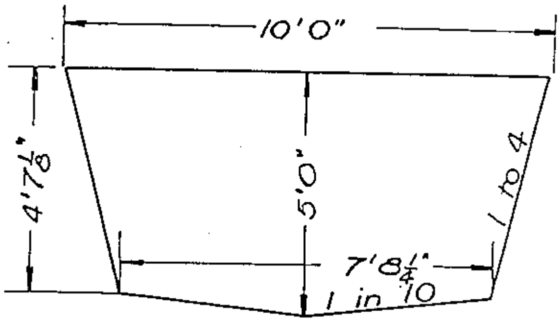


BA1-C

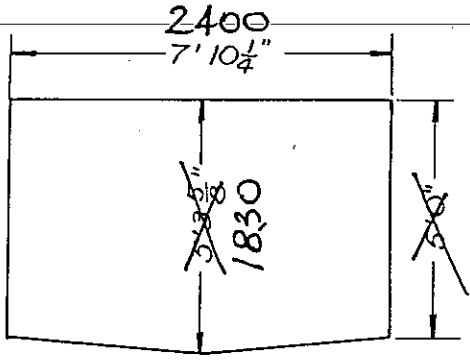
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



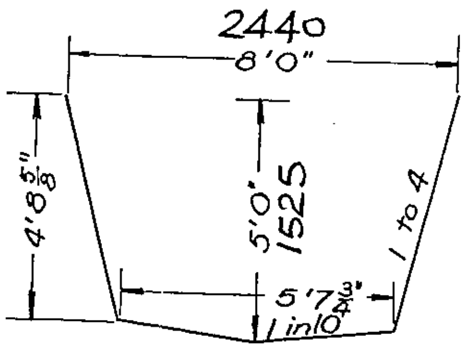
C-D E-F



D-E

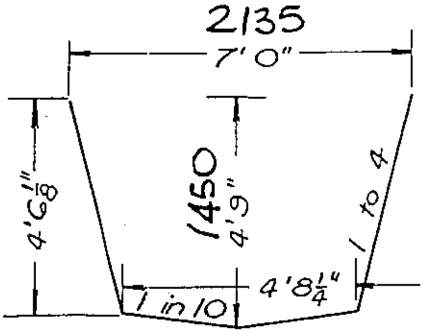


F-G

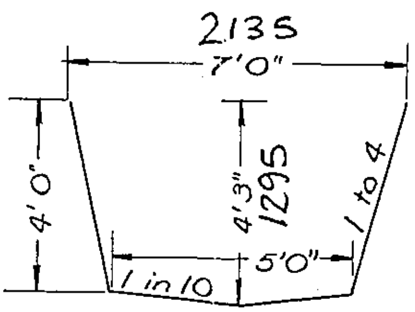


G-H

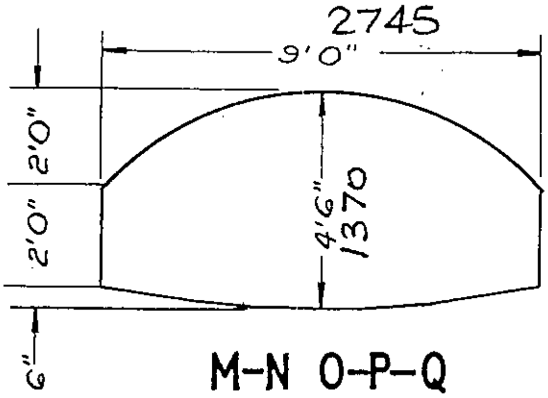
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



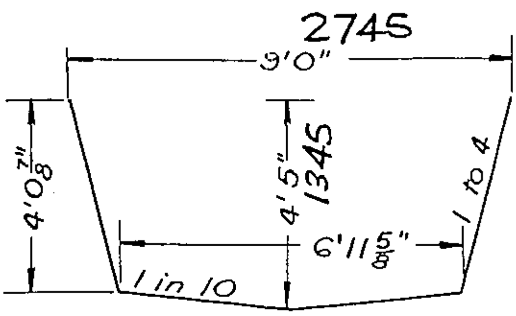
H-J-K-L



I-M



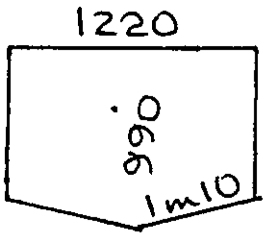
M-N O-P-Q



N-O

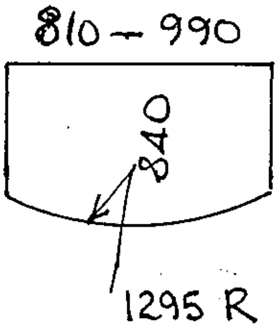
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

BRANCH EB

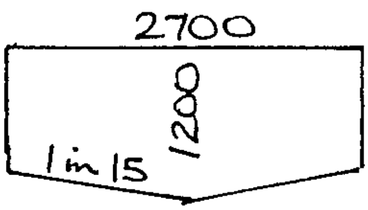


UZ-UZ1

GROVE ST BRANCH



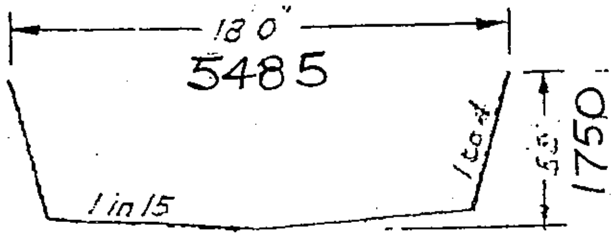
S(A)-S1
S1-S2
S3-S5



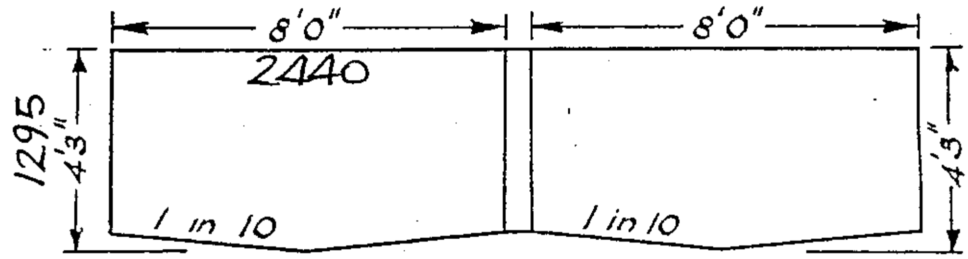
S2-S3

MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

EASTERN CHANNEL AMP

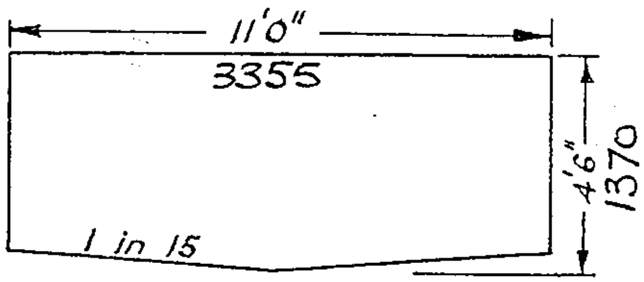


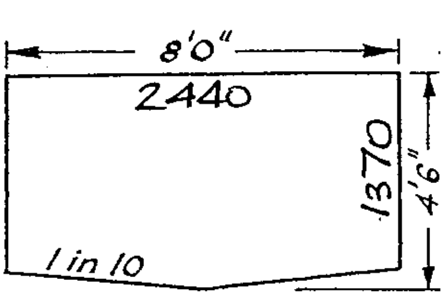
BB-BB1



BB1A-BB2

BB2-BB3

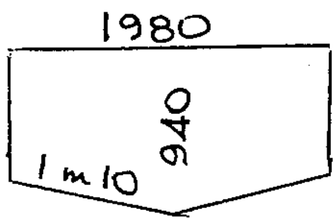




BB3-BB3C

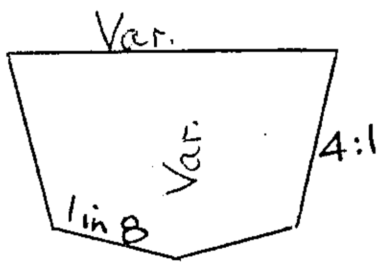
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

MURRAY ST BRANCH

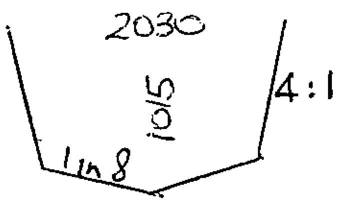


B1A-B1-B2

EDGEWARE RD BRANCH



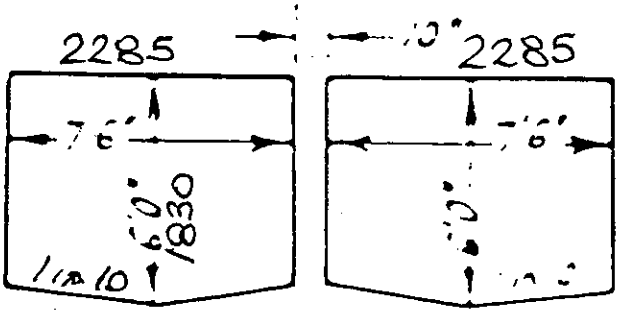
C-C7
C8-C9



C7-C8

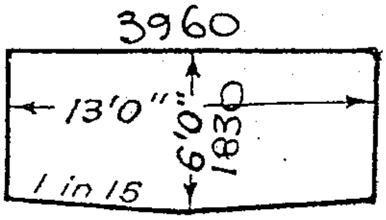
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

EASTERN CHANNEL AMP

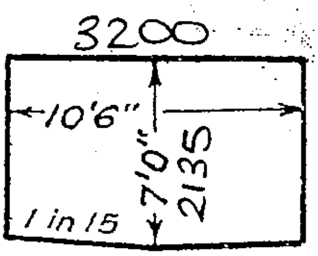


C1-CC1

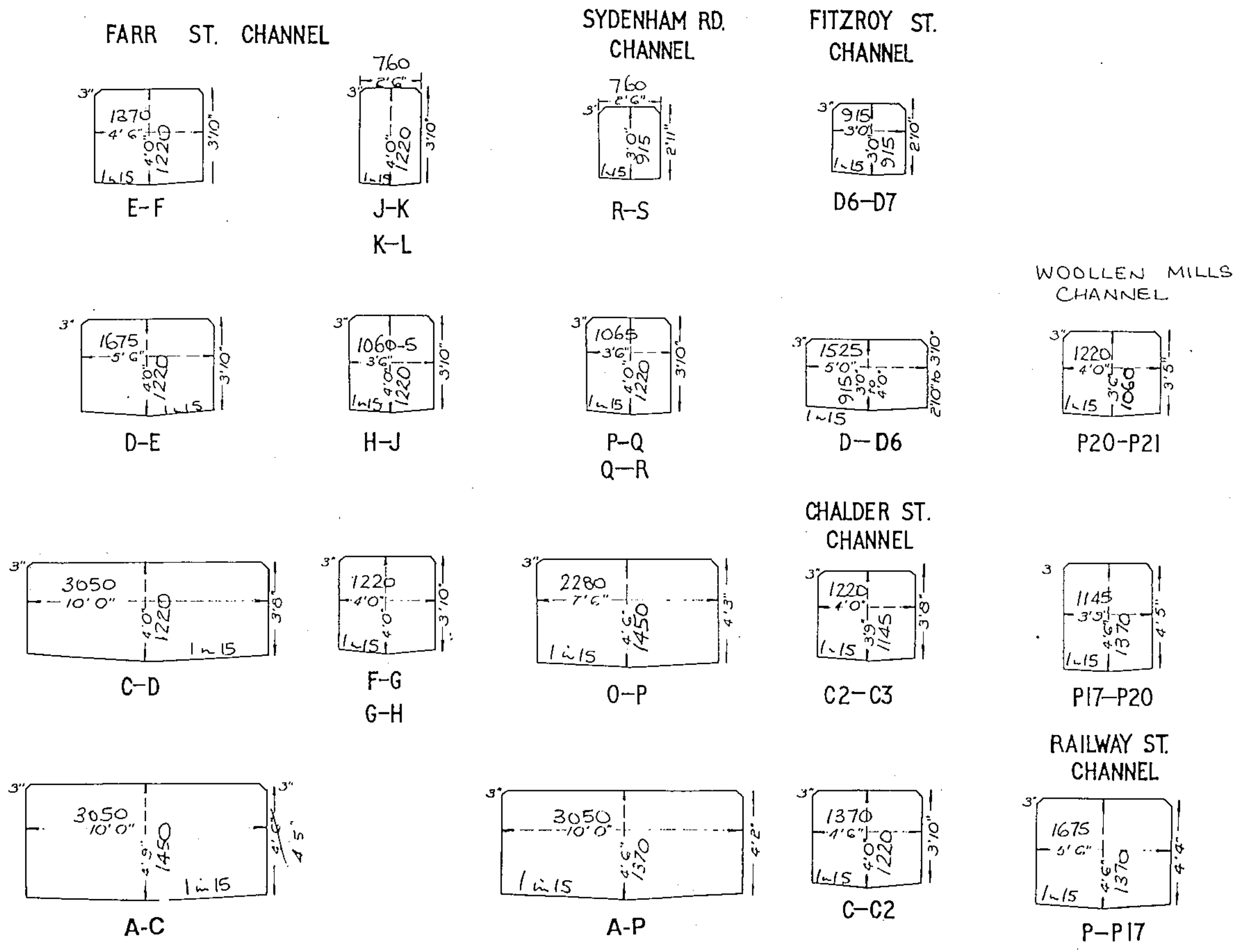
CC1-CC2



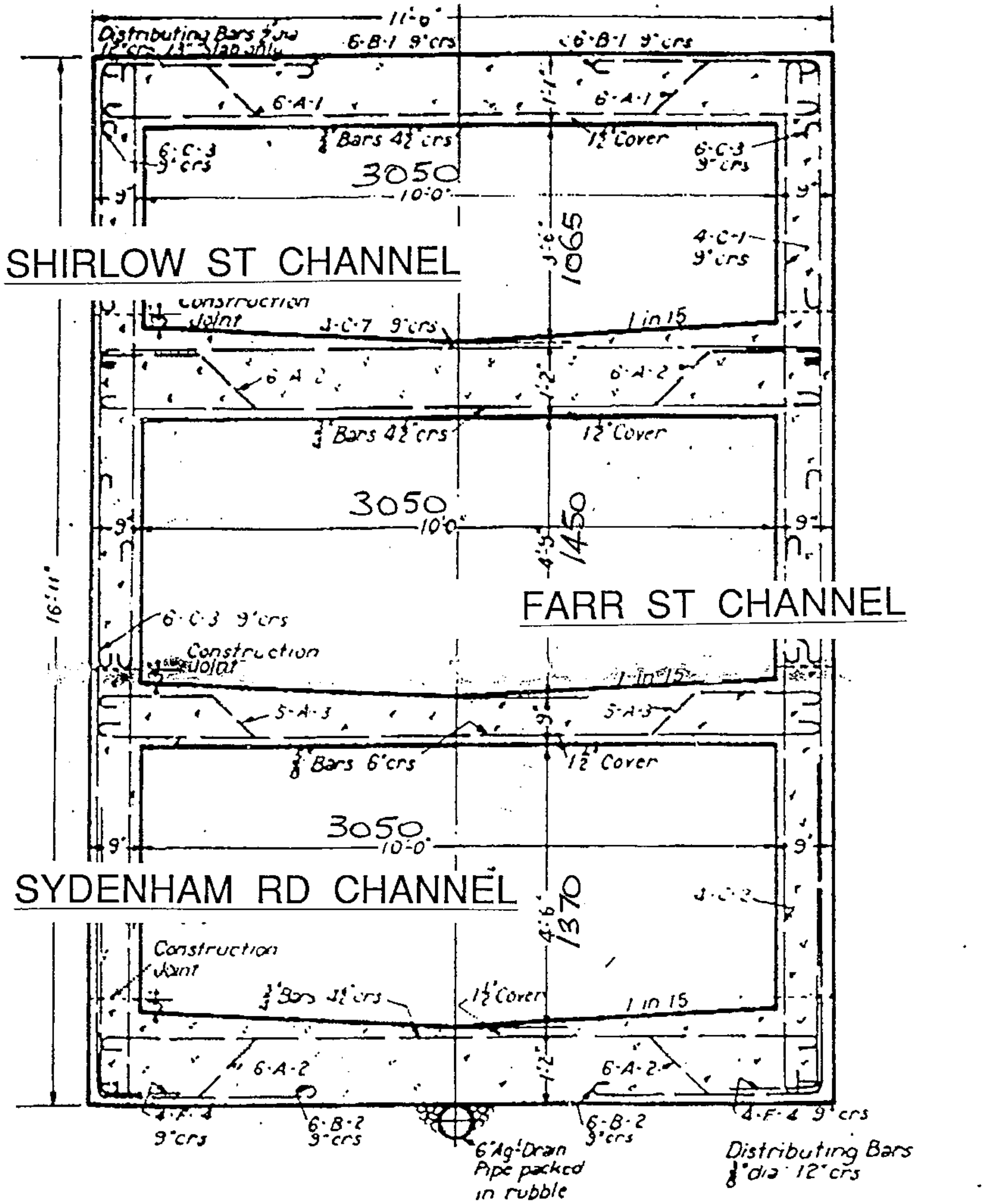
CC2-CC4



LOW LEVEL AREA



MARRICKVILLE VALLEY SWC 66
LOW LEVEL AREA

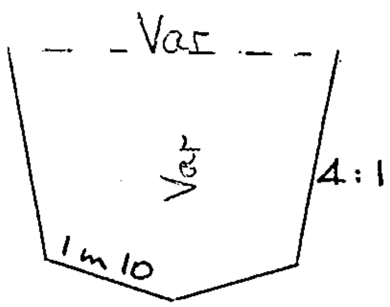


10'-0" x 3'-6" OVER 10'-0" x 4'-9" OVER 10'-0" x 4'-6" RESERVOIR EASEME

CHAINAGE - 23-2 TO 00

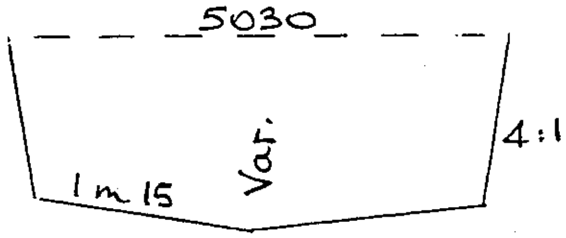
At A

MARRICKVILLE VALLEY SWC 66
CENTRAL CHANNEL 66A

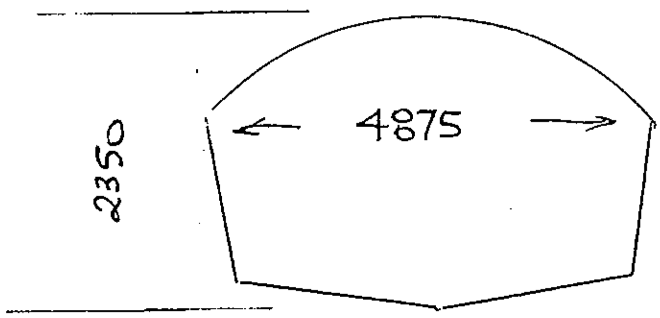


Variously Open and Covered

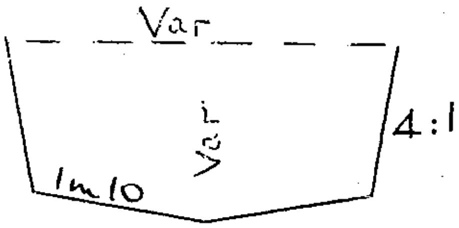
MARRICKVILLE VALLEY SWC 66
WESTERN CHANNEL 66B



A-H



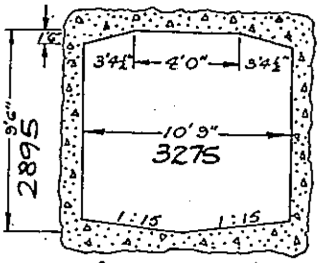
H-J



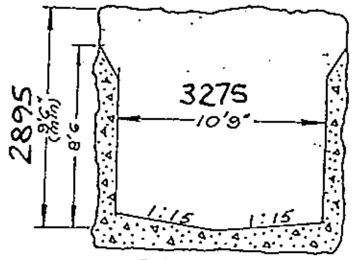
J-ZK

MARRICKVILLE VALLEY SWC 66
WESTERN CHANNEL AMP 66BA

TUNNEL SECTIONS

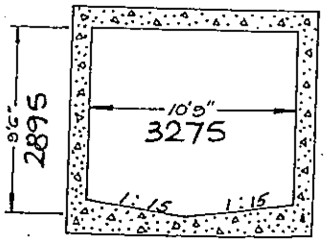


from	to
113	155
1270	1298
1410	1450
1465	1535
2550	3020
3174	3220
3230	3247



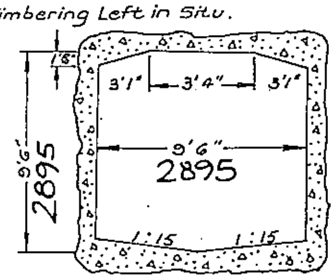
from	to
155	1270
1298	1410
1450	1465
1535	2550

BOX SECTION

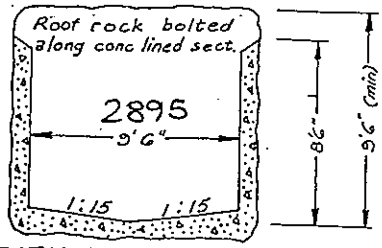


from	to
3020	3174

All Timbering Left in Situ.

