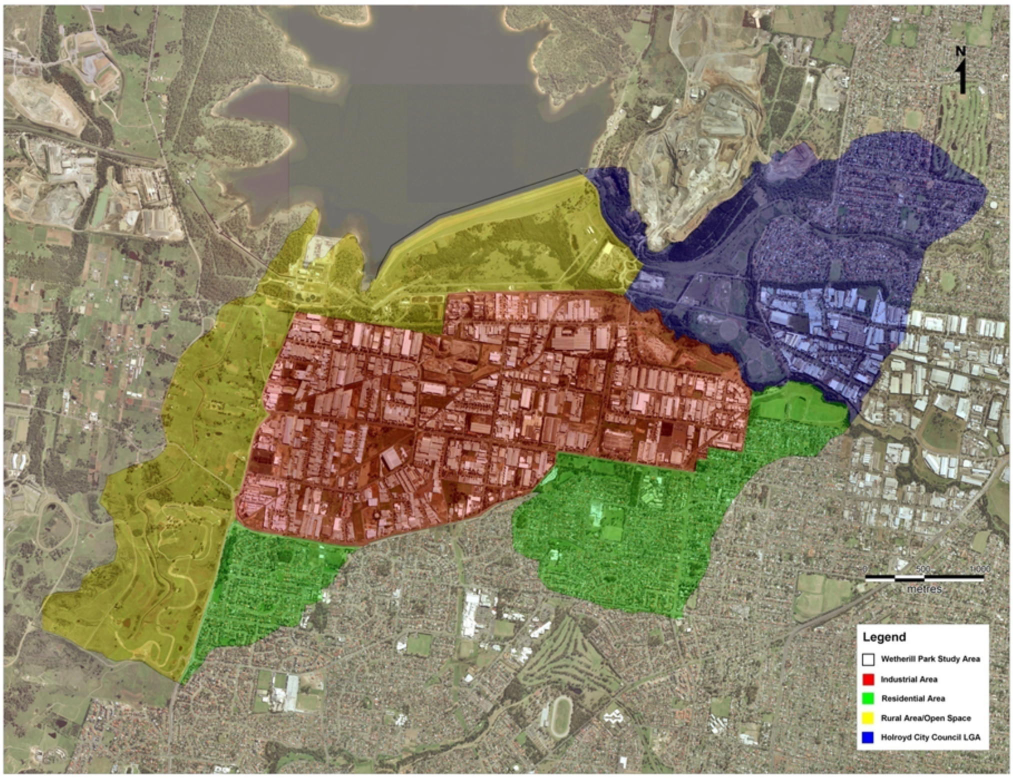
Wetherill Park Overland Flood Study



FINAL REPORT

- Version 2
- February 2015

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Executive Summary

The local government area of Fairfield City is crossed by several major creeks, all of which are prone to mainstream flooding. In addition, parts of Fairfield City are at risk of overland flooding from stormwater that runs off from urbanised catchments to the creeks. Both types of flooding present a significant risk to life and property.

In order to address and mitigate this flood risk, Fairfield City Council is following the NSW Government's 2005 Floodplain Development Manual. The manual outlines a floodplain risk management process, leading to the preparation and implementation of a floodplain risk management plan. Plans are to be prepared for both mainstream and local overland flooding.

A preliminary assessment of the risk of flooding from overland flows within the urban areas of Fairfield was undertaken in 2003-2004 as part of the *Fairfield City Overland Flood Study*. This study prioritised the 18 urban subcatchments for more detailed investigation. The Wetherill Park catchment, which is covers the suburb of Wetherill Park as well as parts of Smithfield and Bossley Park, was chosen as the next catchment for analysis.

The Wetherill Park Overland Flood Study was undertaken as an in-house project by Council. The key objectives of the study were to describe the nature and extent of overland flooding within the subcatchment and to prepare flood risk precinct maps for several events including the Probable Maximum Flood (PMF). This study would then provide the basis for preparing a floodplain risk management study and plan that would identify and recommend a range of measures to reduce the risk of overland flooding.

The methodology for undertaking the study was drawn from the *Canley Corridor Overland Flood Study*, completed in 2009. Modelling of the major trunk drainage network as well as selected 'trouble spots' was found to be the most efficient method of producing reliable results.

The Wetherill Park overland flow catchment is located in the north-western portion of the Fairfield LGA, immediately south of the Prospect Reservoir and Prospect Creek and east of the Sydney Water Supply Channel. Encompassing the suburb of Wetherill Park and parts of Smithfield and Bossley Park, the model covers a total of 1911.7 hectares, including an area of 428.5 hectares within Holroyd City Council. Since it was intended to use the Direct Rainfall Method, it was necessary to include the entire catchment area within the model.

The study area contains a diverse range of land uses. The largest of which is the Wetherill Park Industrial Estate, which covers 644.3 hectares or 43.4% of the study area within the Fairfield LGA. The next largest component is rural/open space, which covers 509.3 hectares or 34.3% of the study area within the Fairfield LGA and incorporates the suburb of Horsley Park and the Western Sydney Regional Parklands. The smallest component is residential, which covers 329.6 hectares or 22.2% of the study area within the Fairfield LGA and is comprised mostly of single dwelling houses.

The catchment drains in a generally north-easterly direction into Prospect Creek. The study area is traversed by three concrete-lined trapezoidal open channels totalling over 3.8km in length.

The longest of these, known in this study as the "Main Channel" covers a length of 1655 metres, from the box culverts underneath the Cowpasture Road/The Horsley Drive roundabout to its outlet into Prospect Creek near Widemere Road. The next longest channel, known as "Tributary 1" covers a length of 1380 metres, from the culvert outlet at Potter Close to its confluence with the Main Channel. The shortest channel, known as the "Rosford Channel" is of length of 775 metres from Victoria Street to Hassall Street. All three channels are trapezoidal in profile with 2 in 3 side slopes and with gradients generally between 0.5% and 1.0%.

Further drainage infrastructure in the study area includes a small detention basin located in the Emerson Street Reserve and an extensive pipe drainage network. The network generally follows the overland flow paths in the study area, usually draining into either one of the concrete-lined open channels or directly into Prospect Creek.

The reach of Prospect Creek within the study area includes its outlet from Prospect Reservoir, the bridge crossing at Widemere Road, the Hassall Street detention basin, the bridge crossing at Gipps Road and the Rosford Street detention basin.

A hydraulic model of the catchment was developed in the hydrodynamic modelling package TUFLOW. The model was set up with the open channels and underground stormwater drainage represented as a 1D stream network nested in a 2D domain to accurately represent the in-channel and pipe hydraulics as well as two-dimensional overland flow patterns on the floodplain.

The three concrete-lined trapezoidal open channels described above, along with their associated bridge and culvert crossings were incorporated into the 1D network of the model. Railings on culvert and bridge crossings were assumed to be fully blocked if the spacing between the bars was less than 150mm, and unblocked if the spacing was greater than 150mm. Fence lines parallel to open channels were excluded from the model.

Prospect Creek was represented in the 2D domain and not explicitly modelled as a 1D reach like the three trapezoidal channels.

The modelled stormwater pit and pipe network was comprised of pipes with a diameter of 900mm and greater and their associated pits, with smaller pipes included as necessary at the known trouble spots to represent these locations in more detail. The model contains a total of 632 lengths of stormwater pipe and a total of 388 inlet pits, comprising 285 kerb inlet pits and 103 grated surface or letterbox inlets. A further 242 junction pits were included in the model.

Buildings were represented in the floodplain by the use of a depth-varying value of hydraulic roughness (Manning's *n*). Across building footprints, a low value of hydraulic roughness was applied for shallow depths to account for roofs and hardstand surfaces that are directly connected to surface and sub-surface drainage paths. Once depths increase, the effects of obstructions due to buildings are accounted for with much higher values of hydraulic roughness.

The Direct Rainfall method was used in this study. Rainfall depths were input to the model in 5 minute increments, and a uniform spatial distribution assumed for calibration and design events. A constant, nominal outflow from Prospect Reservoir was assumed as the upstream boundary on Prospect Creek and a stage-discharge relation was used as the downstream boundary, at a location approximately 70 metres downstream of the outlet of the Rosford Street detention basin.

The TUFLOW model was stabilised by making a number of minor modifications in the 1d domain, such as the breakup of certain long network reaches and the addition of small volumes of additional storage to certain model nodes. The mass balance error in model runs for the 100 year ARI event and up to the PMF were under +/-1% which is considered acceptable.

Detailed model calibration and verification could not be carried out since direct measurements of flood levels in the catchment were not available for historical flood events.

Rudimentary model calibration was carried out to high water marks observed following a moderate flood event which occurred on the 18th April 2012. Analysis of gauge data at the three locations closest to the study area indicated that this was best described as a 30 to 45 minute duration event, with the approximated ARI ranging from between 10 and 50 years.

Calibration runs were carried out using an average of the recorded rain at each of the three gauging stations. The modelled water levels were generally found to be within 200mm of the recorded high water levels, indicating a satisfactory model calibration.

Since detailed calibration and verification could not be carried out for this study, a comparison with an independent hydrologic model was made for a few selected sub-catchments in the study area. Good agreement was achieved with respect to the magnitude and timing of the discharge peaks and reasonable agreement between hydrograph volumes, indicated that no adjustment of input parameters to the TUFLOW model was necessary.

Sensitivity analyses revealed that the TUFLOW model was not sensitive to changes in floodplain roughness, downstream tailwater conditions, varied rainfall losses or the degree of drainage pit blockage. Hence, overland flow estimates are not expected to be significantly impacted by uncertainties in these parameters.

The flood models were run for the 20, 100, 500, 10,000 year Average Recurrence Interval (ARI) events and the Probable Maximum Flood (PMF), for a range of storm durations from 25 minutes to six hours. The peak water level and velocity for each storm duration at each 2D grid point were extracted and used to form a ‘peak of peaks’ grid that was subsequently used a basis for the flood mapping. Areas of nuisance or localised flooding less than 150 mm in depth were manually removed from the flood mapping.

Flood model results and the flood mapping indicate that:

- The three concrete-lined open channels can largely convey the 1:100 year ARI flow within their banks for much of their lengths, with the exception of surcharging at the inlets of certain culverts.
- In the residential area south of The Horsley Drive, overland flow paths generally follow the road network for the 1:100 year ARI event, with some flooding to residential properties most notably at Mulgara Place and at the northern end of Marconi Road.
- In the industrial area upstream of the Tributary 1 confluence, considerable surcharging of flow was evident at the inlets of the Toohey Road culverts and the Victoria Street/Newton Road culverts for the 1:100 year ARI events, causing overflow on roads and the inundation of adjacent industrial properties.
- In the industrial area downstream of the Tributary 1 confluence, a number of overland flowpaths were noted in the 1:100 year ARI event, including a path for much of the length of Davis Road from its western end to its crossing with the Main Channel. Flow depths were generally between 0.3m and 0.5m for the 1:100 year ARI event, with some inundation to

adjacent industrial properties, particularly at the eastern end of the flowpath near the Main Channel.

- In the industrial area draining to Tributary 1, an overland flowpath was noted in the 1:100 year ARI event, originating from near the northern end of Cowpasture Road and traversing through a number of industrial properties before draining to Tributary 1 near Coates Road. Depths for the 1:100 year ARI event were generally below 0.3m, except for a few locations where depths upwards of 0.5m occur.
- In the residential area south of Victoria Street (draining into the Rosford Channel), numerous overland flowpaths were noted in the 1:100 year ARI event, resulting in inundation to residential properties for the 1:100 year ARI event most notably between Shakespeare Street and the Rossetti Street/Mansfield Street intersection, between Maugham Crescent and Ainsworth Crescent, between Ainsworth Crescent and Emerson Street, between Campion Street and The Horsley Drive and along Haywood Close.
- The Emerson Street basin can contain flows for floods up to and including the 1:500 year ARI events, but overtops for the 1:10,000 year and PMF events.
- In the residential area north of Victoria Street (draining into the Rosford Channel), a number of overland flowpaths were noted in the 1:100 year ARI event, including a path traversing east along the north-east section of Hassall Street, with inundation to numerous industrial properties along Hassall Street, Blackstone Street and Lennox Place. Also, considerable surcharging was noted at the inlet of the Hassall Street culverts, resulting in flow breakouts and inundation of industrial properties on both banks. Flow depths of up to 0.5m occurred for the 1:100 year ARI event.