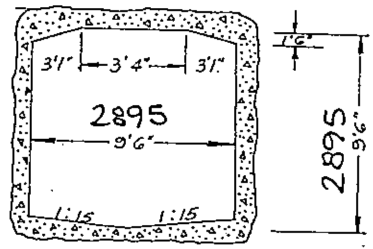


from	to
3252	3370



SECTIONS

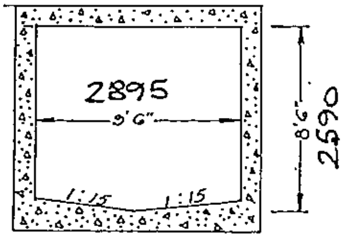
From	To
3370	4210
4230	4320
4420	4860



TUNNEL

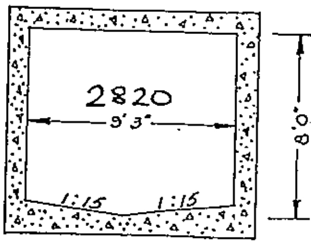
From	To
4210	4230
4320	4420
4860	5584

Timber left in ground



BOX

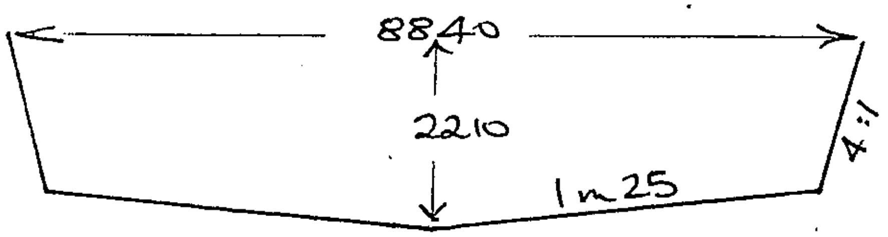
From	To
5584	5719



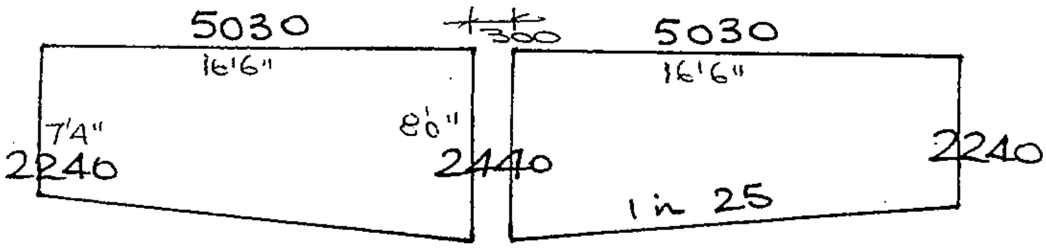
SECTIONS

From	To
5732	6254

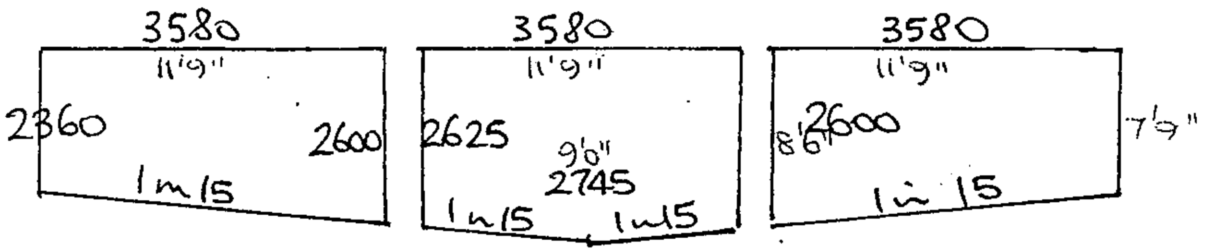
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



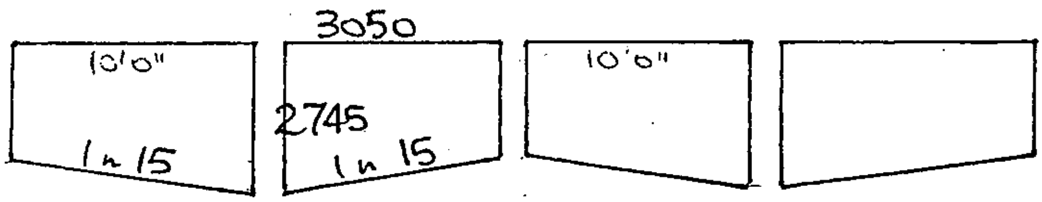
YZ-XZA, UZA-TZA, RZC-RZA



XZ-VZ



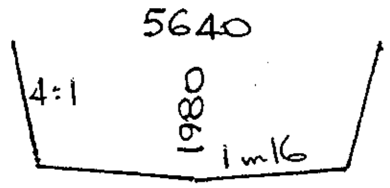
TZ-SZ

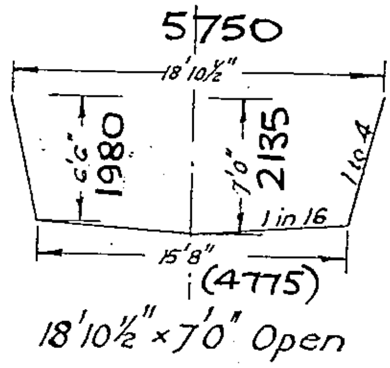


RZ-QZB

MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

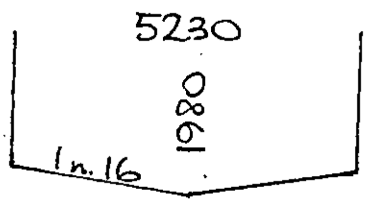
(Looking downstream)

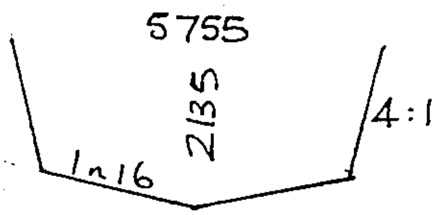




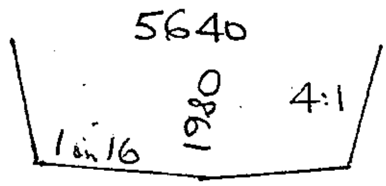
QZ-PZA

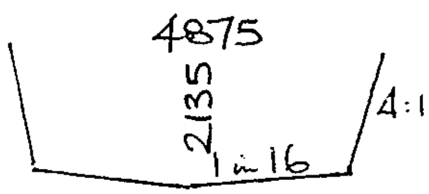
NZ-MZ





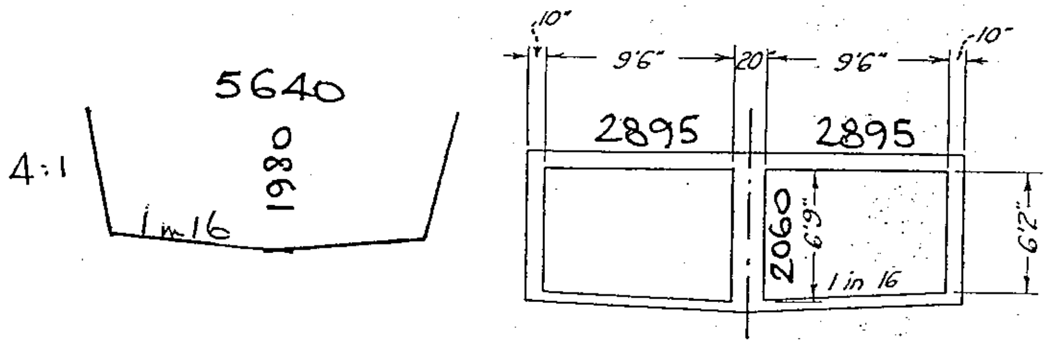
PZ-NZA



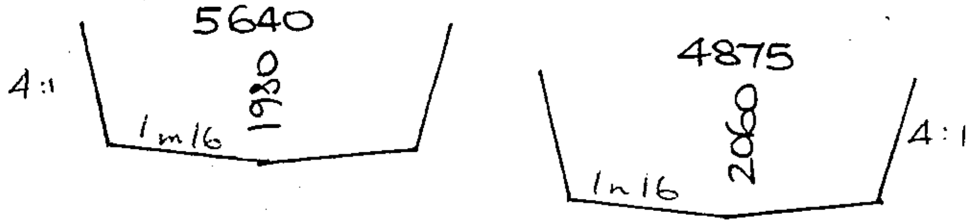


MZ-KZ

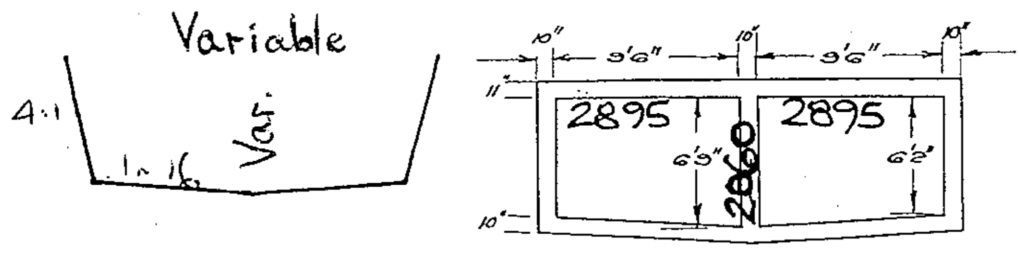
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



KZ-JZ
HZ-GZ

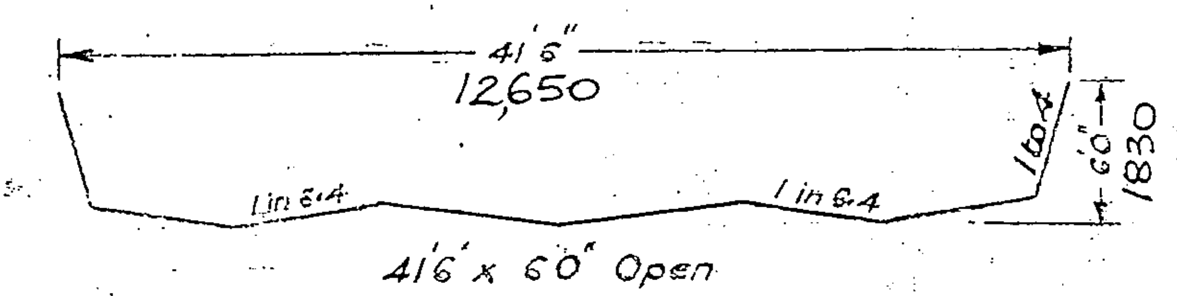


JZ-HZ

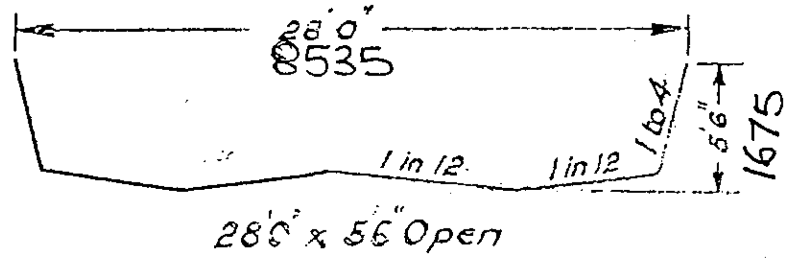


GZ-A2

MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

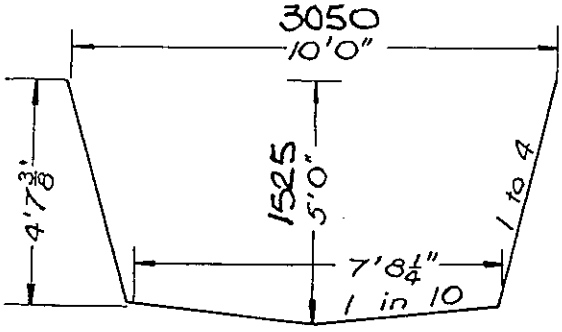


AA2-BA

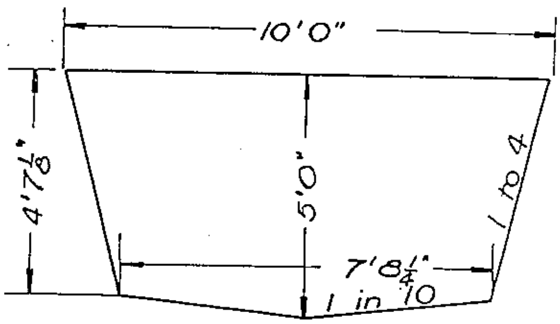


BA1-C

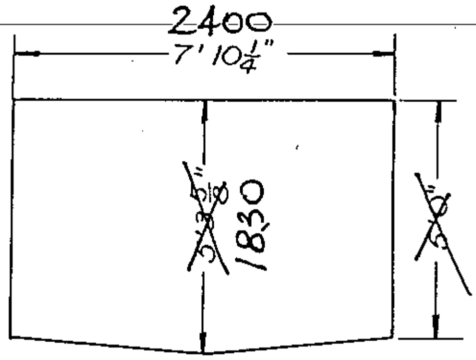
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



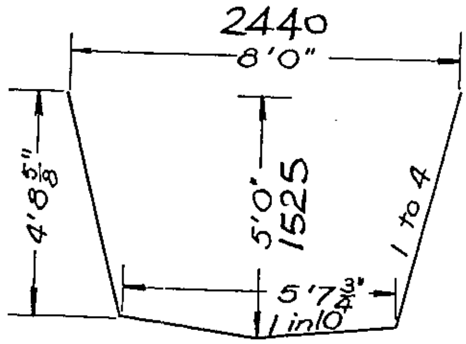
C-D E-F



D-E

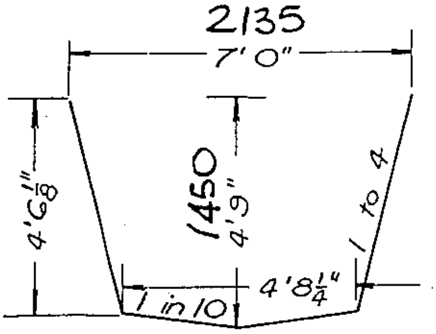


F-G

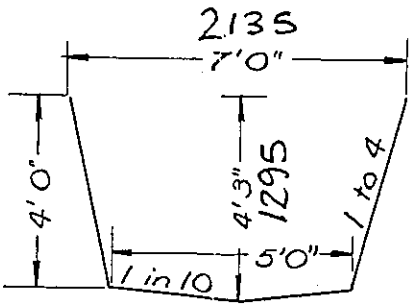


G-H

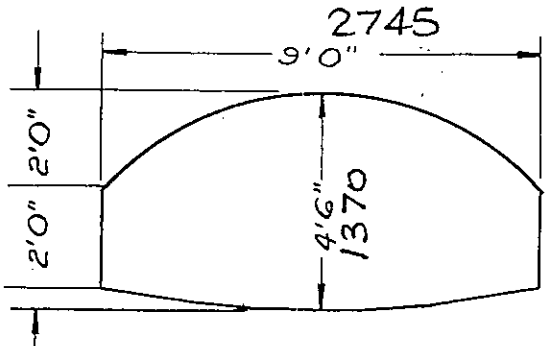
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL



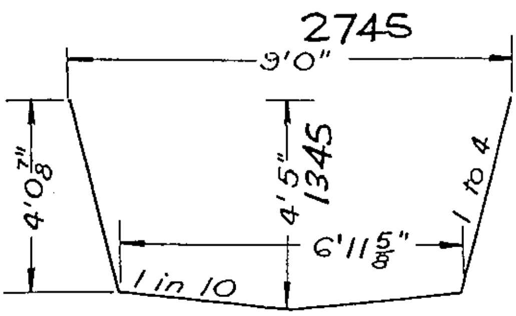
H-J-K-L



I-M



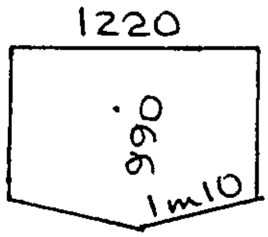
M-N O-P-Q



N-O

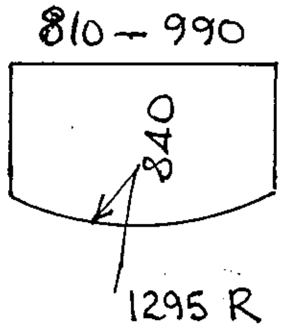
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

BRANCH EB

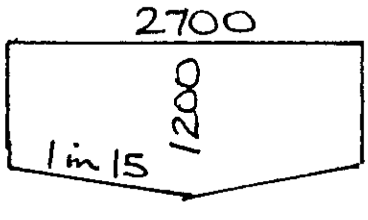


UZ-UZ1

GROVE ST BRANCH



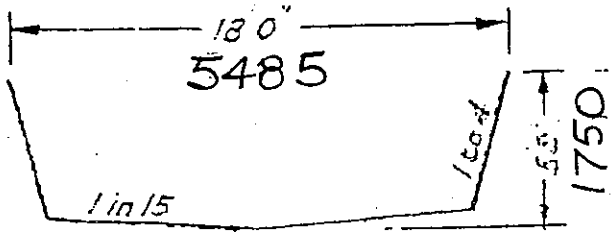
S(A)-S1
S1-S2
S3-S5



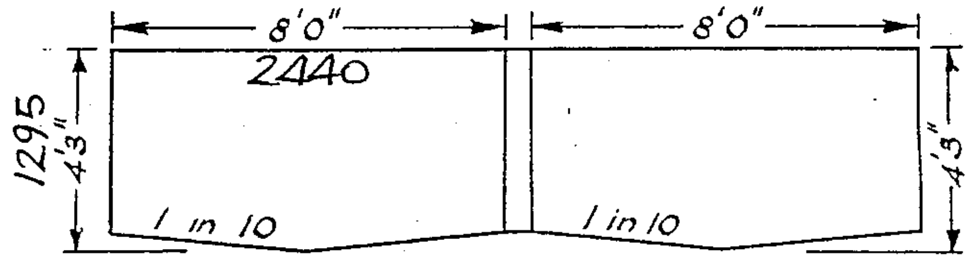
S2-S3

MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

EASTERN CHANNEL AMP

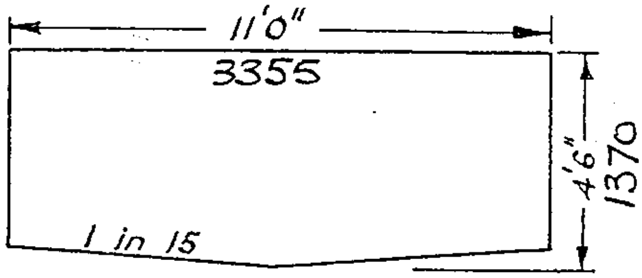


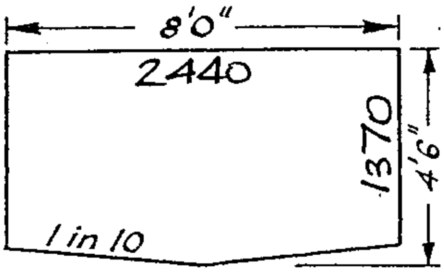
BB-BB1



BB1A-BB2

BB2-BB3

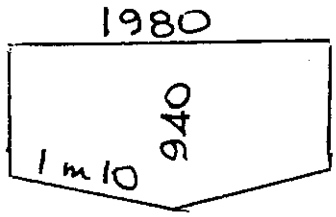




BB3-BB3C

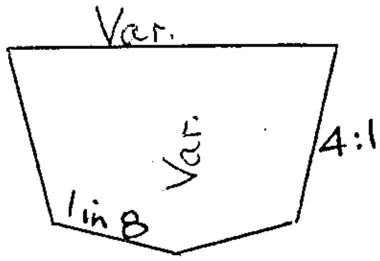
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

MURRAY ST BRANCH

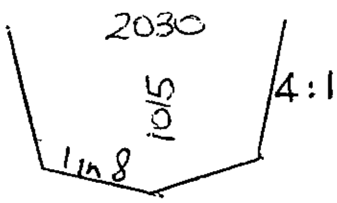


B1A-B1-B2

EDGEWARE RD BRANCH



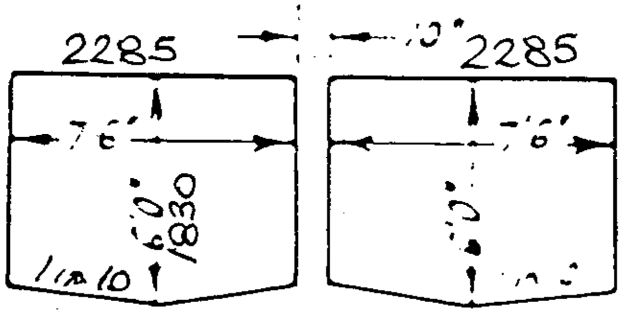
C-C7
C8-C9



C7-C8

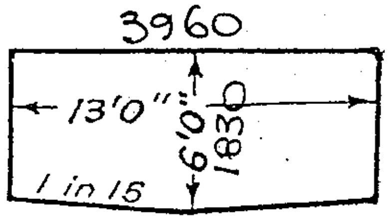
MARRICKVILLE VALLEY SWC 66
EASTERN CHANNEL

EASTERN CHANNEL AMP

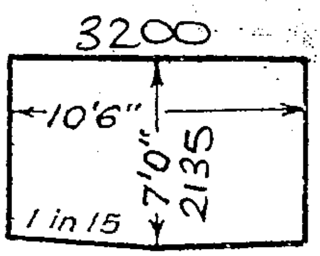


C1-CC1

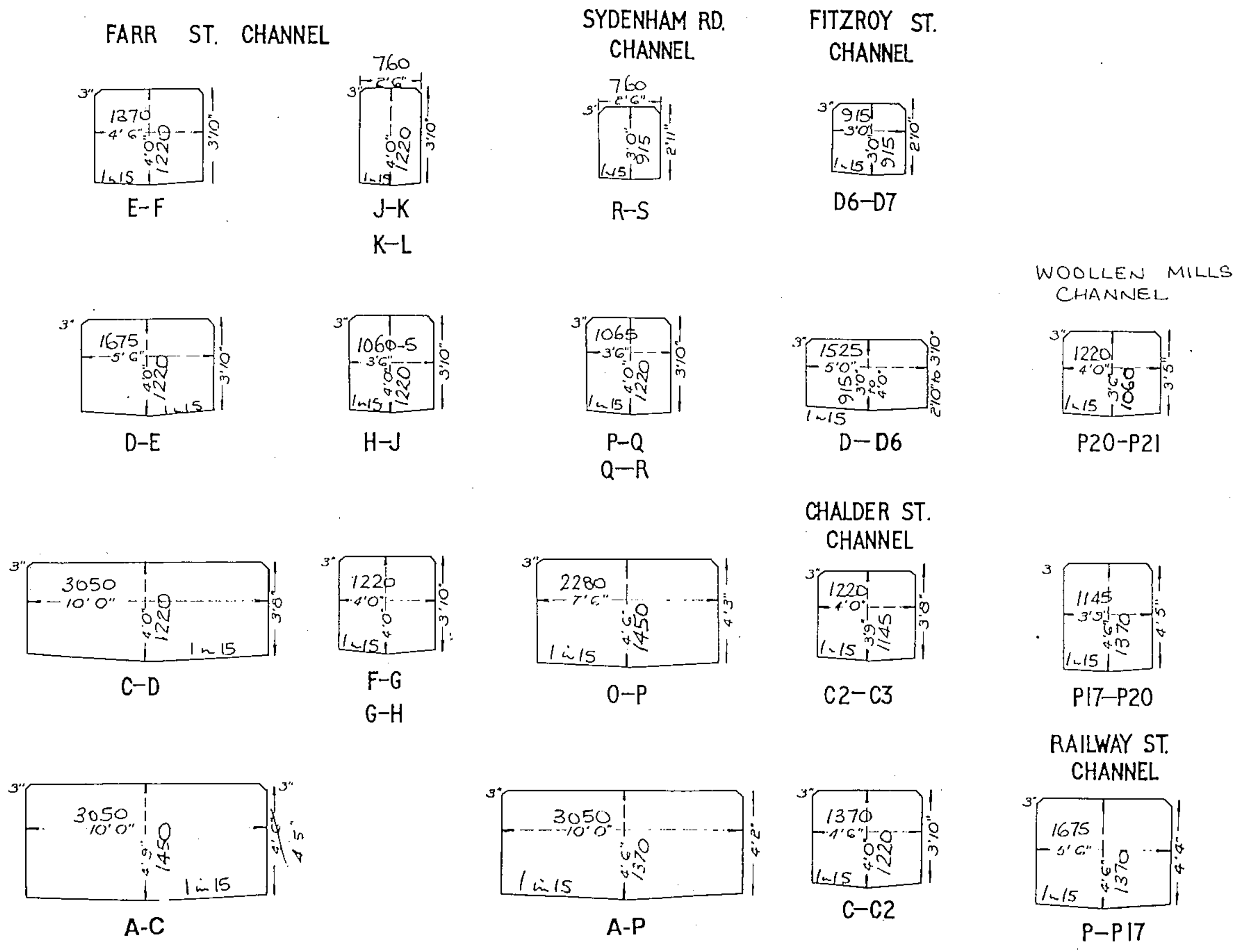
CC1-CC2



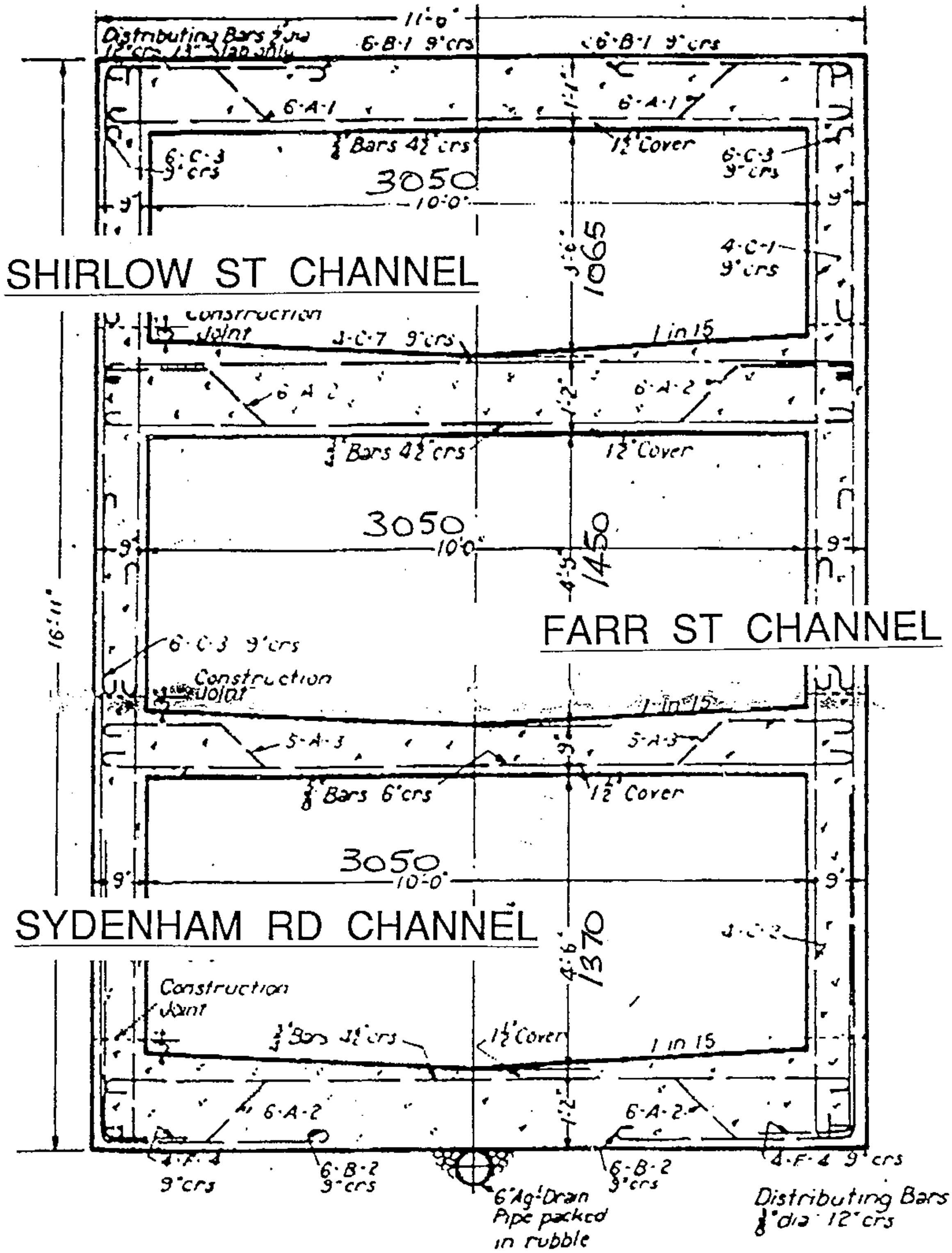
CC2-CC4



LOW LEVEL AREA



MARRICKVILLE VALLEY SWC 66
LOW LEVEL AREA

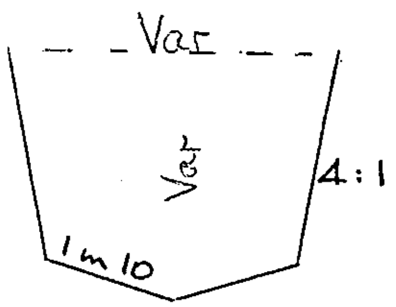


10'-0" x 3'-6" OVER 10'-0" x 4'-9" OVER 10'-0" x 4'-6" RESERVOIR EASEME

CHAINAGE - 23-2 TO 00

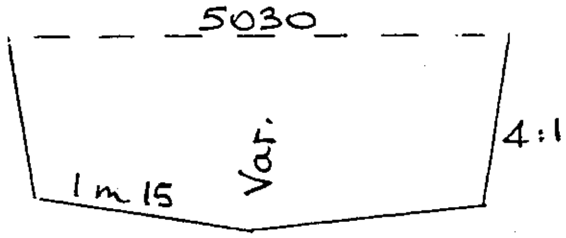
At A

MARRICKVILLE VALLEY SWC 66
CENTRAL CHANNEL 66A

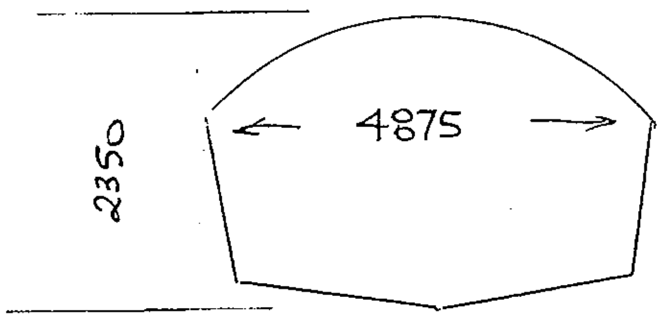


Variously Open and Covered

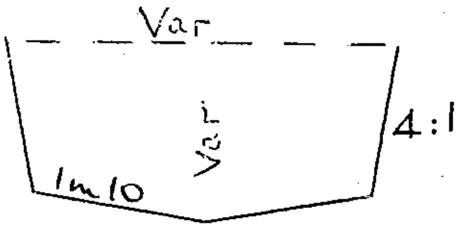
MARRICKVILLE VALLEY SWC 66
WESTERN CHANNEL 66B



A-H



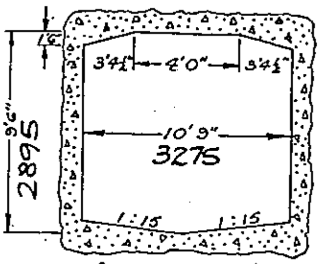
H-J



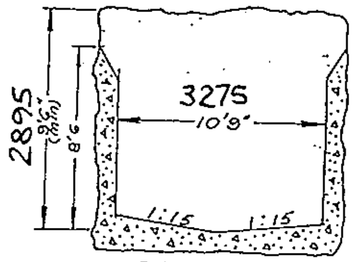
J-ZK

MARRICKVILLE VALLEY SWC 66
WESTERN CHANNEL AMP 66BA

TUNNEL SECTIONS

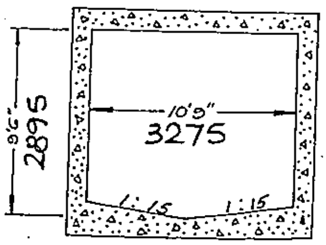


from	to
113	155
1270	1298
1410	1450
1465	1535
2550	3020
3174	3220
3230	3247



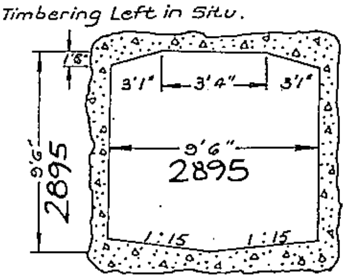
from	to
155	1270
1298	1410
1450	1465
1535	2550

BOX SECTION

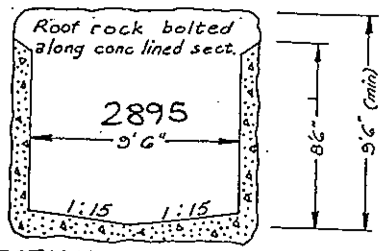


from	to
3020	3174

All Timbering Left in Situ.

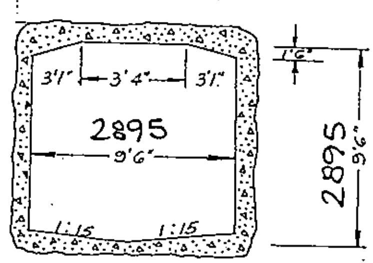


from	to
3252	3370



SECTIONS

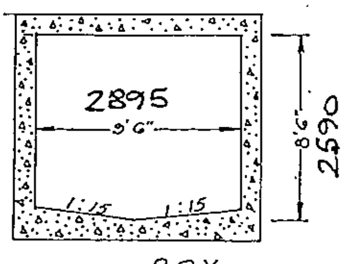
From	To
3370	4210
4230	4320
4420	4860



TUNNEL

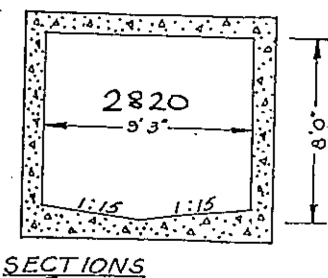
From	To
4210	4230
4320	4420
4860	5584

Timber left in ground



BOX

From	To
5584	5719



SECTIONS

From	To
5732	6254

Figures

MARRICKVILLE VALLEY FLOOD STUDY

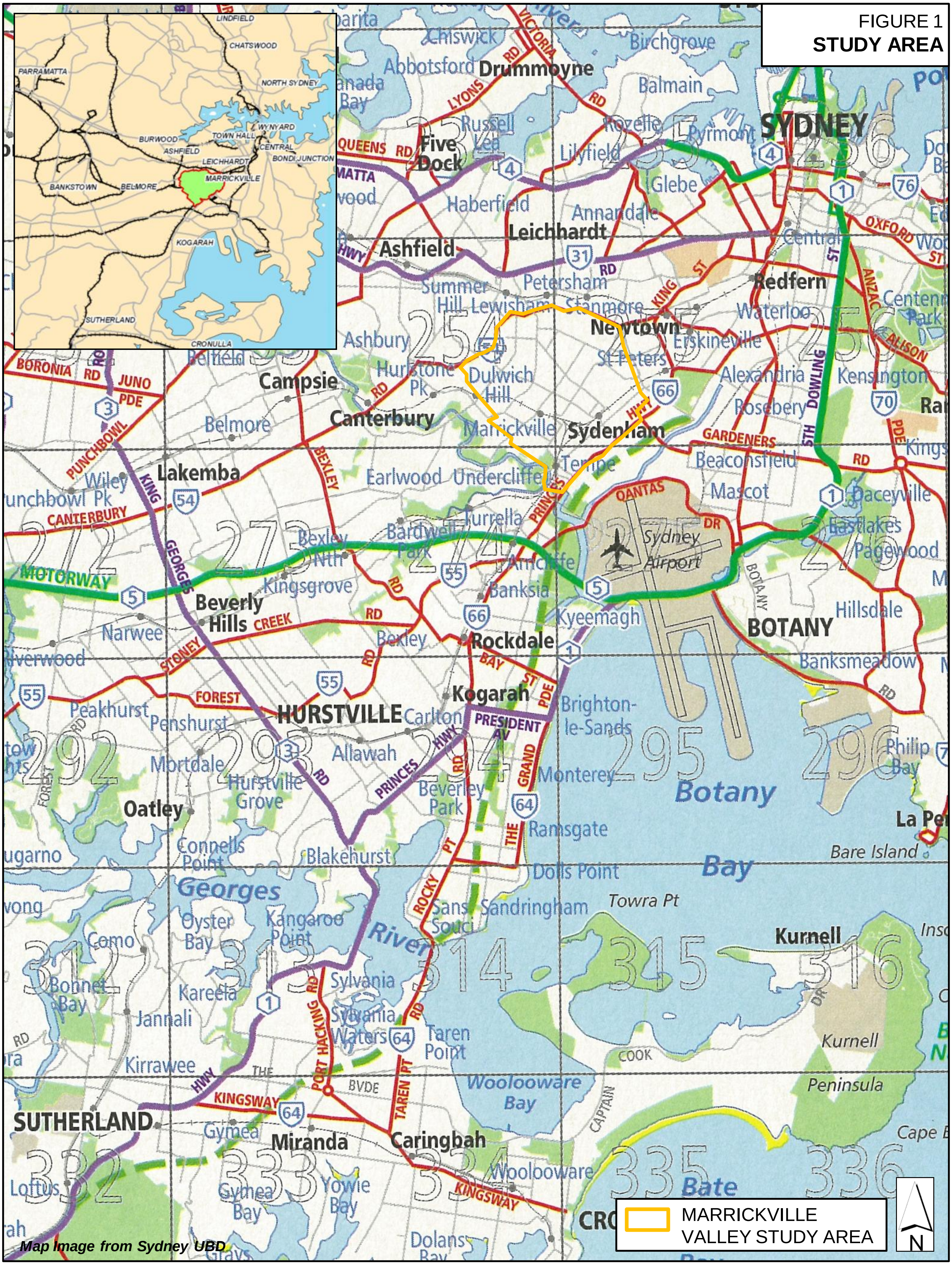


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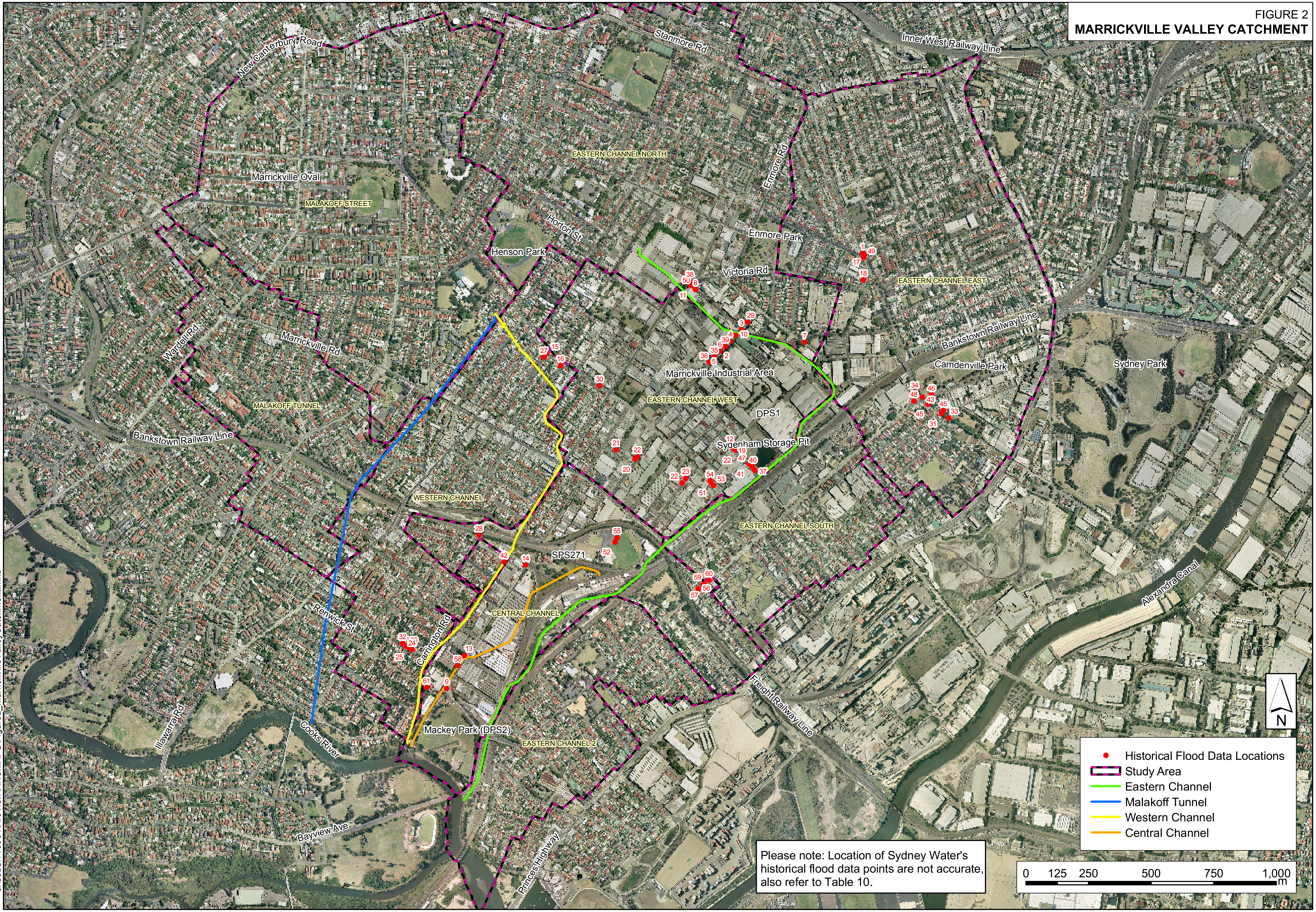
FIGURE 2
MARRICKVILLE VALLEY CATCHMENT



J:\Jobs\110004\ArcView\ARCMAPS\Figure2_MarrickvilleValleyCatchment.mxd

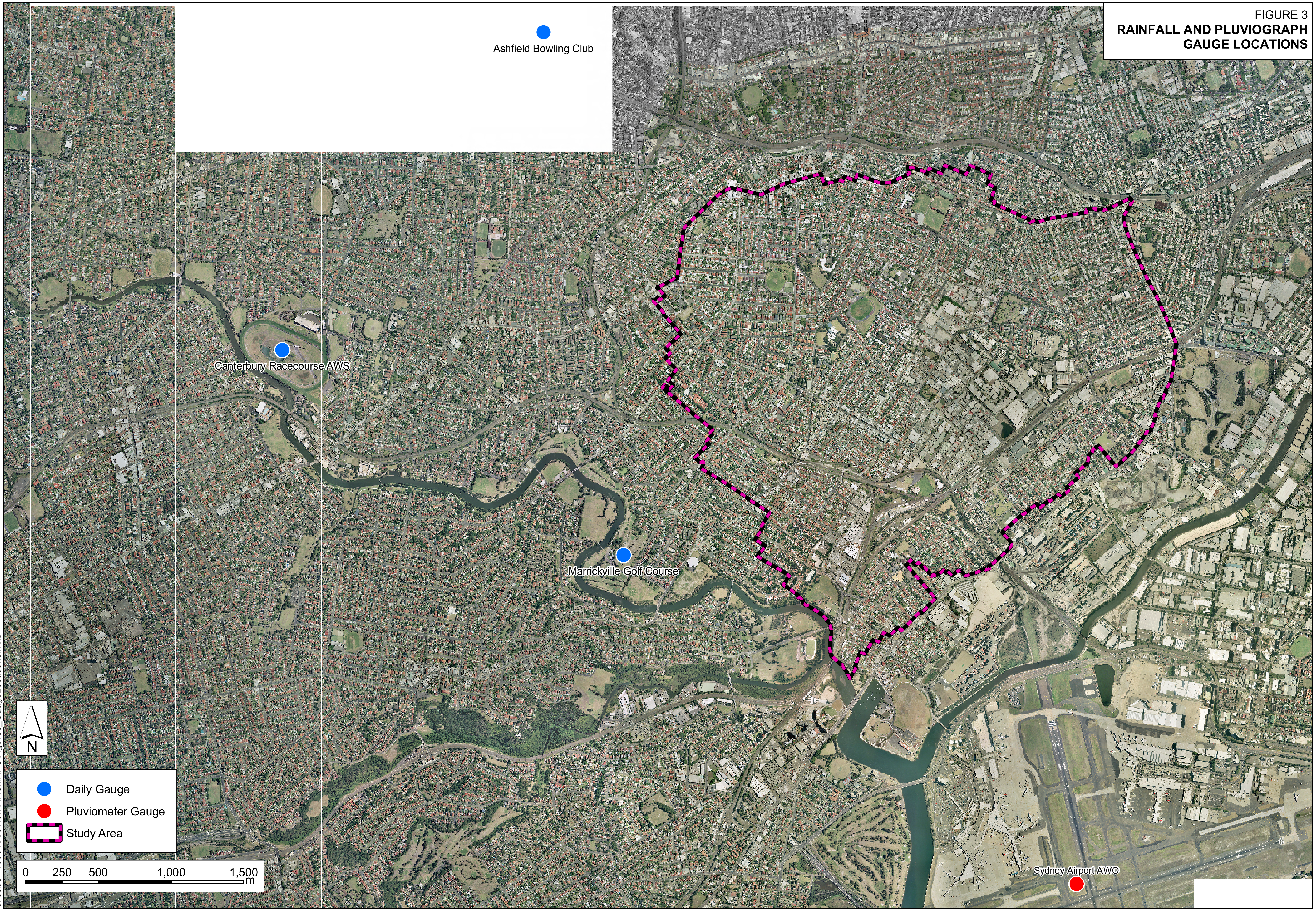
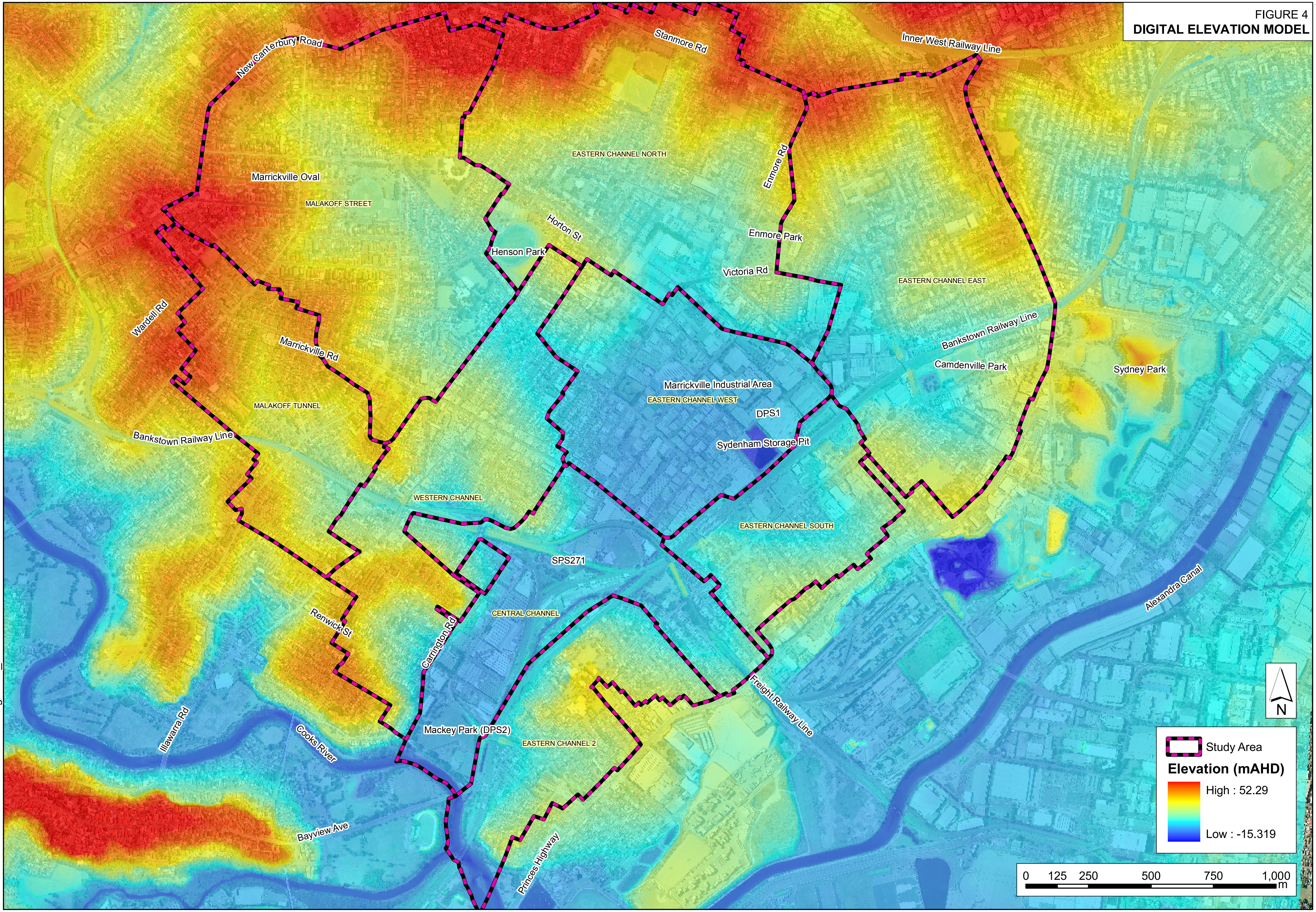