Flood risk precinct maps were prepared based on modelling of the 100 year ARI and PMF events and using the flood risk precinct categories outlined in the Fairfield City-Wide Development Control Plan. This flood mapping was updated after a review by Cardno Pty Ltd in 2014. The updated flood risk precinct mapping has identified:

- Approximately 1489 properties are within the floodplain outline defined by the PMF event. This includes:
  - 138 parcels in the high (or partially high) risk precinct;
  - 431 parcels in the medium (or partially medium) risk precinct;
  - 920 parcels in the low (or partially low) risk precinct.
- Areas of high flood risk occur in numerous separate locations in the study area, including:
  - Along the lengths of all three open channels, and in the natural gullies draining from the rural area to the Main Channel and Tributary 1;
  - In the natural gully parallel to Cowpasture Road;
  - In the Emerson Street Basin and,
  - on sections of Wordsworth Street, Ainsworth Crescent and Haywood Close;
- In the rural areas, the medium flood risk precinct follows the numerous natural gullies and drainage flow-paths leading to the inlets of the Main Channel and Tributary 1.
- In the urban and industrial areas, the medium flood risk precinct largely follows the pattern of the underground drainage system.

- The low flood risk precinct follows the outline of the medium flood risk precinct reasonably closely in most parts of the rural areas, owing to the relatively steep gullies and channels
- For the remainder of the study area, the low flood risk precinct widens considerably from the outline of the medium flood risk precinct, and numerous additional overland flow paths and inundated areas appear. This is particularly pronounced in the industrial zone, where extensive flood-prone areas are noted around the Victoria Street/Newton Road roundabout and along Davis Road and Redfern Road.

The flood risk precinct maps only represent overland flooding from storm water runoff within the Wetherill Park catchment. They do not cover areas in the lower catchment which are at risk of mainstream flooding from Prospect Creek. Mainstream flood extents for Prospect Creek are reported in the 2006 *Prospect Creek Floodplain Management Plan, Flood Study Review*.

It is considered that the study has ultimately provided a good foundation from which to prepare the Wetherill Park Floodplain risk Management Study and Plan as the next step in the floodplain risk management process.

In 2014 Fairfield City Council engaged Cardno Pty Ltd through the NSW Local Government Procurement Panel to create a Catchment Management Plan for the Wetherill Park catchment. As part of the process, Cardno undertook a review of the existing hydraulic model and has recommended minor refinements. These refinements have resulted in slightly altered depth and velocity maps, which has consequently produced updated flood precinct mapping. This revised mapping is included within this report and has replaced the initial mapping that was created. The Hydraulic Model Update Summary Report produced by Cardno dated 8 January 2015 is included within this report as Appendix I.

1. Introduction

1.1. Background

The Local Government Area (LGA) of Fairfield City covers an area of around 102.5 km^2 and is located on a number of floodplains. These floodplains comprise the low-lying land next to the Georges River and the city's eight major creeks. These creeks span over 80km in length and flow into both the Georges River and Hawkesbury-Nepean catchments. Being within a floodplain means that many suburbs in the LGA are prone to flooding.

In addition to the city's creeks, there are a number of watercourses and tributaries throughout the LGA that have been piped over the years, especially in the period between post-World War II and the 1970s, as part of the increasing urbanisation. Most of these piped flow paths are in urban areas. This gives rise to the potential for damage to properties and hazard to residents due to flooding.

Flooding in Fairfield LGA can occur in two different ways. These are mainstream flooding and local overland flooding. Mainstream flooding is the inundation of normally dry land due to flood waters overflowing the natural or artificial banks of a stream, river, estuary, lake or dam. Conversely, local overland flooding is the inundation caused by local runoff during heavy storms, usually from stormwater pits and pipes which have exceeded their capacities, rather than overbank discharge. Overland flows eventually end up in the local creek system.

Both types of flooding can cause significant damage. For example, major mainstream flooding occurred along lower Prospect Creek and Cabramatta Creek in August 1986 and April-May 1988. The 1986 flood caused an estimated total damage of $4.8 million. A smaller flood in January 2001 caused damage to the upper reach of Prospect Creek.

In addition, there are different scales of local flooding. At the lower end of the scale, minor flooding may result from a number of sources including blockage of drainage pits and pipes. At the upper end of the scale, major flooding can occur due to water flowing along natural floodways or across land due to the runoff exceeding the capacity of the trunk drainage system.

To mitigate the risk of flooding the NSW Government has adopted the Flood Prone Land Policy, as outlined in the 2005 NSW Floodplain Development Manual (FDM). The FDM describes the process by which Councils can undertake flood studies and prepare floodplain risk management studies and plans.

In accordance with the floodplain risk management process, Council has prepared a number of flood studies for both mainstream and overland flooding, as well as floodplain risk management plans for the Georges River and Cabramatta Creek. Council is also in the process of adopting a floodplain management plan for Prospect Creek. Eventually, flood studies and floodplain risk management plans will be prepared for all the city's sub-catchments for both mainstream and overland flooding. The plans detail a range of flood modification, property modification and emergency response measures that can be used to reduce flood risk. This may include voluntary house raising, vegetation management of the creeks, the construction of detention basins and floodways and implementation of development controls. Development controls are outlined in Council's City Wide Development Control Plan (DCP).

In the past, FCC concentrated primarily on studying mainstream flooding from the city's creeks as this was considered to be the main source of flood risk in the LGA. However, flooding from major overland flow paths and the resulting flood risk was not well understood. FCC has therefore embarked upon a program of undertaking overland flood studies in order to identify these major overland flow paths and to address the requirements of the FDM.