FIGURE 3
RAINFALL AND PLUVIOGRAPH GAUGE LOCATIONS

J:\Jobs\110004\ArcView\ARCMAPS\Figure3_GaugeLocations.mxd

FIGURE 4
DIGITAL ELEVATION MODEL



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SYDNEY AIRPORT

Burst Intensities and Frequency

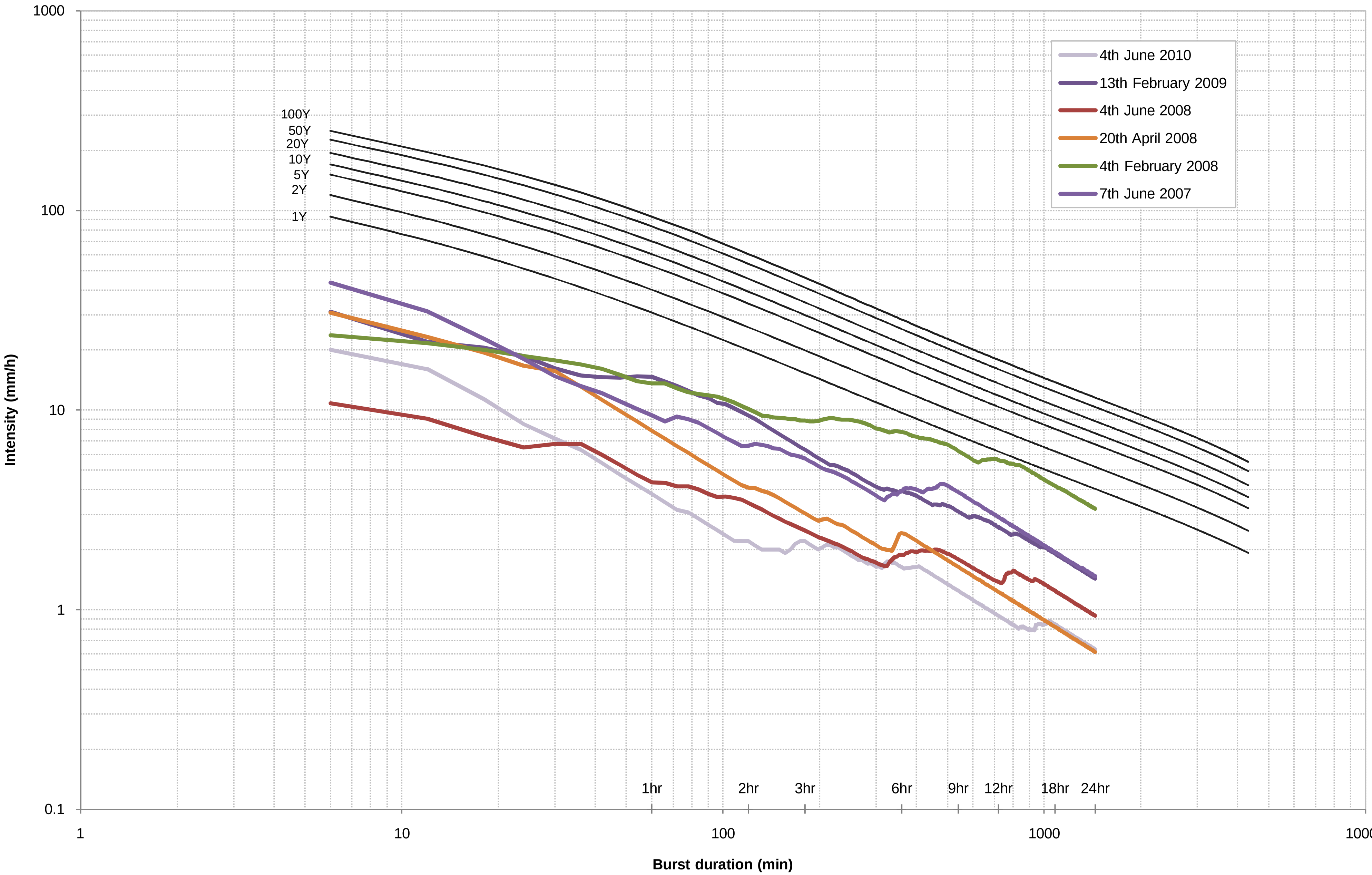
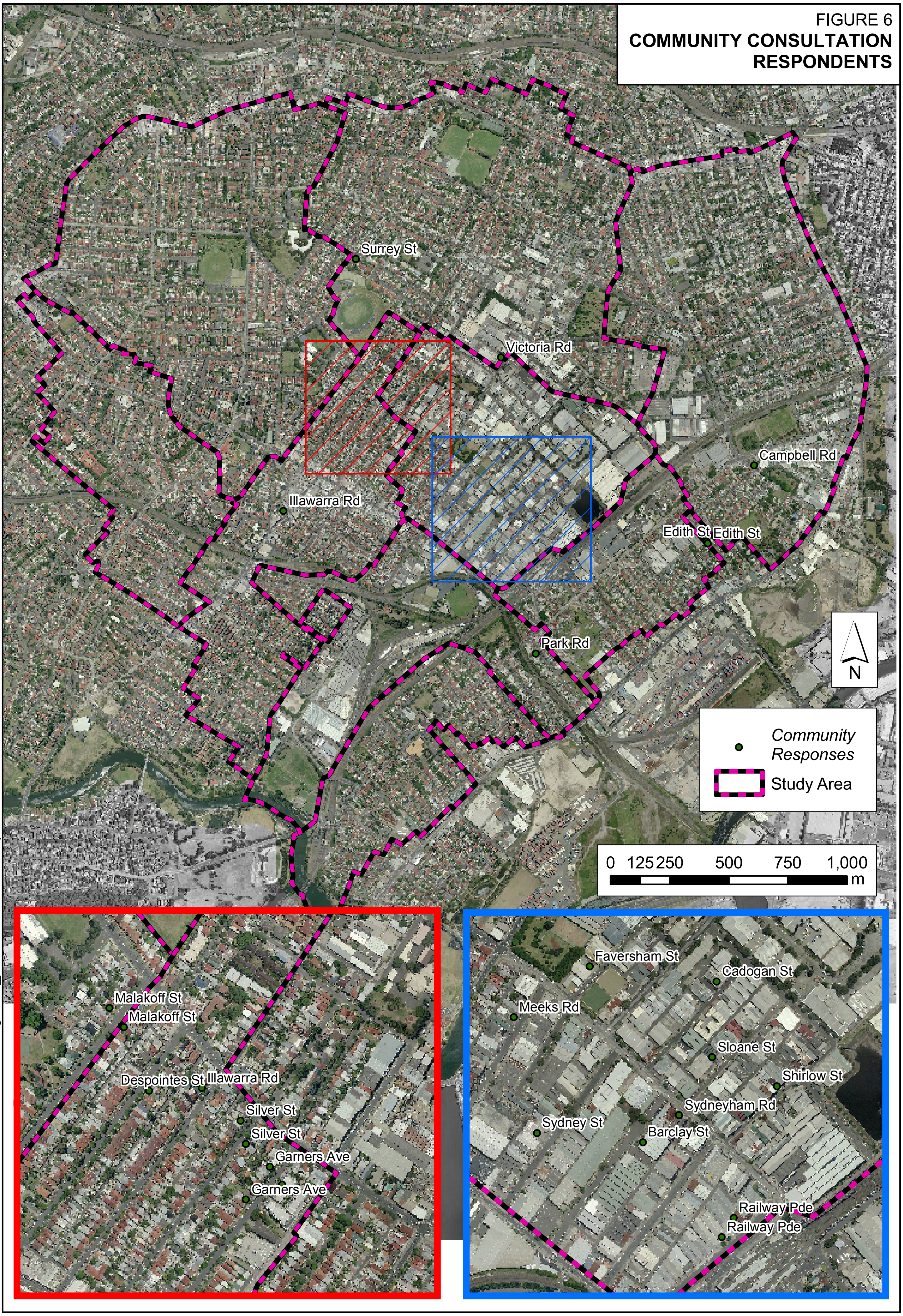


FIGURE 5
IFD PLOT OF RECENT EVENTS

MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 6
COMMUNITY CONSULTATION RESPONDENTS



J:\Jobs\110004\ArcView\ARCMAPS\Figure6_CommConsultResponse.mxd

MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 7

COMMUNITY CONSULTATION
FLOOD AFFECTED RESPONDENTS

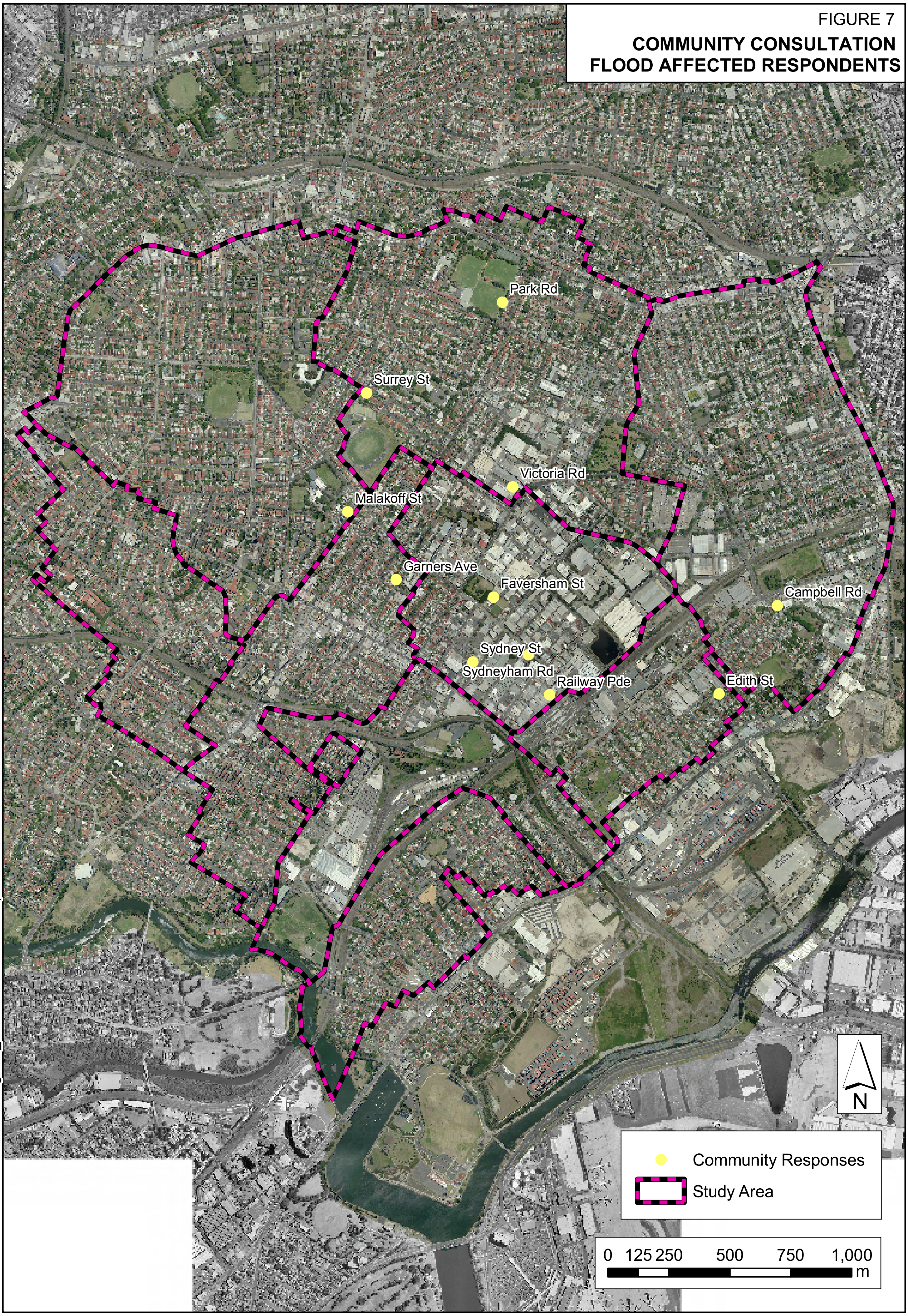


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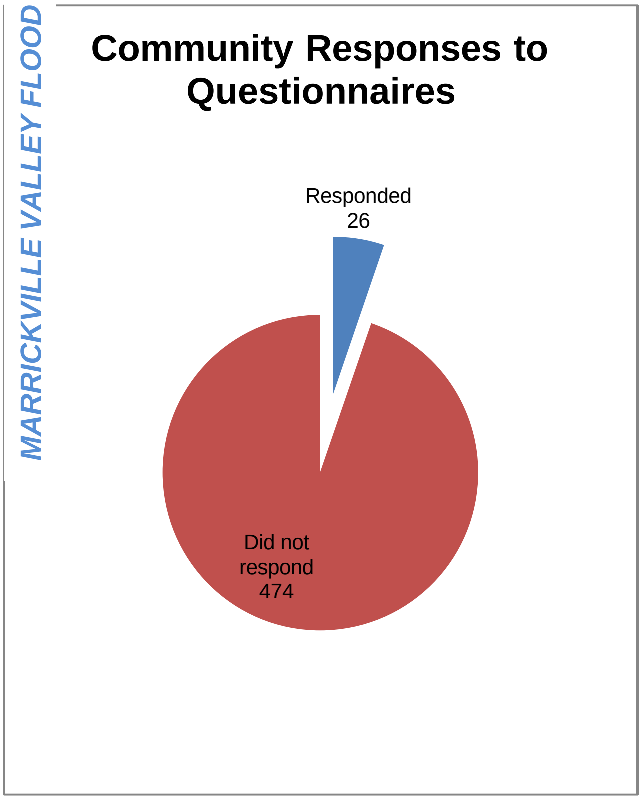
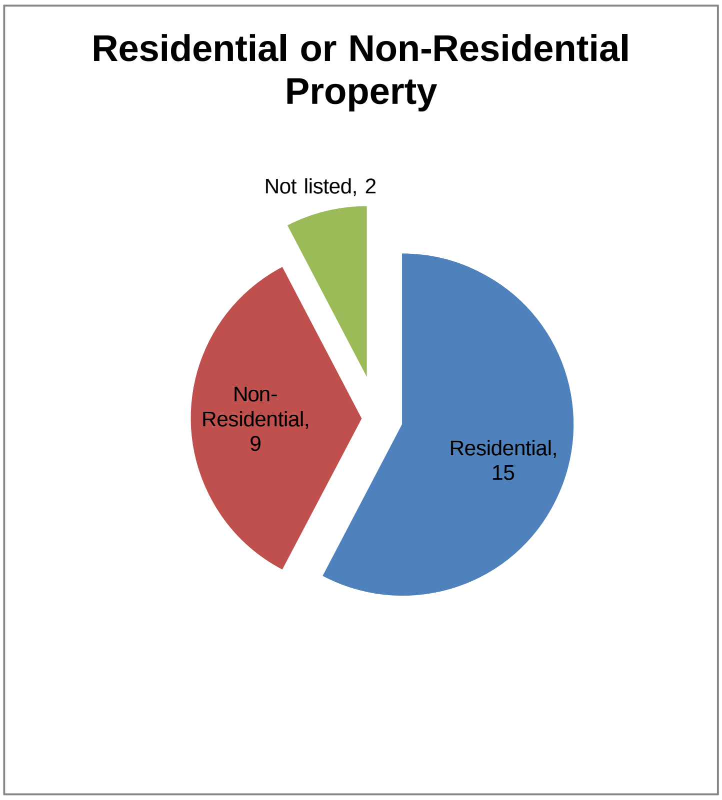
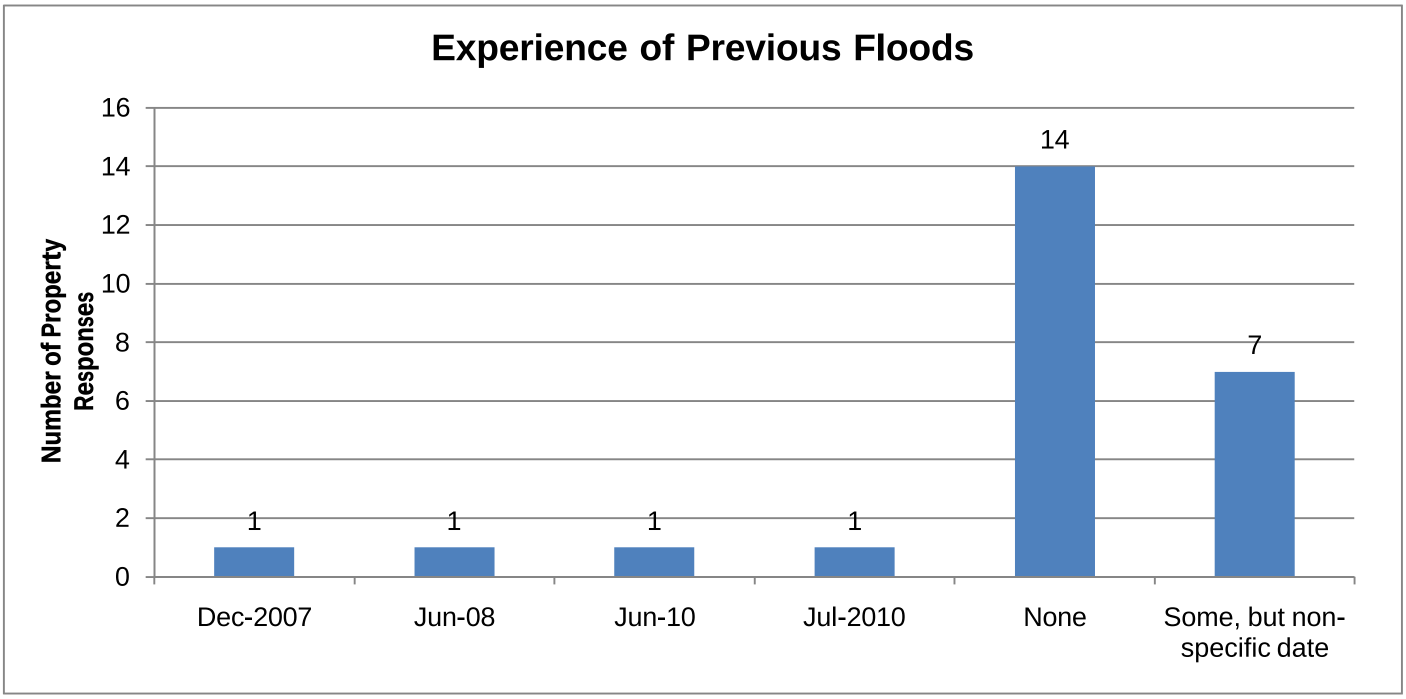
FIGURE 8
COMMUNITY CONSULTATION RESULTS







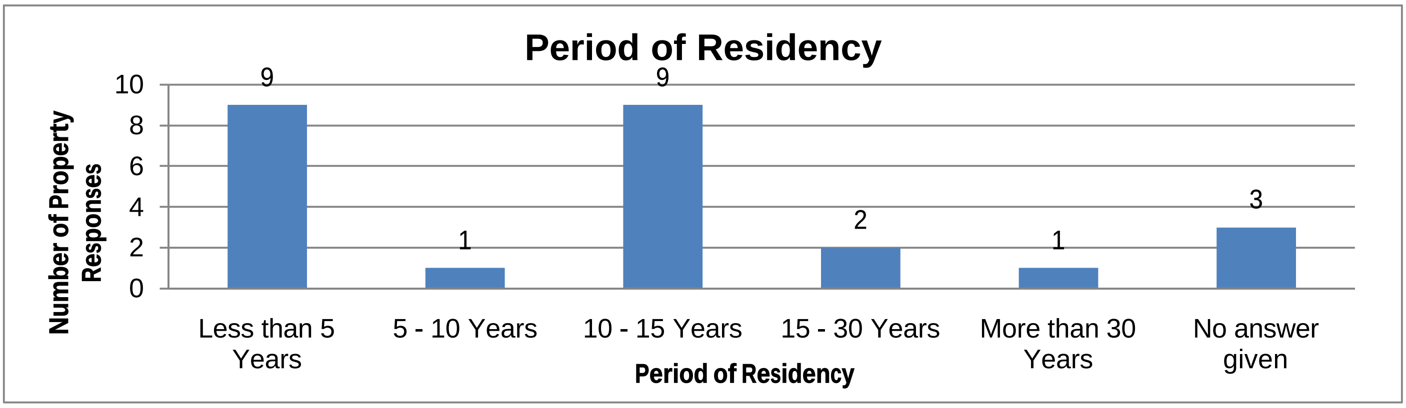
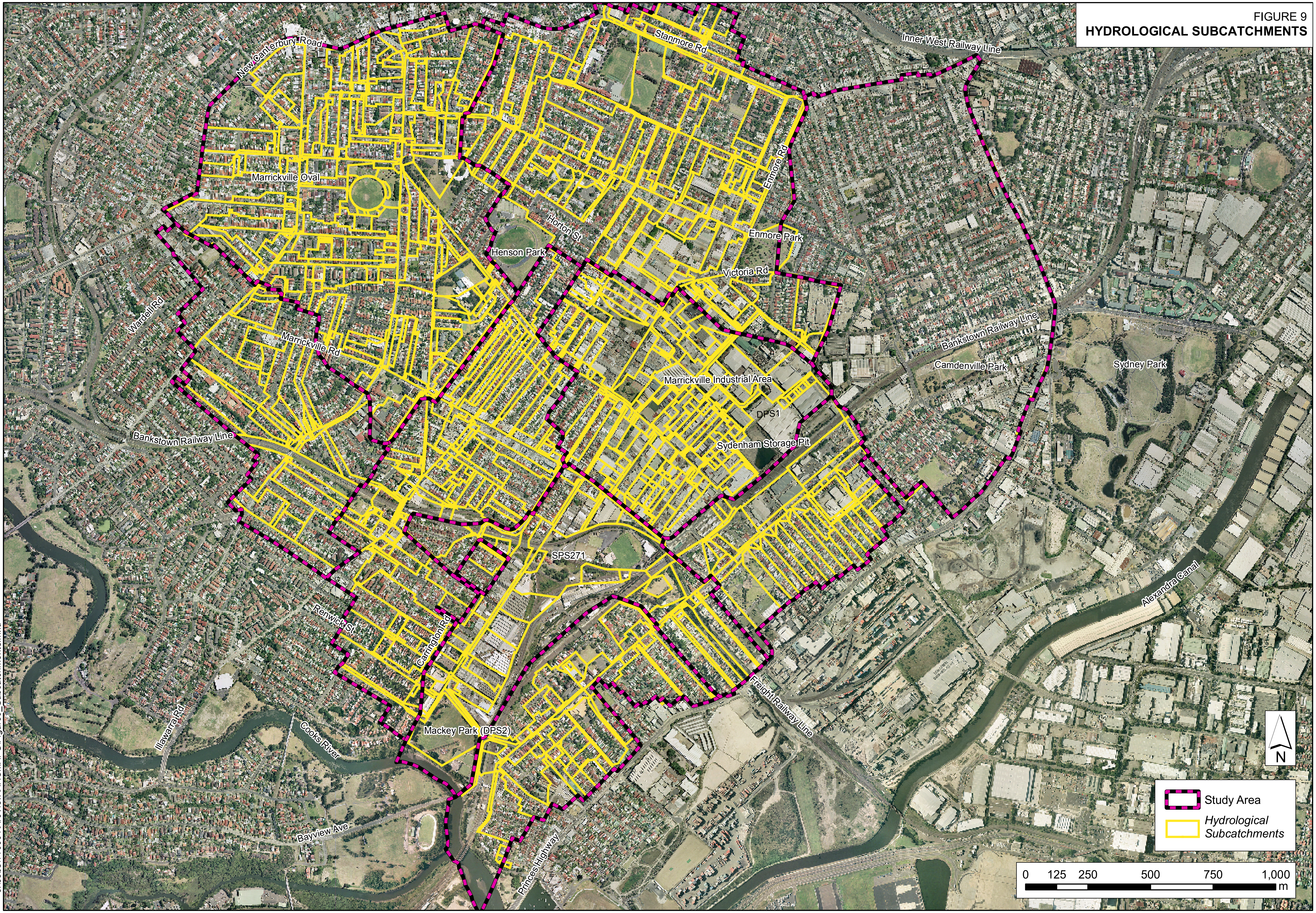


FIGURE 9
HYDROLOGICAL SUBCATCHMENTS



J:\Jobs\110004\ArcView\ARCMAPS\Figure9_Subcatchments.mxd

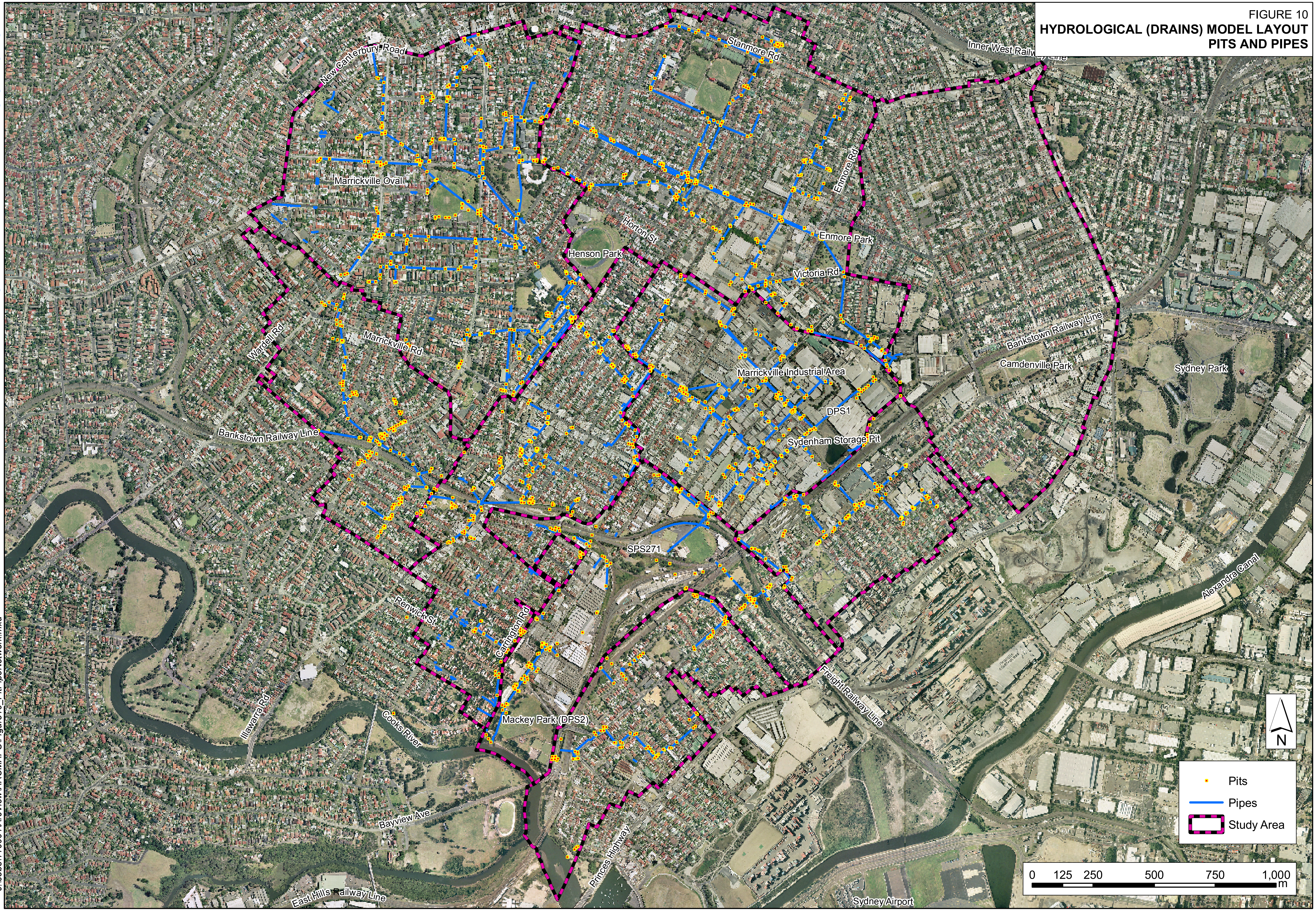


FIGURE 10
HYDROLOGICAL (DRAINS) MODEL LAYOUT PITS AND PIPES

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MARRICKVILLE VALLEY FLOOD STUDY

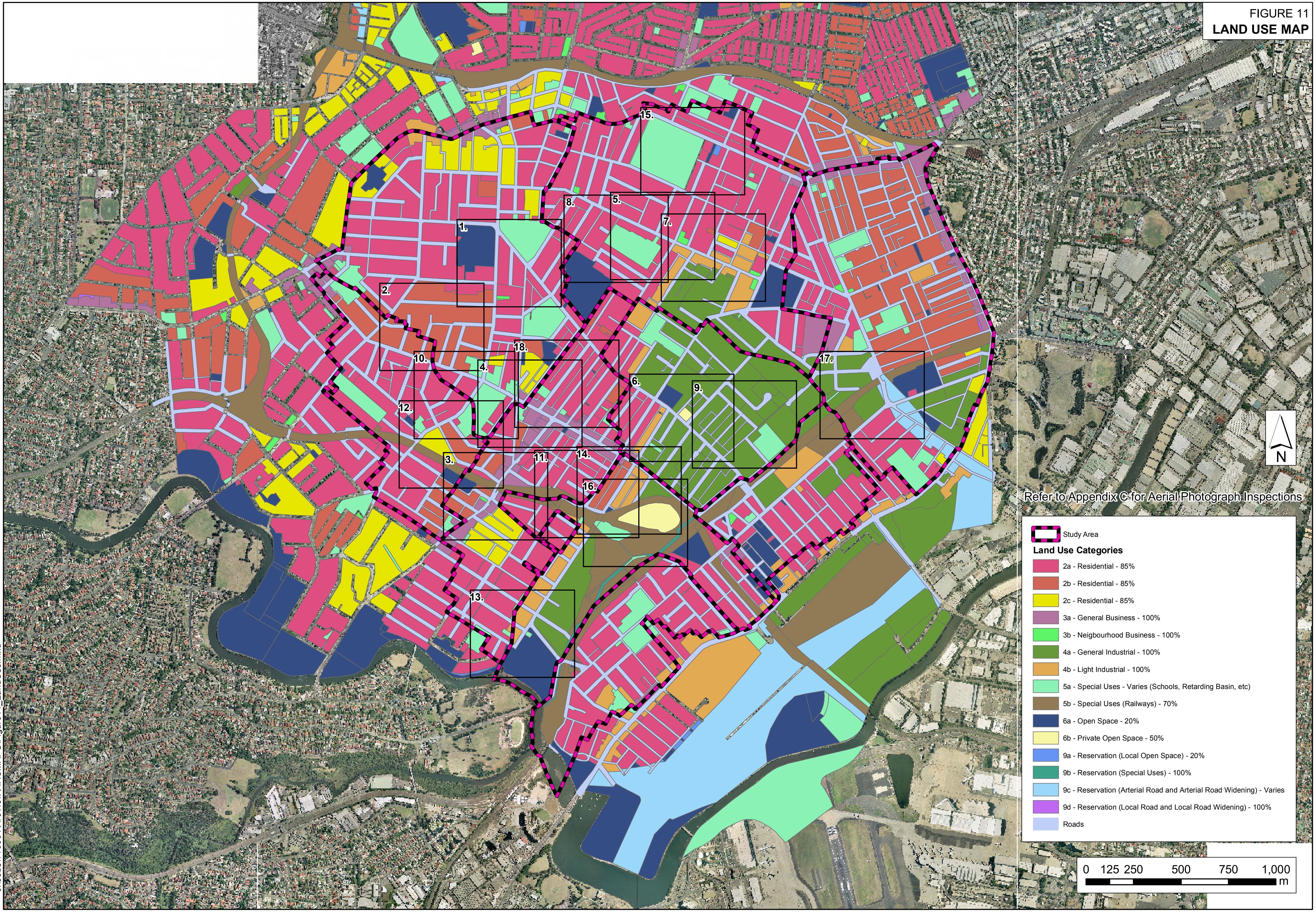

FIGURE 11
LAND USE MAP

J:\Jobs\110004\ArcView\ARCMAPS\Figure11_LandUse.mxd

MARRICKVILLE VALLEY FLOOD STUDY

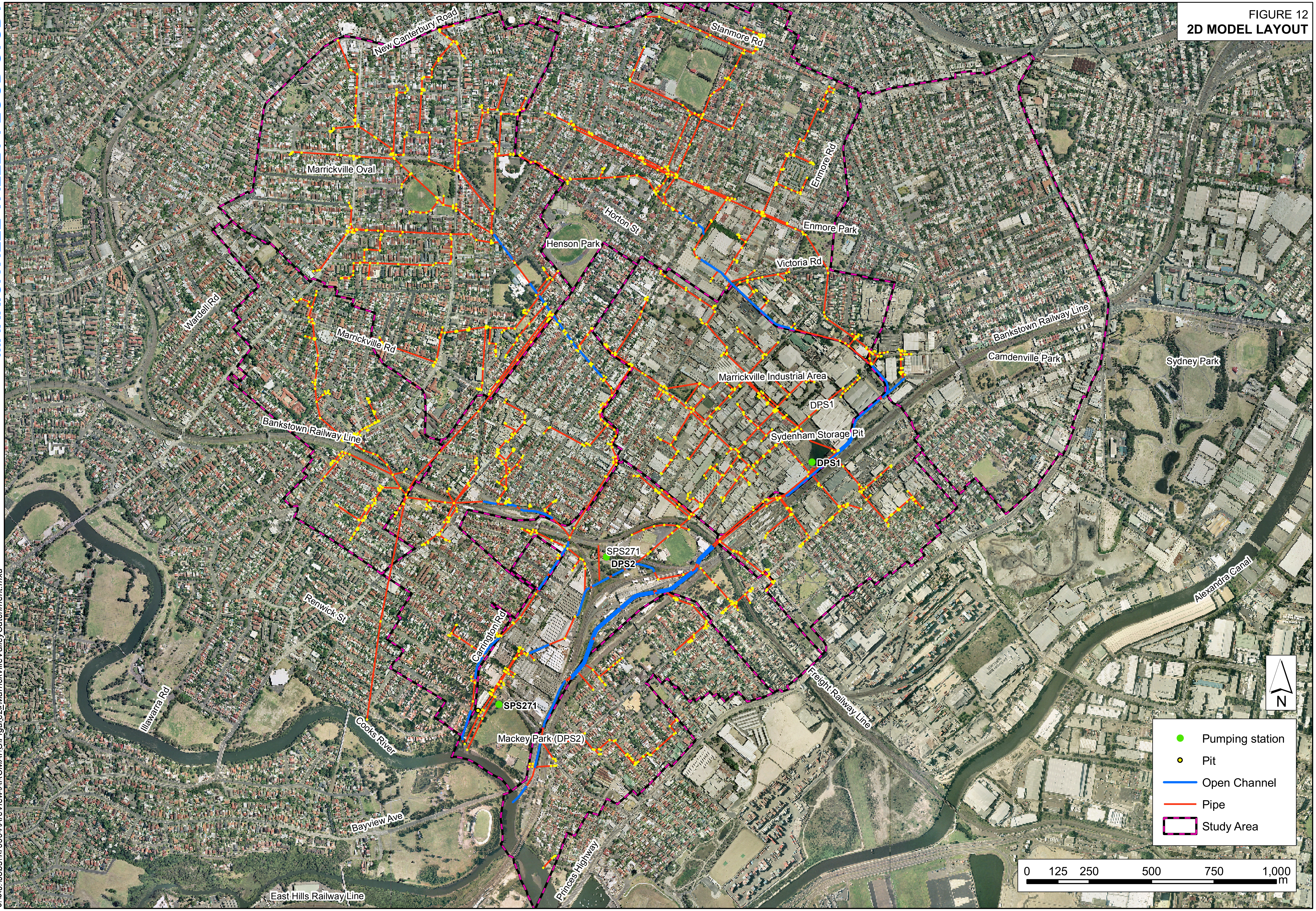

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FIGURE 13
TABULATED FLOODING LOCATIONS

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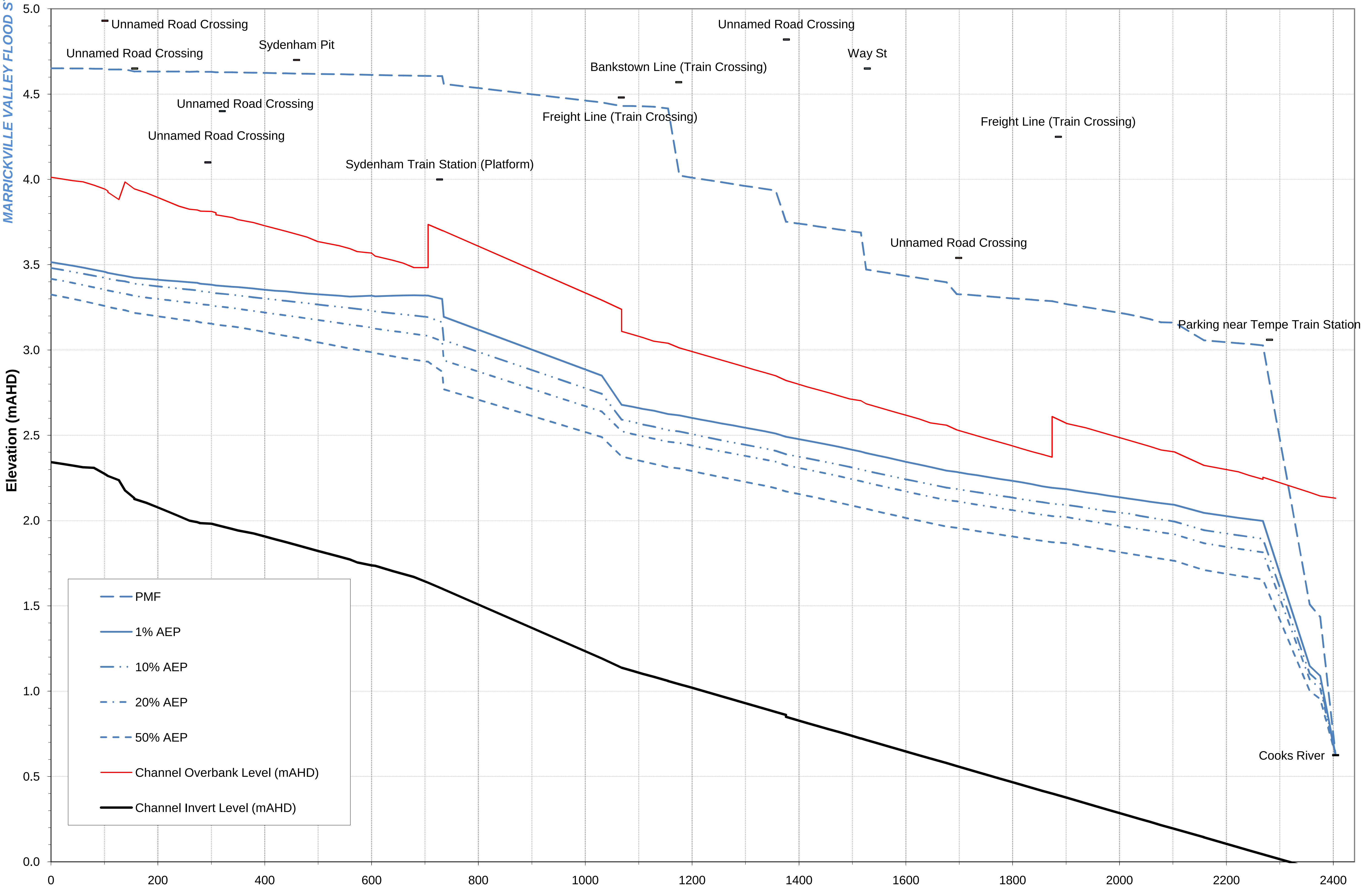
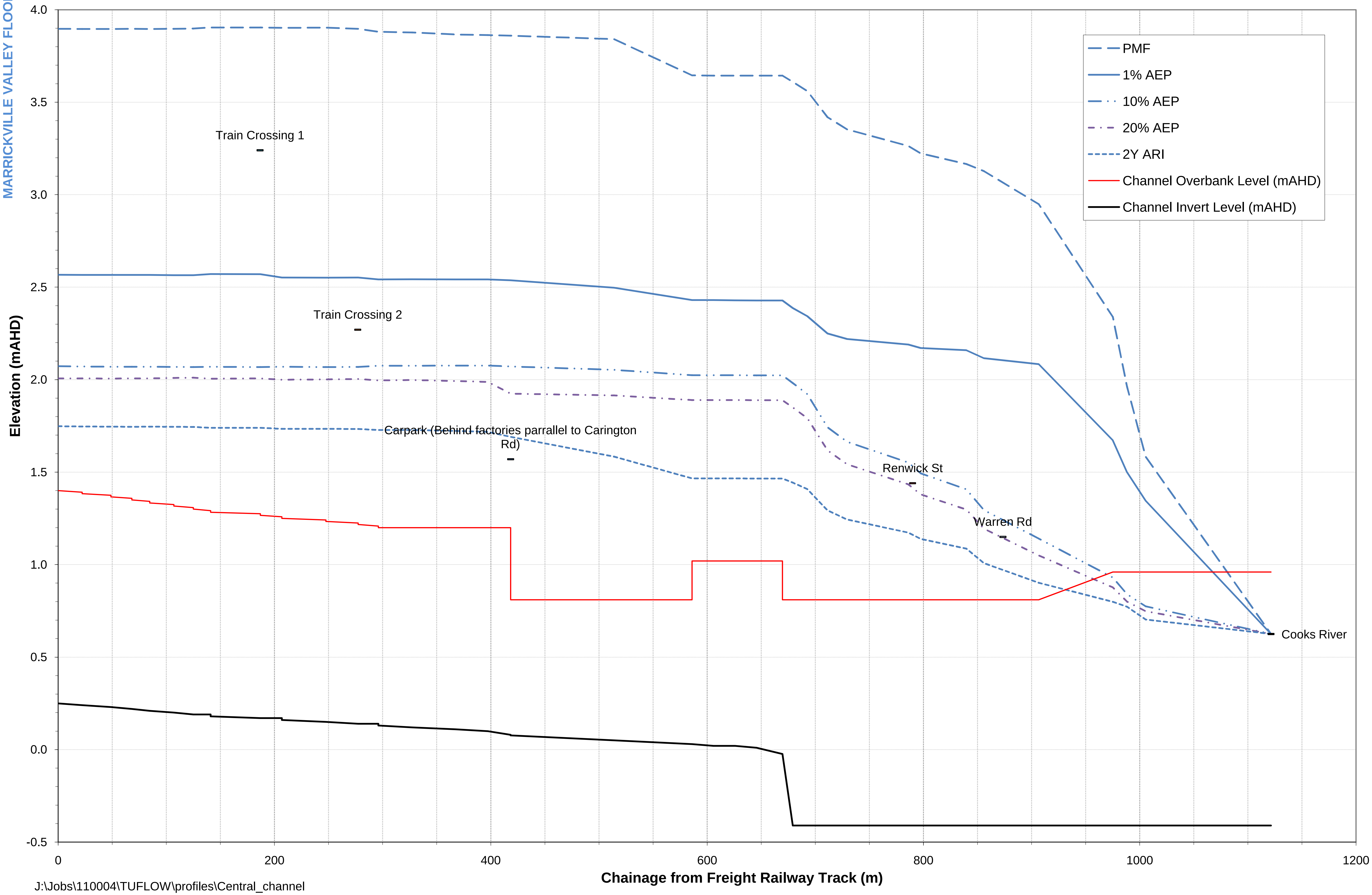
FIGURE 14
PEAK FLOOD PROFILES - EASTERN CHANNEL



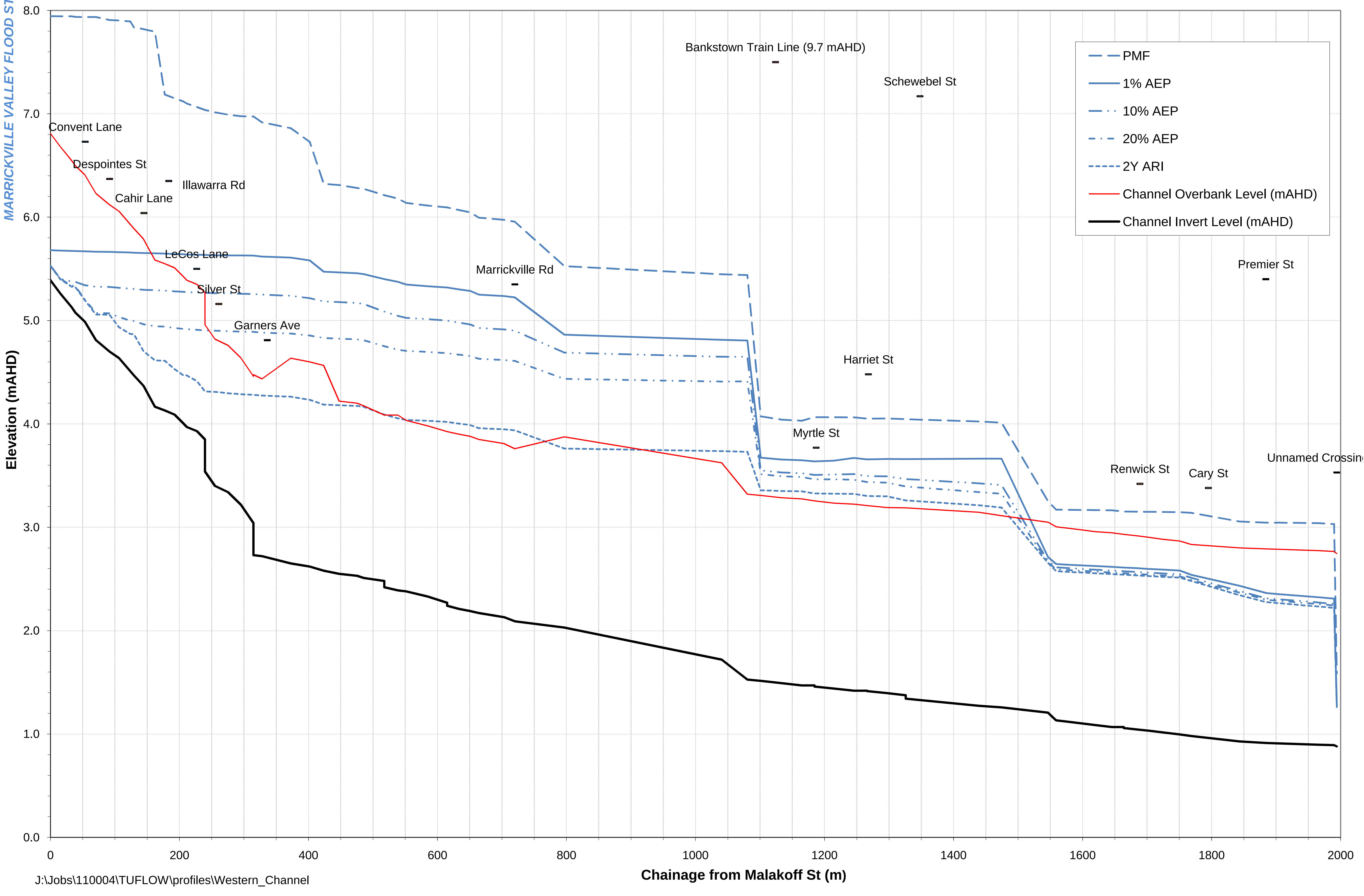
J:\Jobs\110004\TUFLOW\profiles\Eastern_Channel