Identifying properties at risk of overland flooding within the entire LGA is a major task. Instead of carrying out detailed assessment for the entire LGA in one step, FCC decided to undertake overland flood studies in a number of stages. In 2003-2004, Sinclair Knight Merz (SKM) was engaged by FCC to undertake the *Fairfield City Overland Flood Study* (SKM, 2004). This was a preliminary assessment of the flood risk from overland flows within the urban areas of the Fairfield LGA. The study divided the LGA into 18 sub-catchments and ranked each sub-catchment in terms of the potential severity of overland flooding.

The *Canley Corridor Overland Flood Study* (SKM, 2009), which primarily covered the Canley Heights sub-catchment, was undertaken as the first of a series of detailed overland flood studies by FCC, as there was a large amount of asset data available, and because there was a significant amount of urban renewal occurring in the study area. The Canley Corridor study served as a pilot study to evaluate a number of alternative flood modelling and mapping methodologies, based on different assumptions made about the capacity of the stormwater drainage system. The Canley Corridor overland flood study defined the flood behaviour and identified the major overland flow paths within the Canley Corridor catchment, identified properties at risk of overland flooding for the preparation of flood risk precinct maps.

It was concluded from the Canley Corridor overland flood study that the remaining overland flood studies should apply a similar methodology that was developed and selected as the preferred approach in the Canley Corridor study. FCC subsequently commissioned SKM to undertake overland flood studies for the Fairfield, Old Guildford and Smithfield sub-catchment. These studies were carried out in association with Fairfield Consulting Services (FCS), a business unit division of FCC.

The Wetherill Park catchment was chosen to be the next catchment for analysis. This flood study was undertaken as an in-house project by Council.

1.2. Study Area

1.2.1. Description

The Wetherill Park overland flow catchment is located in the north-western portion of Fairfield LGA, immediately south of the Prospect Reservoir and Prospect Creek and east of the Sydney Water Supply Channel. The study area locality is shown on **Figure 1-1**.

Figure 1-2 shows the study area in detail. A total area of 1911.7 hectares was included in the model, which covered the suburb of Wetherill Park and parts of Smithfield and Bossley Park. Since it was intended to use the Direct Rainfall Method, the modelled area was extended to include an area of 428.5 hectares within Holroyd City Council, to fully encompass the catchment area of Prospect Creek to the model outlet.

The study area contains a diverse range of land uses. The largest component is the Wetherill Park Industrial Estate, which covers an area of 644.3 hectares, which is 43.4% of the study area with the Fairfield LGA. It is the largest industrial estate in the southern hemisphere and the hub of manufacturing and distribution in Greater Western Sydney, with in excess of 1,000 manufacturing, wholesale, transport and service firms employing over 20,000 people.

The next largest component is rural/open space, which covers an area of 509.3 hectares, which is 34.3% of the study area within the Fairfield LGA. The area covers parts of the suburb of Horsley Park and the Western Sydney Regional Parklands, a large conservation and recreation reserve.

The smallest component is residential, which covers an area of 329.6 hectares, which is 22.2% of the study area within the Fairfield LGA. The development in this area is mostly single dwelling houses.

The topography of the study area is shown in **Figure 1-3**. The highest point is at approximately 90m AHD, located in the far south-western extremity of the catchment near the intersection of Cowpasture Road and Restwell Road.