FIGURE 15
PEAK FLOOD PROFILES - CENTRALCHANNEL



J:\Jobs\110004\TUFLOW\profiles\Central_channel

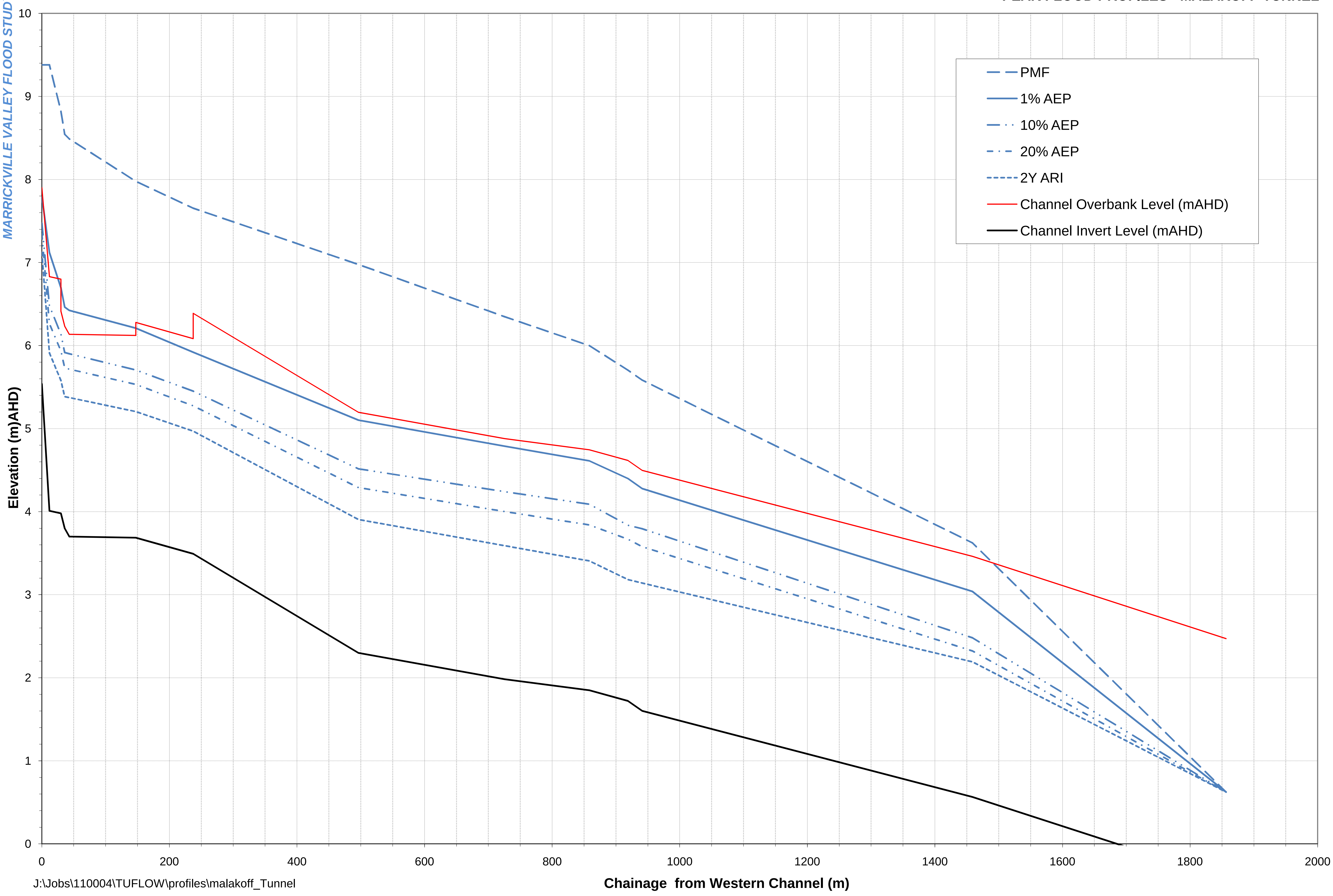
FIGURE 16
PEAK FLOOD PROFILES - WESTERN CHANNEL



J:\Jobs\110004\TUFLOW\profiles\Western_Channel

FIGURE 17

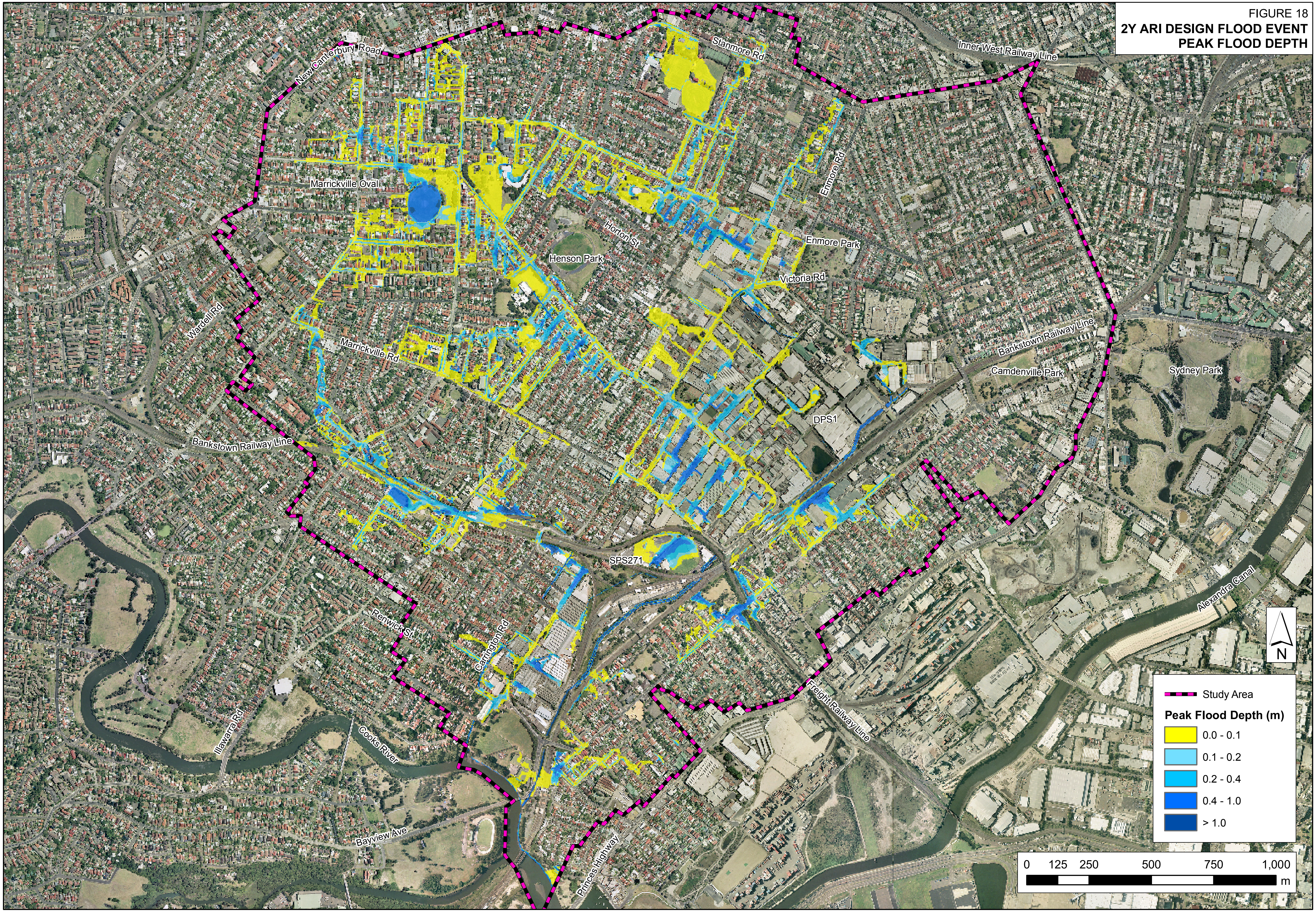
PEAK FLOOD PROFILES - MALAKOFF TUNNEL



J:\Jobs\110004\TUFLOW\profiles\malakoff_Tunnel

MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 18
2Y ARI DESIGN FLOOD EVENT PEAK FLOOD DEPTH



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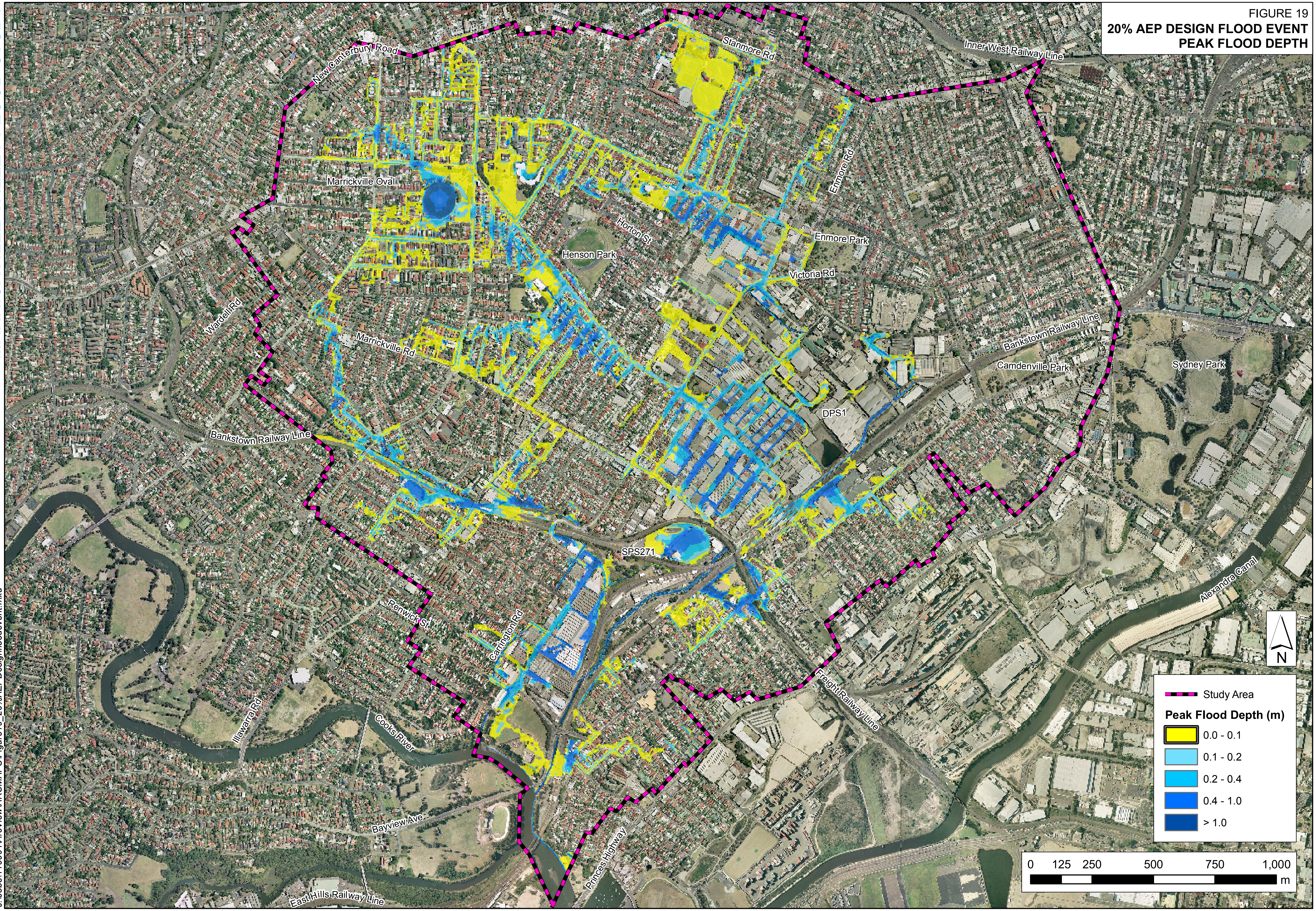

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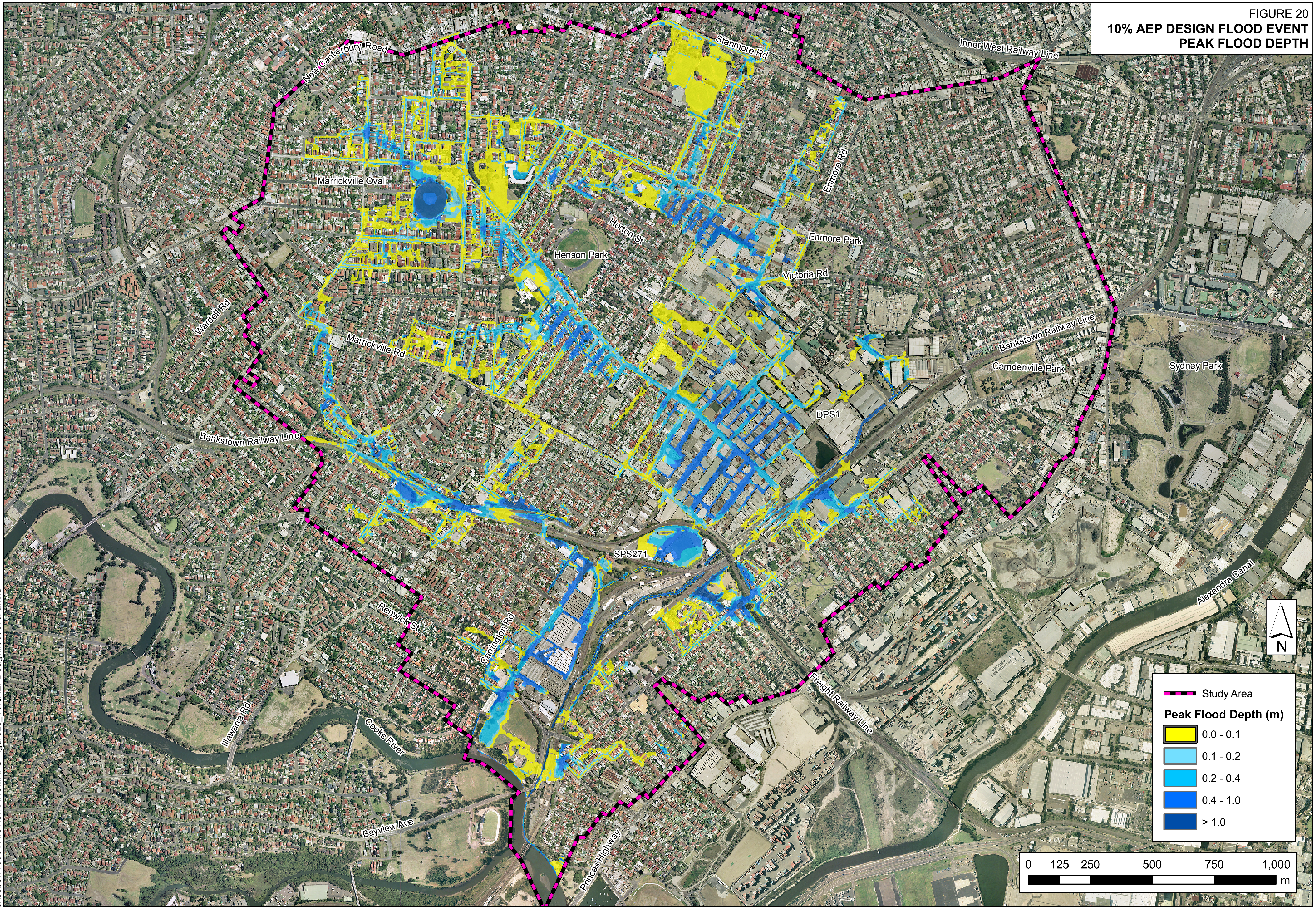

FIGURE 20
10% AEP DESIGN FLOOD EVENT
PEAK FLOOD DEPTH

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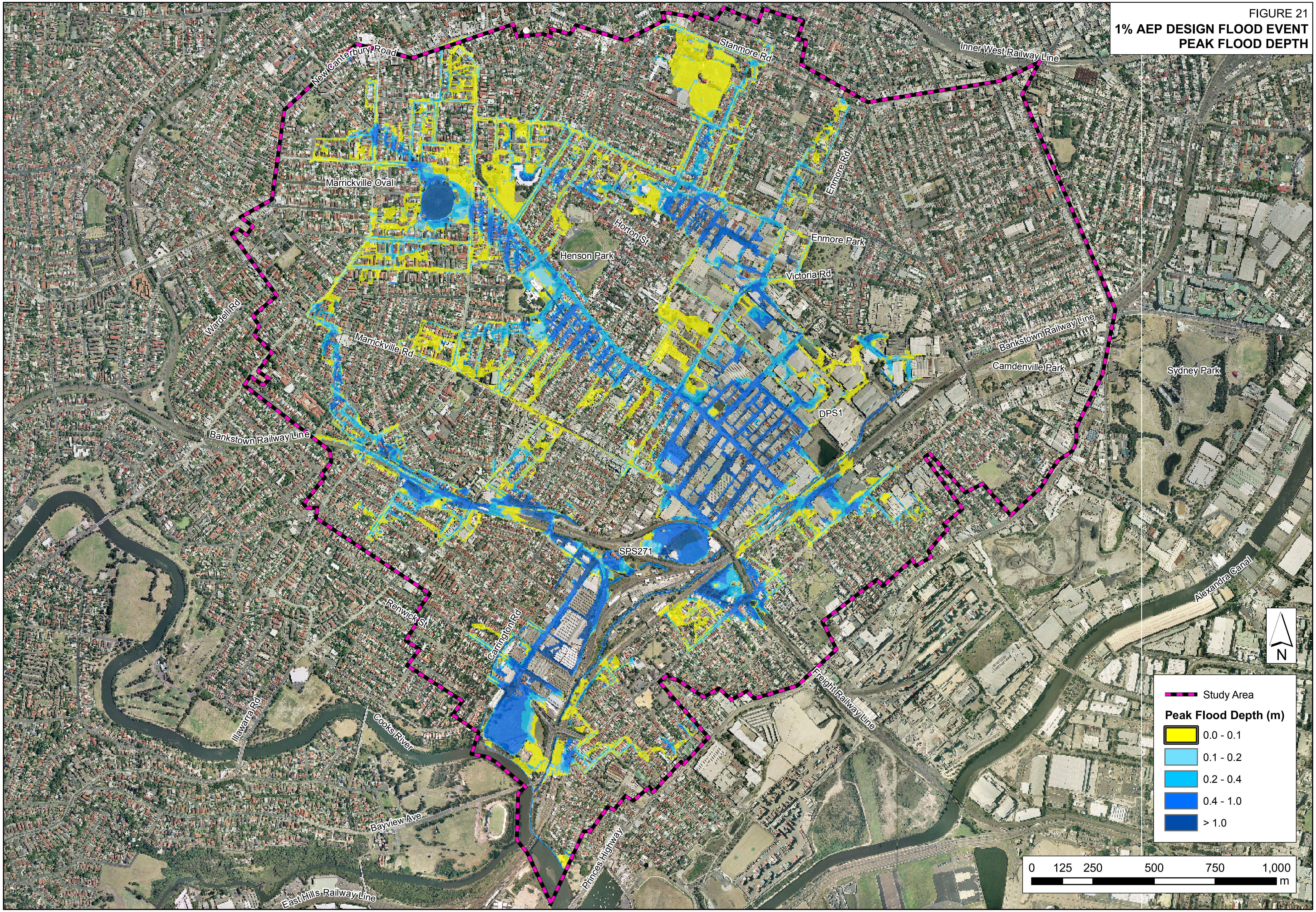
FIGURE 21
1% AEP DESIGN FLOOD EVENT
PEAK FLOOD DEPTH



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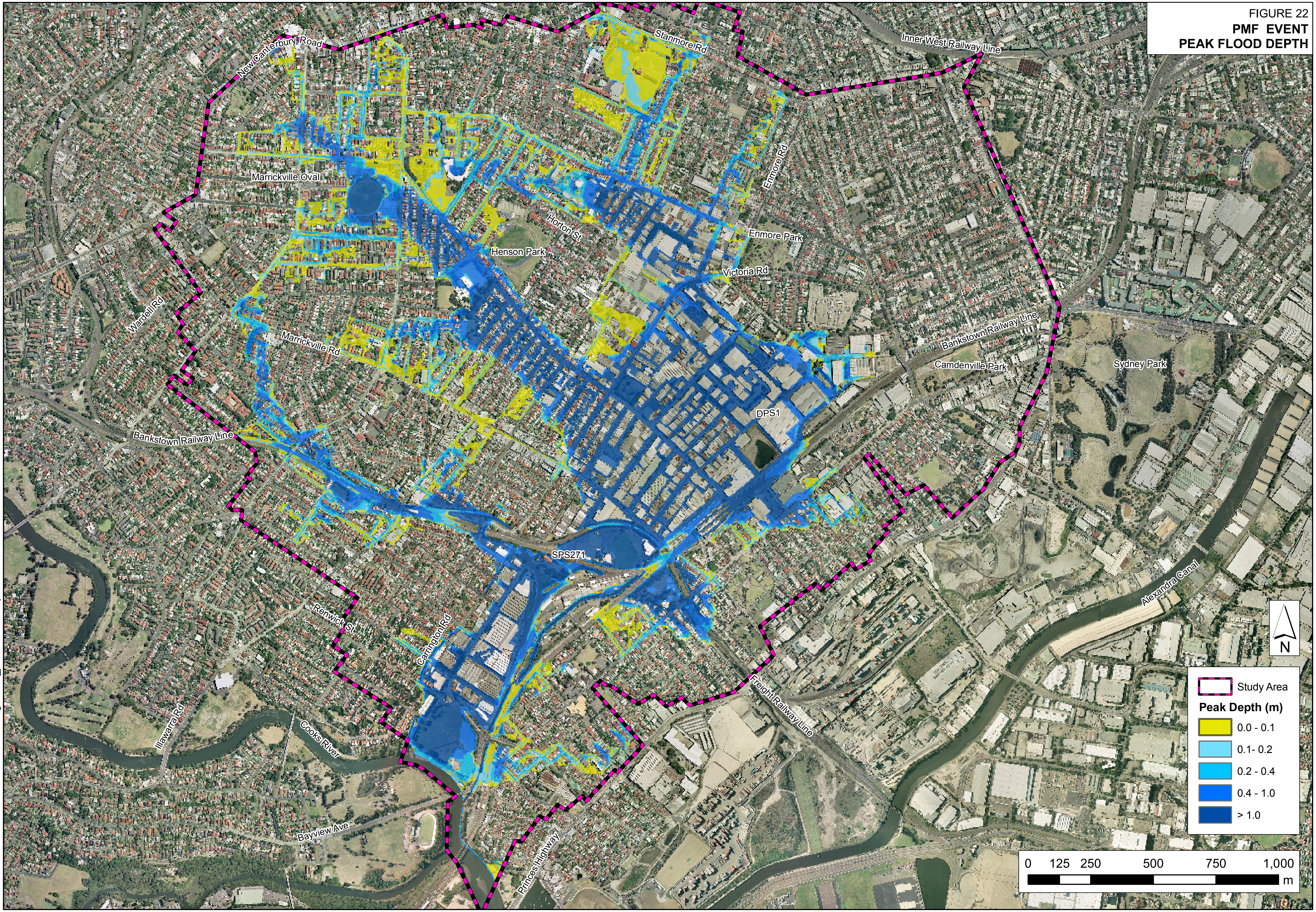

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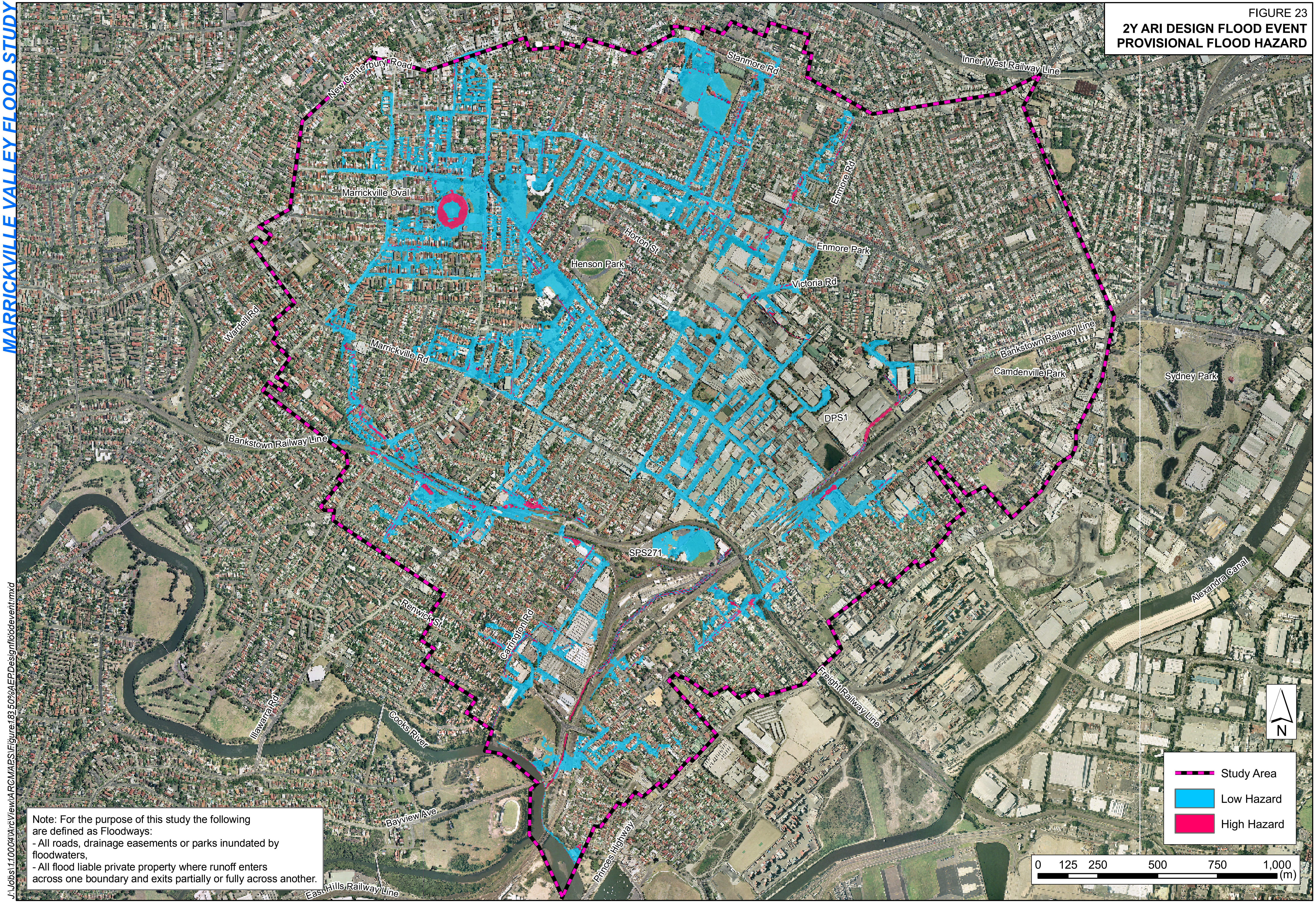
MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 23
2Y ARI DESIGN FLOOD EVENT
PROVISIONAL FLOOD HAZARD



Note: For the purpose of this study the following are defined as Floodways:
- All roads, drainage easements or parks inundated by floodwaters,
- All flood liable private property where runoff enters across one boundary and exits partially or fully across another.