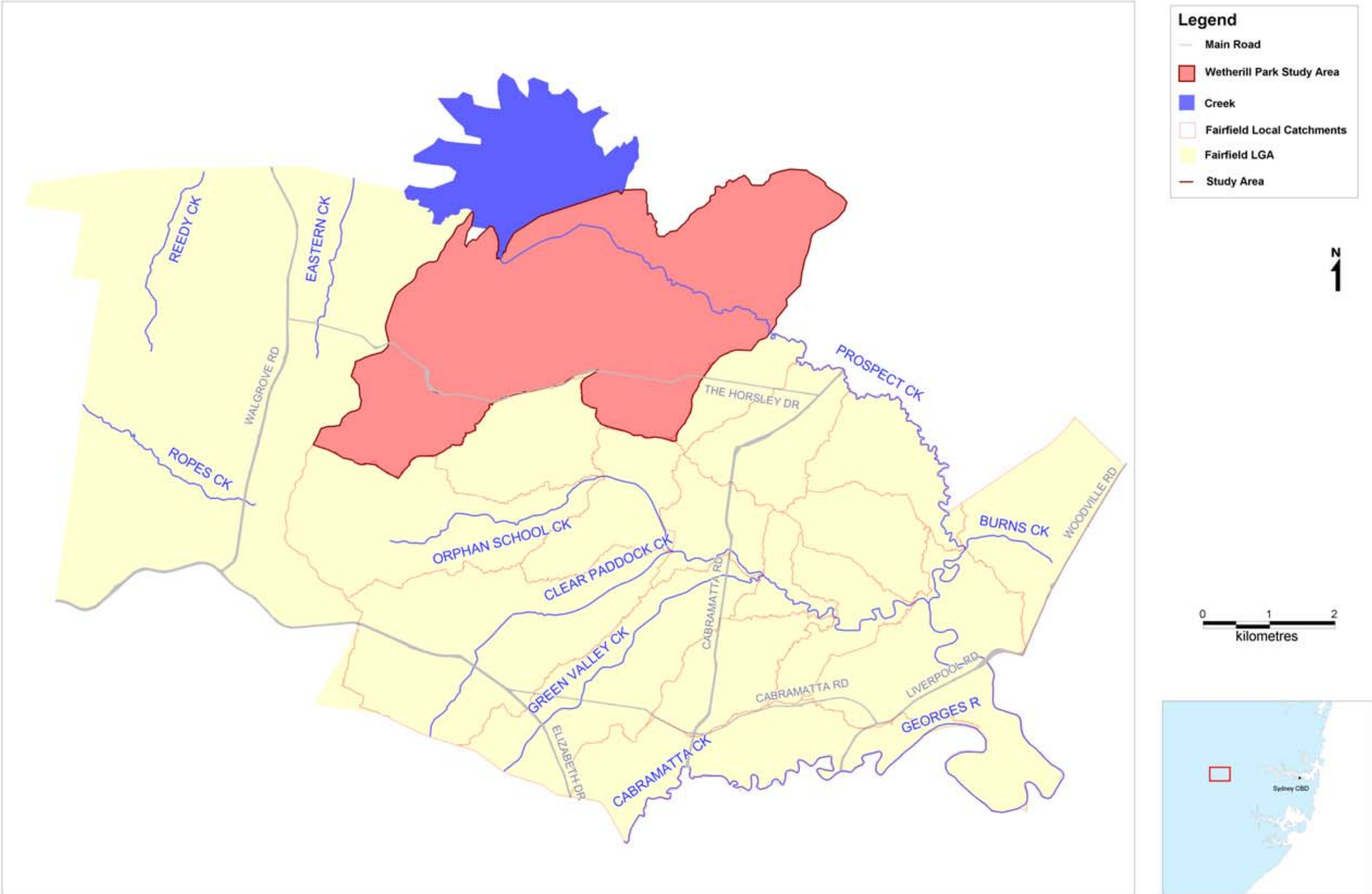


Figure 1-1 Locality

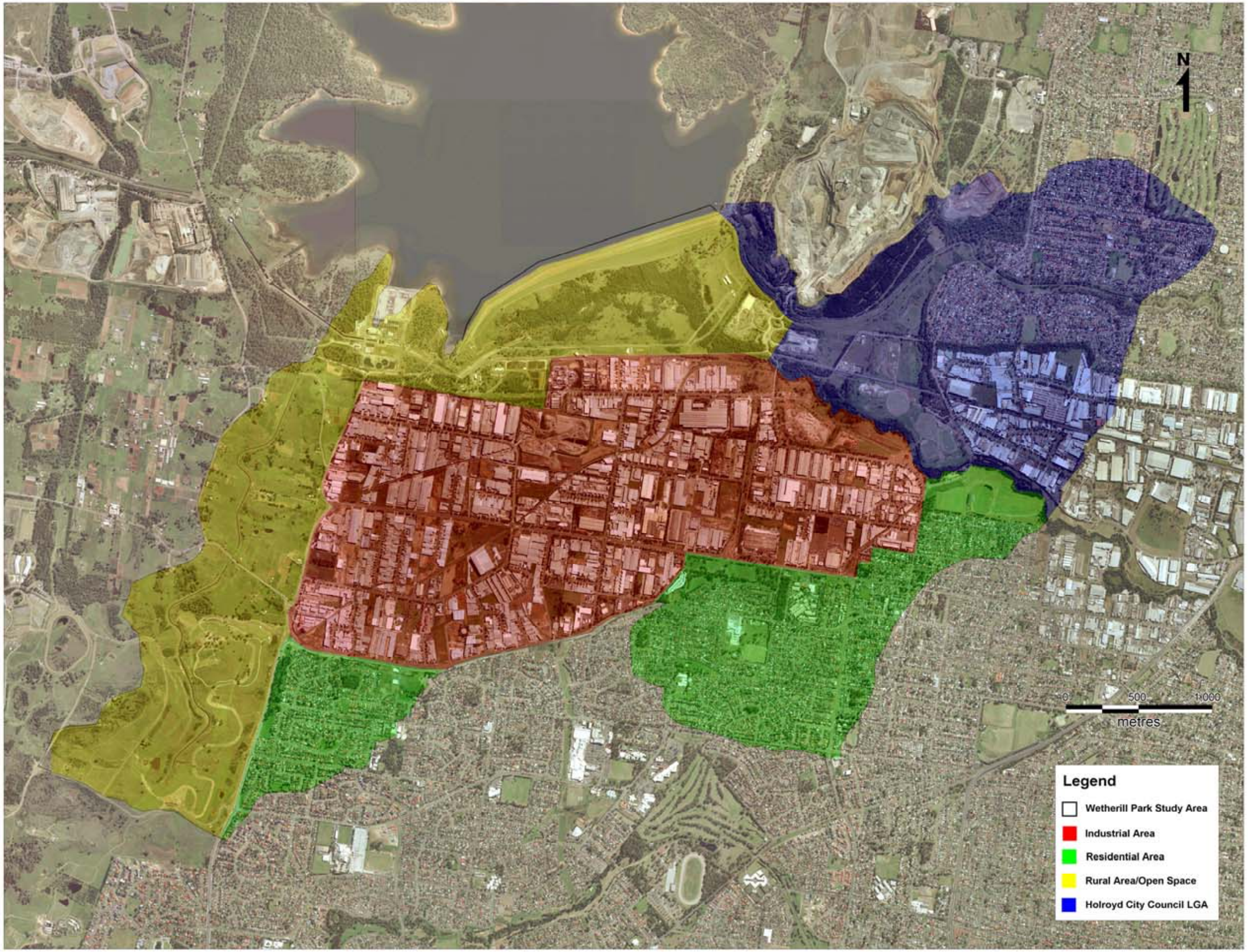


Figure 1-2 Study Area

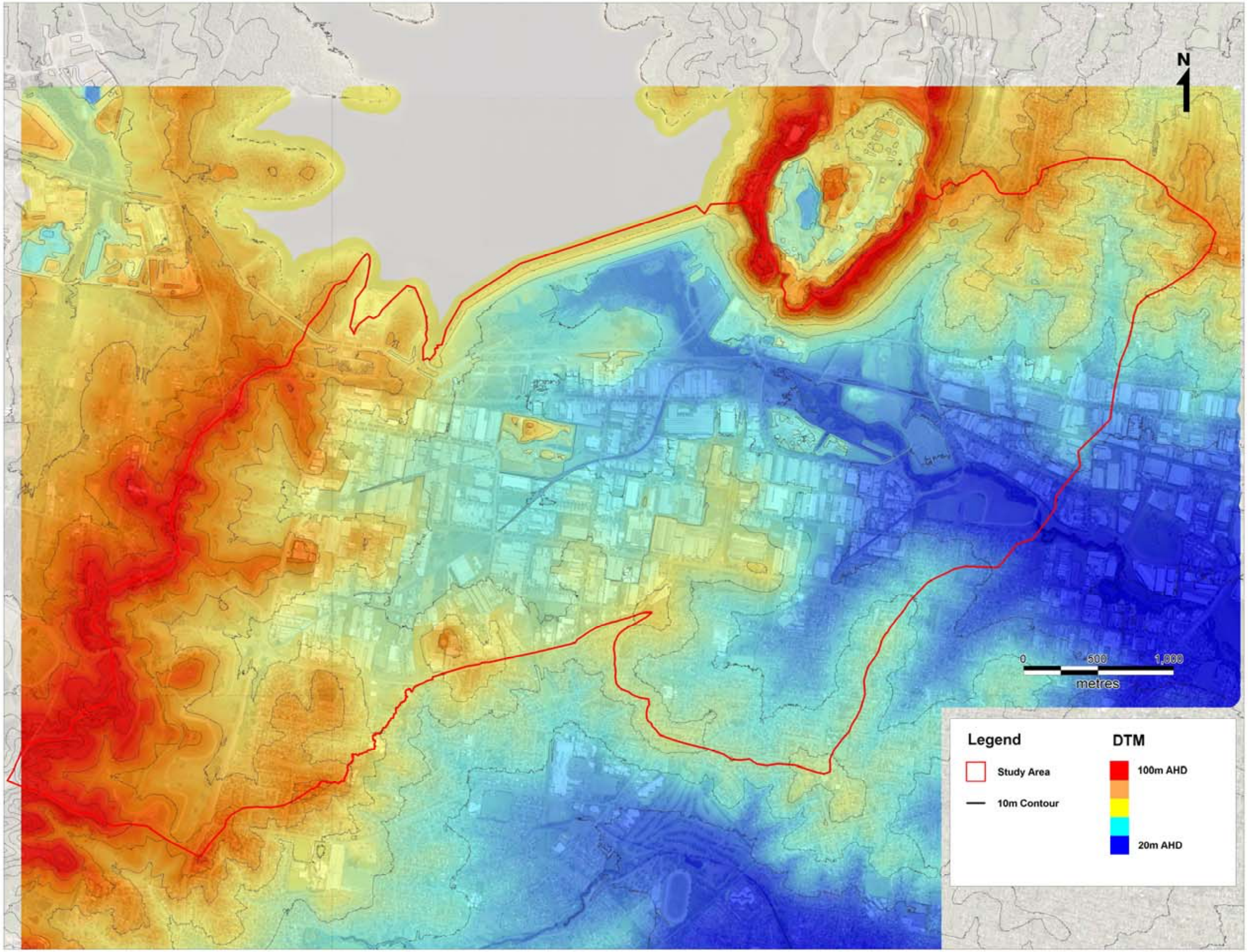


Figure 1-3 Study Area Topography

1.2.2. Drainage Conditions

The catchment drains in a generally north-easterly direction into Prospect Creek. The study area is traversed by three concrete-lined trapezoidal open channels, totalling over 3.8km in length.

The longest of these, known in this study as the "Main Channel" originates at the outlet of the box culverts underneath the roundabout at the intersection of Cowpasture Road and The Horsley Drive. The channel traverses in a north-easterly direction to Toohey Road, where it is conveyed in a rectangular culvert for just over 200 metres to its outlet at Bentley Street. The channel then continues in a roughly easterly direction with bridge crossings under Hallstrom Place and Durian Place, before continuing north-east to the Victoria Street/Newton Road intersection, where flow is conveyed in twin rectangular box culverts under the roundabout. The channel continues further north-east under two small private bridges, and then under additional bridge crossings at Elizabeth Street, Davis Road, the Liverpool-Parramatta Transitway (LPT), and Widemere Road, before discharging into Prospect Creek.

The total length of the Main Channel is 1655 metres. Its gradient is approximately 0.8% upstream of the Victoria Street/Newton Road intersection, before flattening to approximately 0.5% downstream of the intersection.

The next longest channel, known in this study as "Tributary 1" originates at the outlet of the box culvert under Cowpasture Road and Potter Close. The channel traverses in a roughly easterly direction, flowing through twin rectangular box culverts under Newton Road before joining into the Main Channel approximately 320 metres upstream of its crossing with Elizabeth Street.

The total length of Tributary 1 between Potter Close and its confluence with the Main Channel is 1380 metres and it has a reasonably constant gradient of approximately 1.0%.

The shortest and most easterly channel is known in this study as the "Rosford Channel" and originates just west of the Victoria Street/Wetherill Street roundabout. The channel traverses in a roughly north-easterly direction to Redfern Street, where flow is conveyed under the road in a 3 cell circular box culvert. Downstream of Redfern Street, the channel continues to Hassall Street, where flow is conveyed under the road in another 3 cell circular box culvert and into the Hassall Street detention basin.

The total length of the Rosford Channel between Victoria Street and Hassall Street is 775 metres and it has a reasonably constant gradient of approximately 0.5%.

All three channels are trapezoidal in profile with approximately 2 in 3 side slopes. The upper reaches of the Main Channel (upstream of its confluence with Tributary 1) has a base width of between approximately 1.5 metres to 1.75 metres and a height ranging from about 2.0 metres to 3.0 metres. Downstream of its confluence with Tributary 1, the base width increases to approximately 4.0 metres to 4.25 metres and its height ranges between about 2.75 metres and 3.25 metres.

The profile of Tributary 1 is considerably smaller, with a base width ranging from approximately 0.80 metres to 1.10 metres and a height of approximately 2.0 metres.

The profile of the Rosford Channel is more similar to the lower reaches of the Main Channel, with a typical base width of 3.0 metres and an average height ranging from 1.5 metres to 2.0 metres.

Other drainage infrastructure in the study area includes a small detention basin located in the Emerson Street Reserve, which is bounded by The Horsley Drive to the north and by Emerson Street to the east. The embankment is just over 2m in height and the volume is approximately 18,000 m^3. The basin outlet is a 900mm diameter pipe which continues east and then north and ultimately drains into the Rosford Channel.

The study area contains an extensive pipe drainage network. The network generally follows the overland flow paths in the study area, usually draining into either one of the concrete-lined open channels or directly into Prospect Creek.