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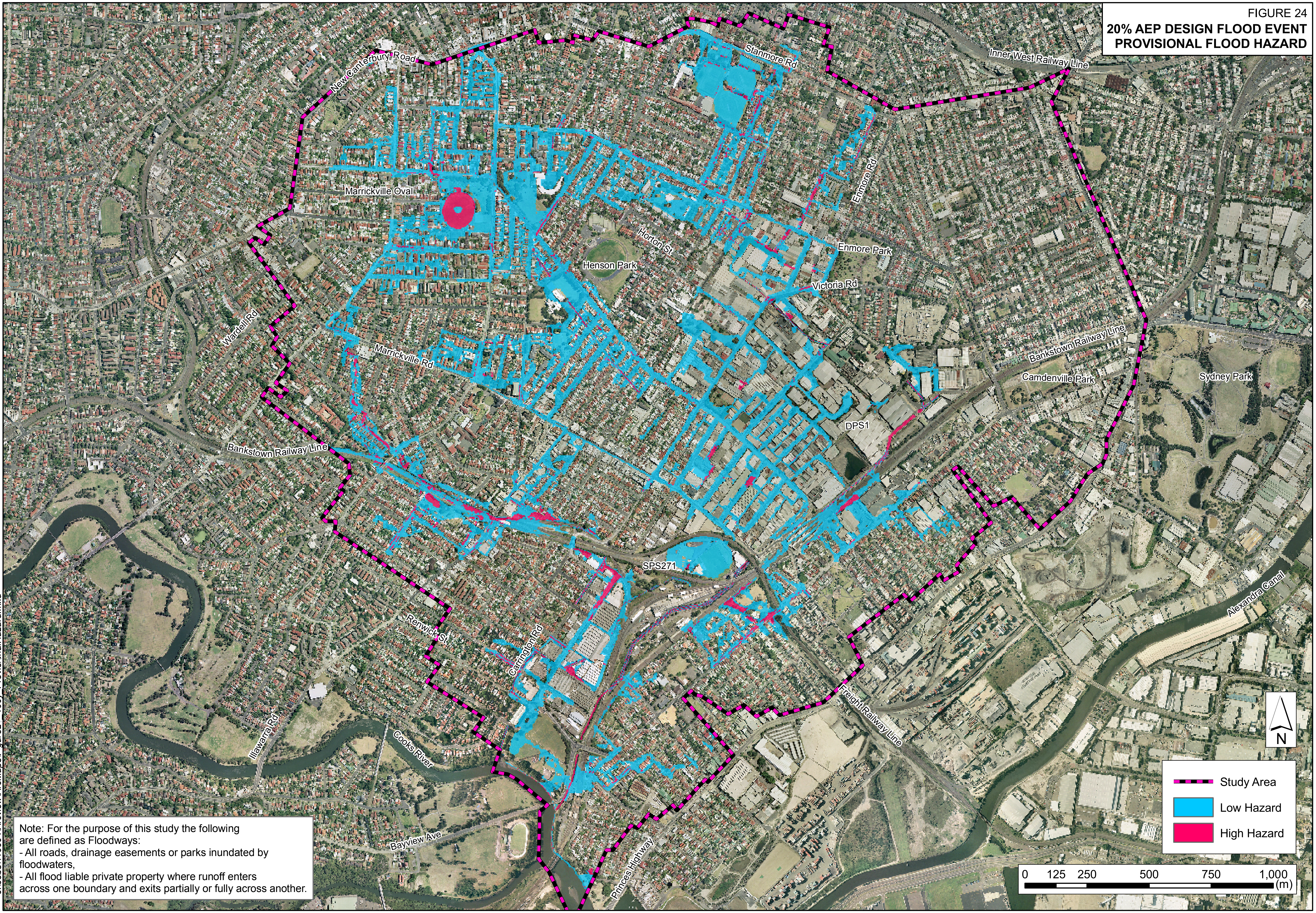
FIGURE 24
20% AEP DESIGN FLOOD EVENT
PROVISIONAL FLOOD HAZARD



Note: For the purpose of this study the following are defined as Floodways:
- All roads, drainage easements or parks inundated by floodwaters,
- All flood liable private property where runoff enters across one boundary and exits partially or fully across another.

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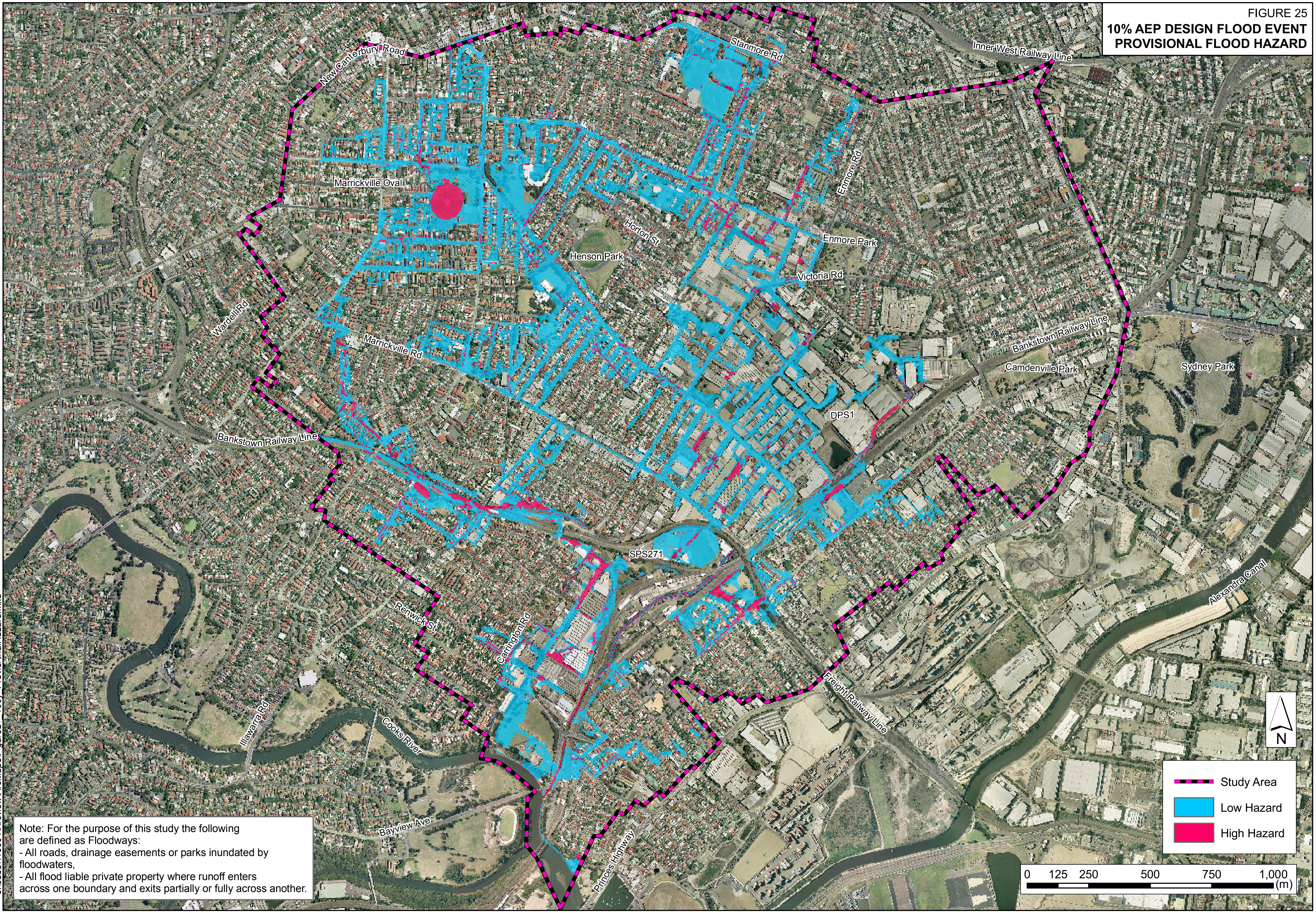
MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 25
10% AEP DESIGN FLOOD EVENT
PROVISIONAL FLOOD HAZARD



Note: For the purpose of this study the following are defined as Floodways:
- All roads, drainage easements or parks inundated by floodwaters,
- All flood liable private property where runoff enters across one boundary and exits partially or fully across another.

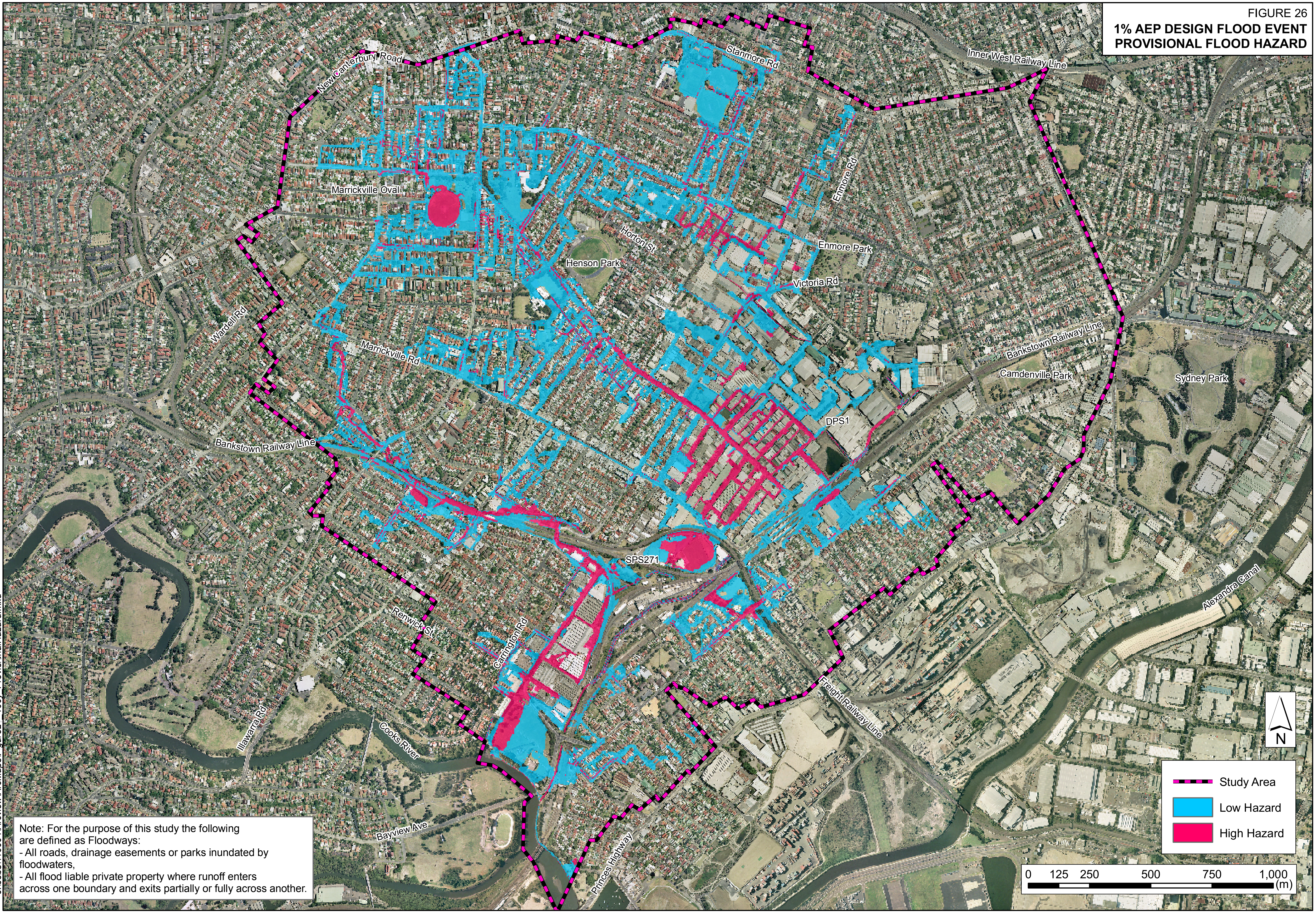
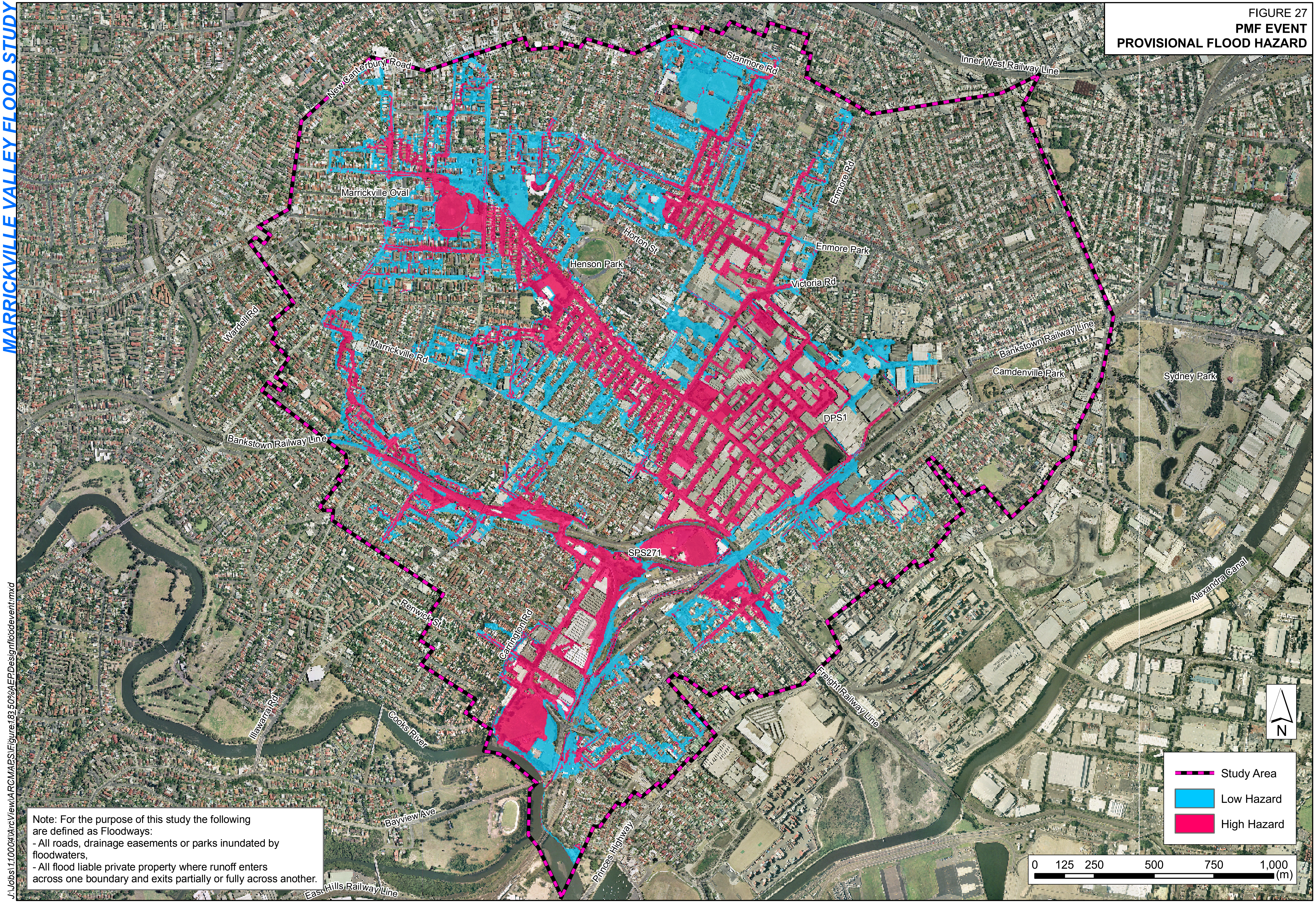


FIGURE 26
1% AEP DESIGN FLOOD EVENT
PROVISIONAL FLOOD HAZARD
New Canterbury Road
Stanmore Rd
Inner West Railway Line
Enmore Rd
Marrickville Oval
Horton St
Enmore Park
Henson Park
Victoria Rd
Wardell Rd
Marrickville Rd
Bankstown Railway Line
Camdenville Park
Sydney Park
DPS1
SPS271
Renwick St
Carrington Rd
Illawarra Rd
Cooks River
Freight Railway Line
Alexandra Canal
Bayview Ave
Princes Highway
N
Study Area
Low Hazard
High Hazard
0 125 250 500 750 1,000
(m)
Note: For the purpose of this study the following
are defined as Floodways:
- All roads, drainage easements or parks inundated by
floodwaters,
- All flood liable private property where runoff enters
across one boundary and exits partially or fully across another.

FIGURE 27
PMF EVENT
PROVISIONAL FLOOD HAZARD



Note: For the purpose of this study the following are defined as Floodways:
- All roads, drainage easements or parks inundated by floodwaters,
- All flood liable private property where runoff enters across one boundary and exits partially or fully across another.

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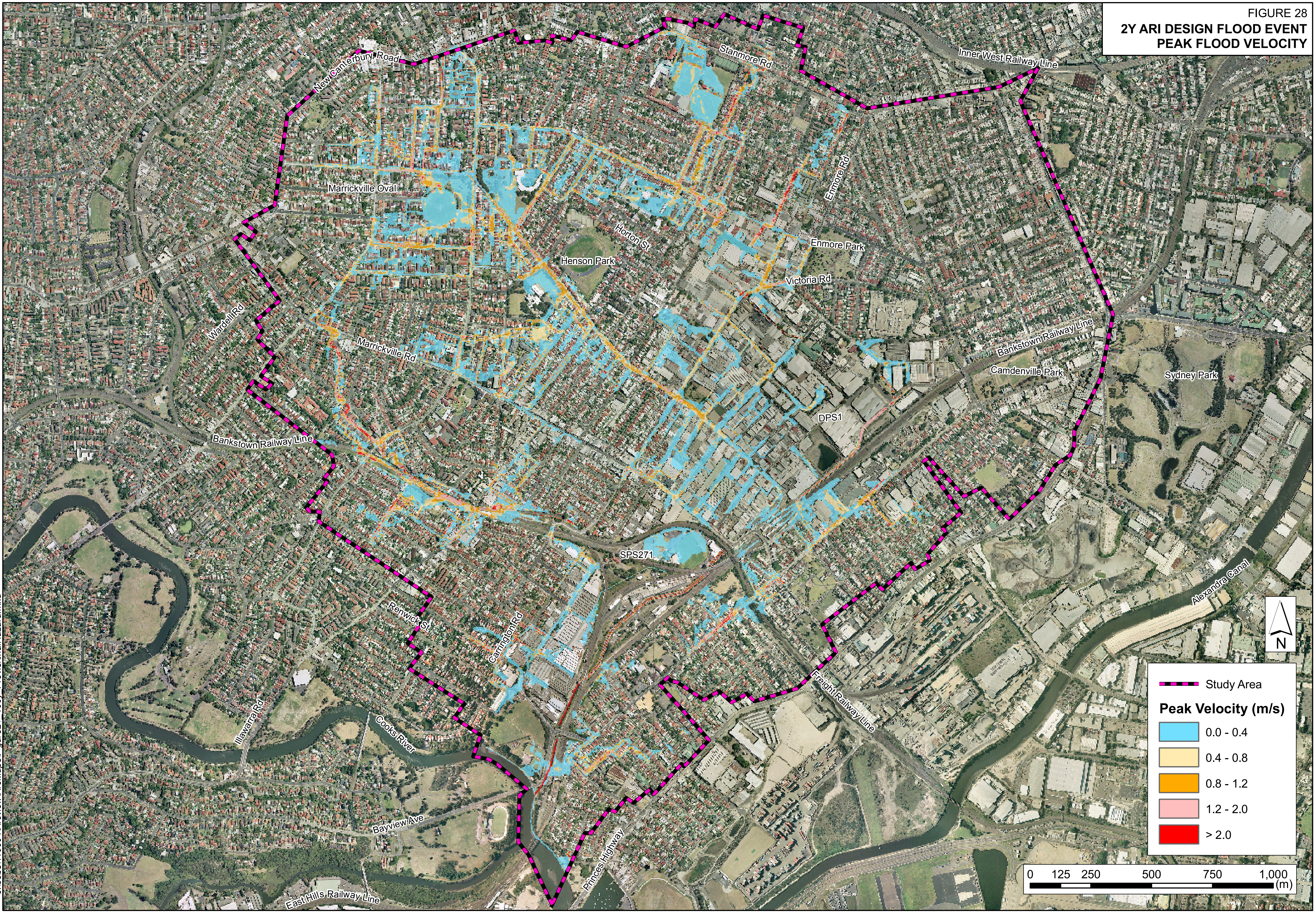

FIGURE 28
2Y ARI DESIGN FLOOD EVENT
PEAK FLOOD VELOCITY

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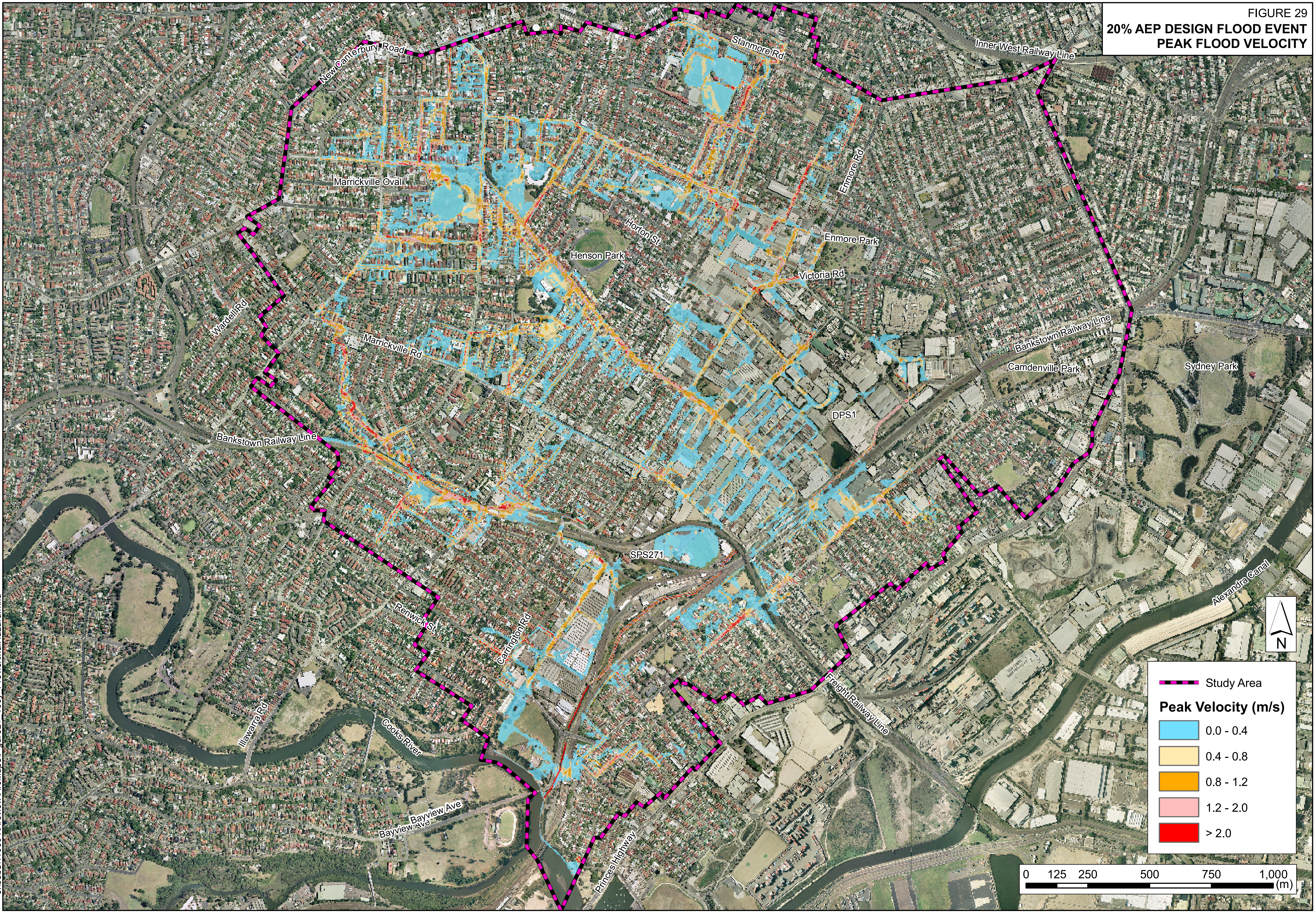
FIGURE 29
20% AEP DESIGN FLOOD EVENT
PEAK FLOOD VELOCITY