The reach of Prospect Creek within the study area includes it's outlet from Prospect Reservoir, the bridge crossing at Widemere Road, the Hassall Street detention basin, the bridge crossing at Gipps Road and the Rosford Street detention basin.

Construction of both basins was completed in the mid 1980's. The Hassall Street basin is the more upstream of the two, originally constructed with an embankment level of approximately 30.1m AHD and an outlet structure made up of a 4-cell box culvert, with an average width of 2.4m and average height of 1.8m. Improvement works carried out in the mid-1990's saw two of the four cells on the outlet structure closed and the embankment raised to approximately 31.1m. This resulted in a maximum embankment height of about 5.25m and increased the total basin storage area from 350,000 m^3 to 600,000 m^3.

The Rosford Street Basin is further downstream on Prospect Creek, constructed with an embankment level of 24.35m AHD and an outlet structure made up of a 5-cell box culvert. Improvement works in the mid-1990's saw one of the five cells on the outlet structure closed. The four open cells vary between 2.40m to 2.44m in width and 2.08m to 2.24m in height.

1.3. Study Objectives

Key objectives of this study are to:

- Identify the major overland flow paths within the Wetherill Park catchment study area.
- Determine the nature and extent of overland flooding and flood risk in the study area.
- Identify properties at risk of local overland flooding and quantify the risk of flooding to these properties.
- Produce flood model results (flood level, velocity and flow) for the 20, 100, 500 and 10,000 year ARI and PMF storm events
- Prepare flood extent (depth and velocity) maps and flood risk precinct maps for the study area for the 100 year ARI and PMF events.
- Assess the sensitivity of flood behaviour to changes in hydrologic and hydraulic characteristics in the catchment.

Originally, it was intended to also identify "Zones of Significant Flow" to determine those sections of overland flow paths through properties which would need to be kept clear in order to reduce flood risk. Due to time constraints, these zones will be identified in the floodplain risk management study and plan.

2. Review of Available Data

2.1. Previous Studies

The only previous study of direct relevance to the study area was the *Flood Risk Assessment for 449 Victoria Street, Wetherill Park (Cardno 2011).* This involved the construction of a TUFLOW model of the reach of the Main Channel from the bridge at Durian Place to a short distance downstream of the culvert crossing under the intersection of Victoria Street and Newton Road. The proposed development site was on a pair of triangular shaped blocks of land located on opposite banks of the concrete-lined open channel immediately upstream of the Victoria Street/Newton Road intersection. A number of development options were considered including vehicular and pedestrian crossings of the open channel as well as various degrees of land filling. Model results were presented for existing and future conditions for 20 year and 100 year flooding assuming different degrees of culvert blockage.

2.2. Topographic Survey

2.2.1. Airborne Laser Survey

Airborne Laser Survey (ALS), conducted in January 2003, was used to generate a Digital Terrain Model (DTM) for the entire Fairfield LGA. The DTM has subsequently been used in a number of projects undertaken for FCC, including this current study. The ALS data used had been filtered to reduce the density of points and to remove non-ground points such as buildings, bridges and over/underpasses.

2.2.2. Ground Survey

Ground survey was obtained by FCC staff in order to provide more accurate information than ALS in certain areas.

Cross section profiles were obtained at specified intervals for all three concrete-lined open channels. Survey of all culvert and bridge crossings was also undertaken to obtain the necessary geometric details required for model input. Measured details included:

- upstream and downstream invert, obvert and soffit levels;
- dimensions of the waterway opening;
- structure length in the direction of flow;
- details of any railing or fencing.

2.2.3. Design and Works as Executed Drawings

Design plans and works as executed (WAE) drawings were obtained for the following features for use in this study:

- Design drawings W5064-4 ("Hassall Street Basin, 4 Cell Outlet Culvert, General Arrangement", FCC, January 1984) and D/R2/12 ("Hassall Street Basin Improvement Work",FCC, June 1995) for geometric details of the outlet of the Hassall Street Basin
- Design drawings W5064-9 ("Rosford Street Basin, 5 Cell Outlet Culvert, General Arrangement", FCC, February 1984) and D/R2/13 ("Rosford Street Basin Improvement Work",FCC, June 1995) for geometric details of the outlet of the Rosford Street Basin

- Design drawings 392274/C/502 (“Bridge No.2 Infrastructure Works, General Arrangement Plan”) and 392274/C/313 (“Longitudinal Section, Widemere/Reconciliation Road”) by ACOR Appleyard Consultants Pty Ltd, March 2010 for geometric details of the bridge crossing of Prospect Creek at Widemere Road

2.2.4. Pit and Pipe Survey

The levels and dimensions of key pits and pipes were surveyed by Bankstown City Council surveyors between February and May 2010. Typical details surveyed include:

- Pit name/asset number
- Pit coordinates (Easting, Northing)
- Pit surface level (m AHD)
- Pit invert level (m AHD)
- Structural type (eg kerb inlet, grated surface inlet, letterbox inlet, junction pit, etc)
- Inlet type (sag or on grade)
- Pit entry dimensions – lintel length and/or inlet grate dimensions.
- Downstream pipe length
- Downstream pipe dimensions – diameter or width/height

Data on the pits and pipes is contained in **Appendix A**.

Not all pits and pipes in the stormwater network were surveyed. Only significant infrastructure, defined as pipes having a diameter of 900mm or greater, or infrastructure located in known flooding “hot-spots” were surveyed. This discussed further in **Section 3.3.2.**

2.3. AUSIMAGE™ Aerial Photography

AUSIMAGE™ aerial photography was used extensively in this study, mainly for data validation and presentation of results in the preparation of flood extent and risk maps. The aerial photography that was used was flown in January and February 2011. This photography is at a resolution of 0.10m.

2.4. Spatial Data

Data from a Surface Impervious Area (SIA) study undertaken for FCC by Lagen Spatial Pty Ltd became available in 2009. The intention of the SIA study was to accurately identify all impervious areas across the LGA. However inspection of the received data revealed numerous errors in land classifications and the data was subsequently discarded.