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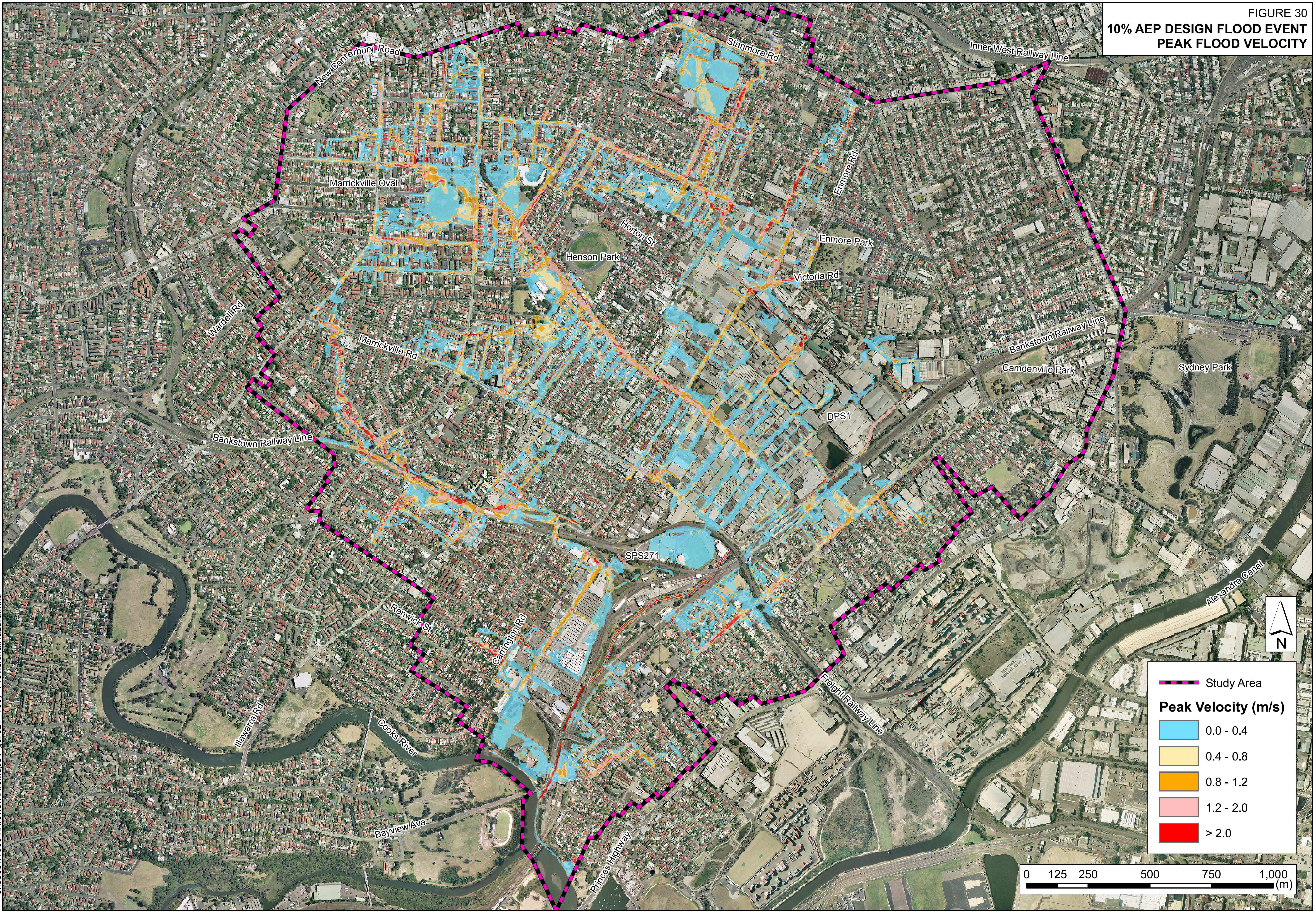
MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 30
10% AEP DESIGN FLOOD EVENT
PEAK FLOOD VELOCITY



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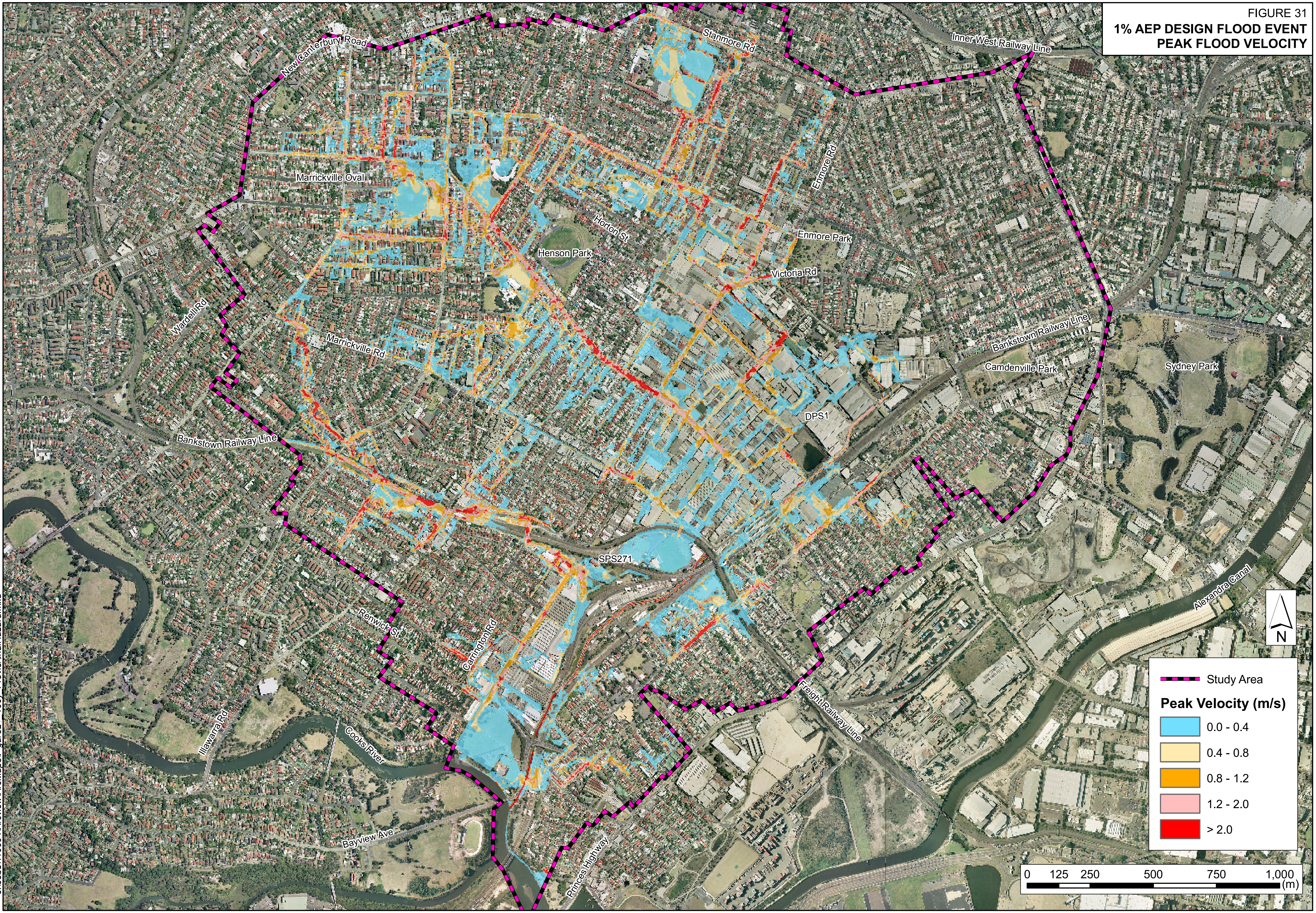
MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 31
1% AEP DESIGN FLOOD EVENT
PEAK FLOOD VELOCITY



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MARRICKVILLE VALLEY FLOOD STUDY

FIGURE 32
PMF EVENT
PEAK FLOOD VELOCITY

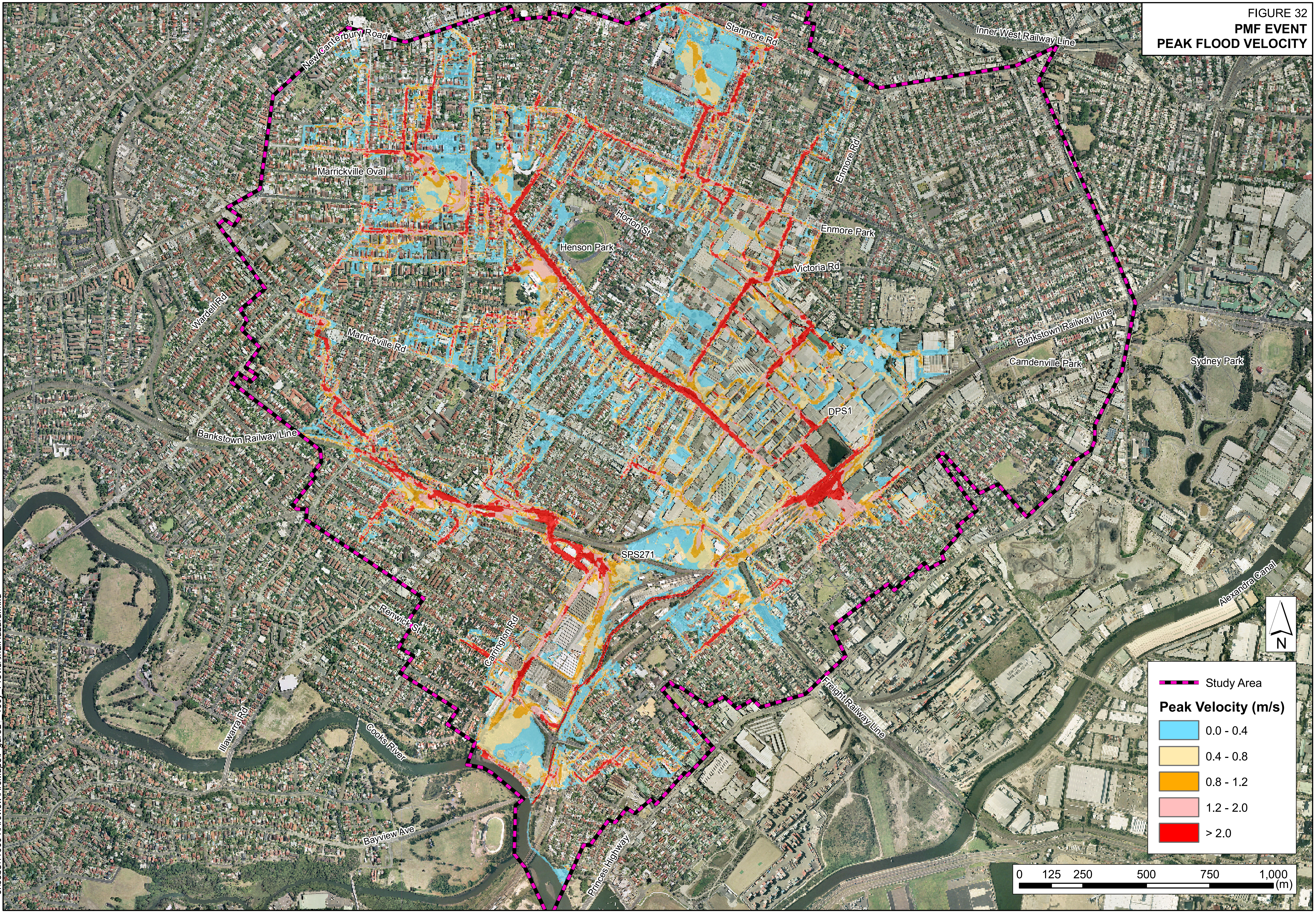

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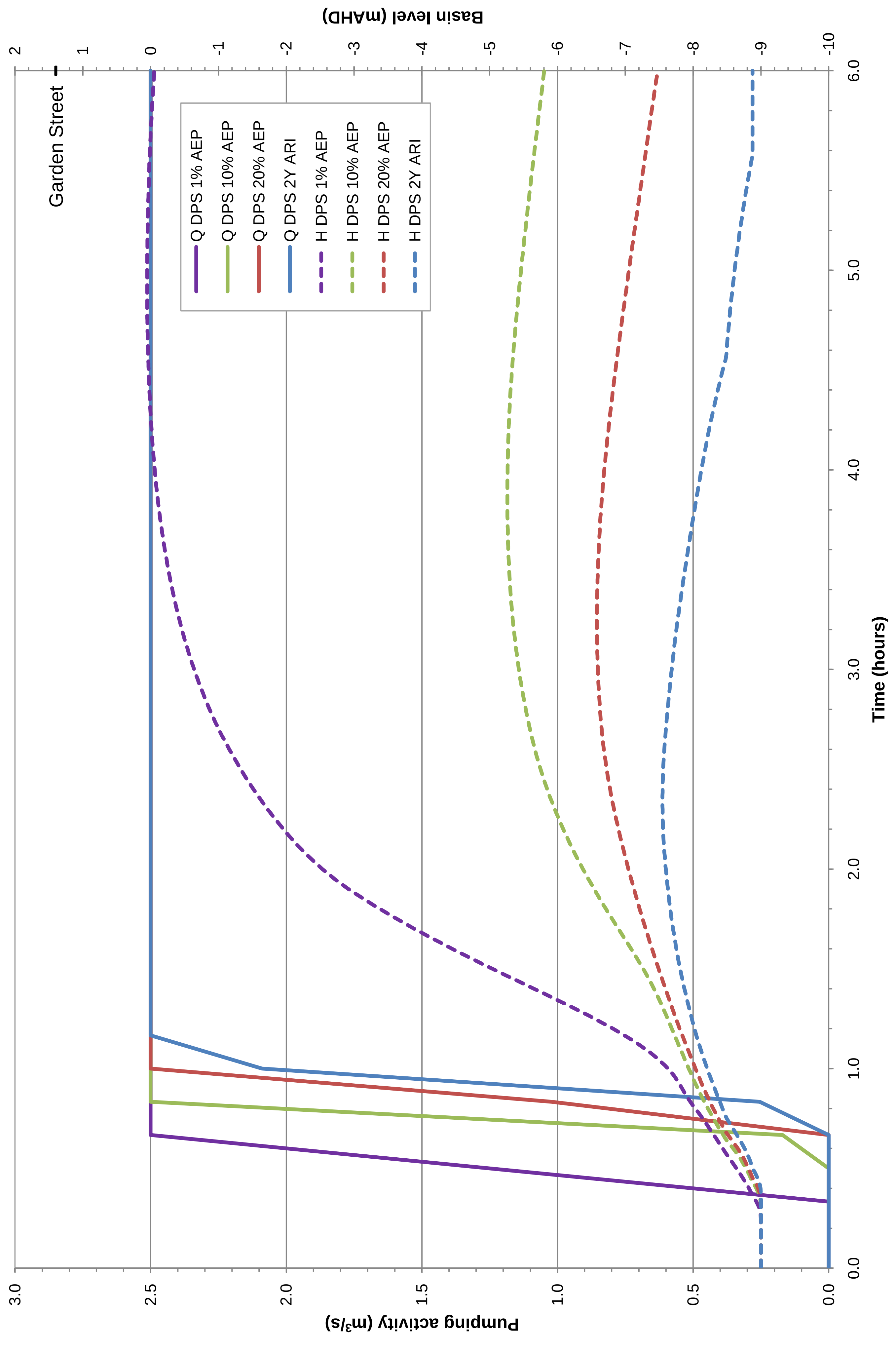
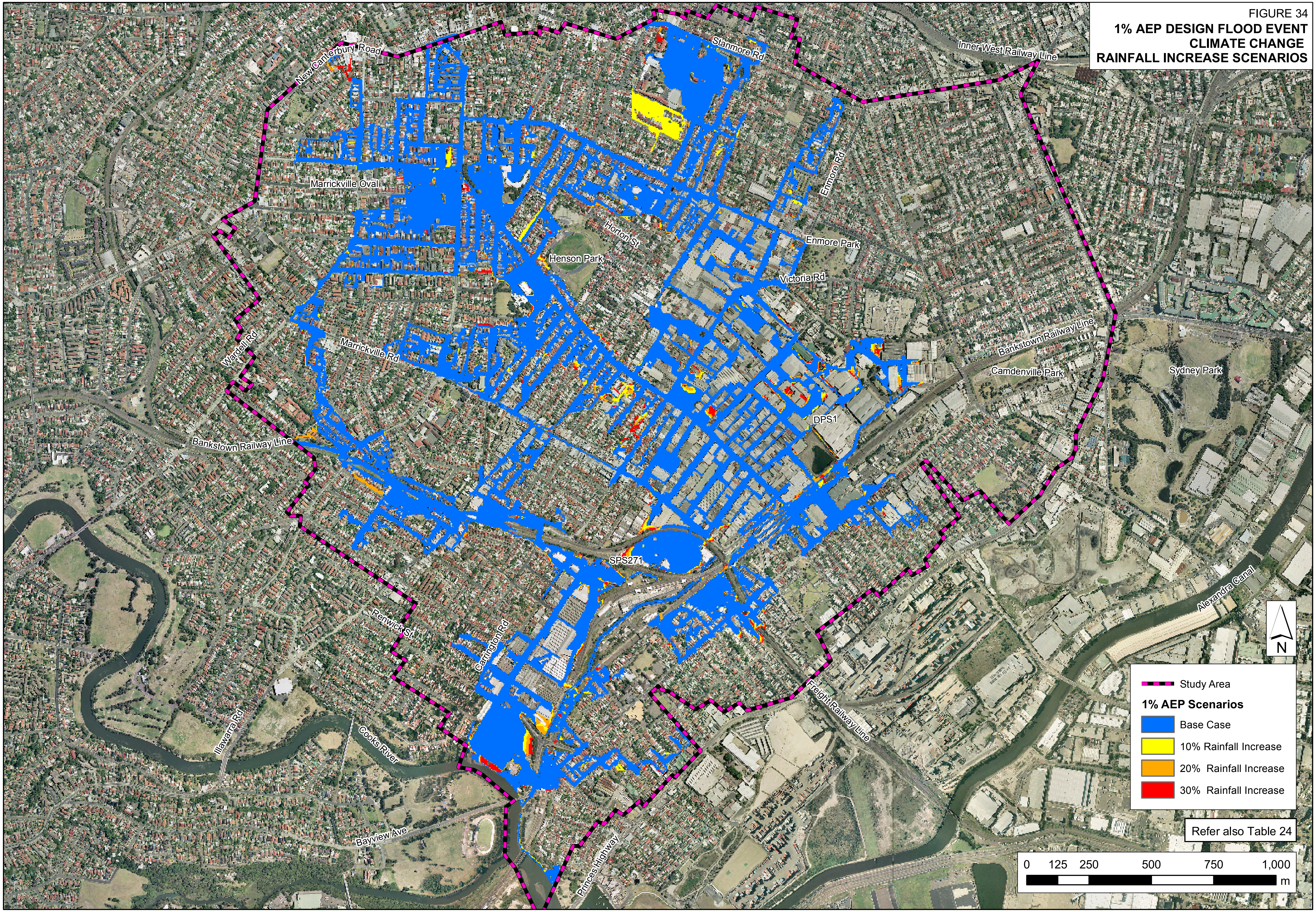
FIGURE 33
PUMPING DISCHARGE/LEVEL FOR SYDENHAM RETARDING BASIN (DPS1)





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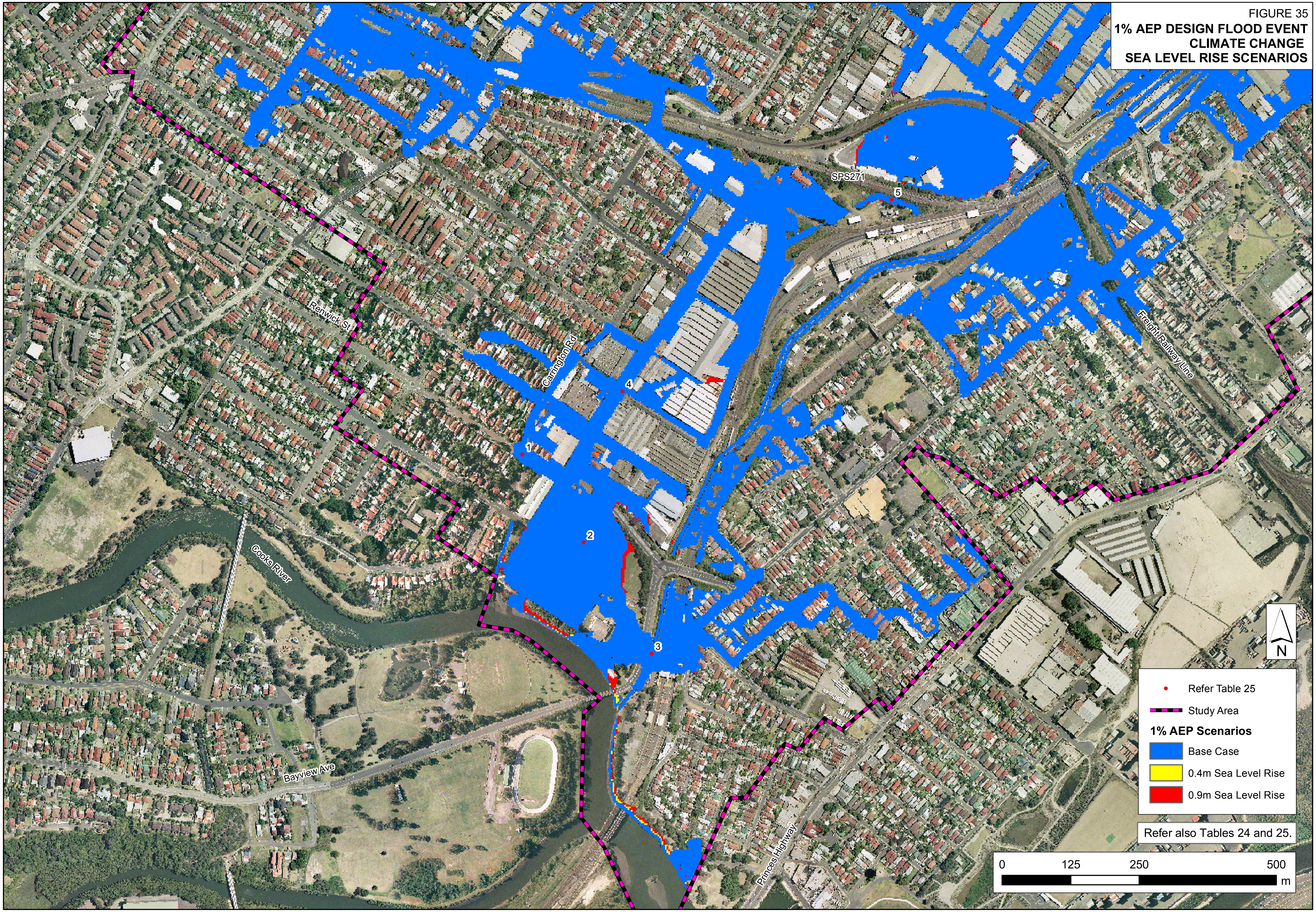
MARRICKVILLE VALLEY FLOOD STUDY



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