2.5. Rainfall Intensity-Frequency-Duration Data

An Intensity-Frequency-Duration (IFD) curve was derived for the centroid of the Wetherill Park catchment (33.850° S, 150.900° E) using the IFD Program provided on the Bureau of Meteorology website (www.bom.gov.au/hydro/has/cdirswebx/cdirswebx.shtml). The derived curve was used to provide design rainfall intensities for events up to and including the 100 year ARI event and the data was extrapolated to derive average rainfall intensities for the 500 year ARI event.

Further detail on rainfall data is provided in Section 3.4.1. The IFD data is provided in Appendix **B**.

2.6. Surveyed Peak Flood Levels For 18th April 2012 Flood Event

Following the flood event of 18th April 2012, field inspections were carried out to document evidence of the peak flood level for use in model calibration. Locations adjacent to the open channels, particularly at and around bridge and culvert crossings were inspected and indications of the peak flood level, such as debris marks, trash lines and flattened vegetation were documented for surveyors to return at a later date and measure the peak levels.

Further information on the measured peak flood levels is provided in **Section 0**.

2.7. Gauged Rainfall Data 18th April 2012 Flood Event

For the purposes of model calibration, rainfall data was obtained for the below gauges for the flood event of 18th April 2012. Rainfall data was obtained for all gauges in one minute increments.

Table 2-1 Gauged Rainfall Data for 18th April 2012 Flood Event

Authority	Station number	Station Name	Latitude	Longitude
Bureau of Meteorology	067119	Horsley Park Equestrian Centre	33°51'04''	150°51'24''
Sydney Water	567169	Abbotsbury	33°52'08''	150°51'33''
Sydney Water	567083	Prospect Reservoir	33°49'09''	150°54'45''

3. Model Development

3.1. Background

In preparation for this study, a review titled "Wetherill Park Overland Flood Study, Choice of Flood Model and Modelling Methodology" was undertaken. In this review the available modelling packages and methodologies were considered and the merits of each assessed. It was intended that this would establish the methodology not only for this study, but for subsequent investigations for the remaining overland flow catchments in the LGA.

The findings of this review are summarised below. Full details of this Memo are provided in **Appendix H**.

3.1.1. Choice of Hydraulic Model

The TUFLOW and XP-STORM models were compared to determine which would be more suitable for use for future overland flood studies in the Fairfield City Council LGA.

The use of XP-STORM in the Old Guildford Overland Flood Study encountered a number of issues, particularly regarding the model's capability of reading certain input data, as well as model stability, memory requirements (thereby restricting the minimum grid size) and the capability of performing batch runs or multiple runs concurrently. By contrast, the TUFLOW model was applied to the comparable Fairfield CBD overland catchment and none of these issues applied.

It was therefore decided that TUFLOW was the preferred model for use in this and future overland flood studies.

3.1.2. The Direct Rain Method

The Direct Rain Method was investigated in detail to determine whether it could be applied in the flood modelling of the Wetherill Park catchment and for subsequent catchments in the Fairfield City Council LGA.

In the Direct Rain Method, the model effectively considers each cell of the 2D grid as a subcatchment and calculates runoff based on user-specified rainfall inputs (e.g. a rainfall hyetograph, Initial Loss (IL), Continuing Loss (CL) etc.) as per a conventional hydrologic model. This approach eliminates the need to set up an independent hydrologic model of the catchment, and allows the catchment hydrology, underground pipes and overland flow to be represented in a single model, rather than using separate models for the hydrology and the hydraulics.

The method also offers notable technical advantages, such as the allocation of subcatchment runoff to certain pits or open channels based on hydraulic principles, rather than judgement of subcatchment delineation. The Direct Rain Method also allows for proper facilitation of cross catchment flows which may occur in larger events.

The Direct Rain Method must to be properly verified, via the modelling of a few selected subcatchments using an independent hydrologic model. The flows generated by the independent hydrologic model should be compared with those of the hydraulic model and where there are differences, some interpretation of the results can be made and, if deemed necessary, adjustments made to the model input parameters.

TUFLOW's Direct Rain Method has been applied to a number of urbanised catchments in the Bankstown City Council LGA and the feedback received was that the model performed well and provided satisfactory results.

It was therefore decided to use the Direct Rain Method in the analysis of this catchment and for future overland flood studies.

3.1.3. Modelling Approach

The modelling approach adopted in the Wetherill Park Overland Flood Study therefore consisted of the following:

- Development of a TUFLOW model to represent catchment hydrology via Direct Rain and the 2D floodplain including topography, surface roughness and boundary conditions;
- Further development of the TUFLOW model to represent the open channels and selected key/critical pits and pipes of the drainage network;
- Rudimentary model calibration to the high water marks observed for the 18th April 2012 flood event, and comparison with the results of an independent hydrologic model with adjustment of model parameters where necessary;
- Execution of the TUFLOW model for the various design flood events. Maximum flood levels, depths, velocities, flow rates and flooding extents are output in results files.

The adopted modelling approach in TUFLOW allows representation of the open channels and the stormwater drainage system in 1D, in addition to the overland flow floodplain in 2D, with dynamic linking between the 1D and 2D domains in TUFLOW. This means that water is able to flow between the open channels and the 2D floodplain, and between the drainage system and the 2D floodplain, depending on the hydraulic conditions.

3.2. 2D Domain Setup

3.2.1. Topography

The topography of the catchment is represented in the model using a 5m grid, which is larger than that used in previous overland flood studies. However the catchment of the study area is also considerably larger than previous overland flood studies, as summarised in Table 3-1 below.

Table 3-1 Comparison of 2D Cell Size and Catchment Area for Previous Overland Flood Studies

Overland Flood Study	2D Cell Size (m)	Catchment Area (ha)
Canley Corridor	2.0	258
Fairfield CBD	2.0	232
Old Guildford	2.5	385
Smithfield	2.0	292
Wetherill Park	5.0	1911

However, as indicated earlier the Wetherill Park catchment is dominated by its industrial component which has considerably larger individual buildings and larger inter-building spacing than those of previous overland flood studies, which cover largely residential areas. The Wetherill Park catchment also has a significant rural/open space component. Therefore, it is not considered necessary to use a 2.0m cell size, and it is expected that flood behaviour in the study area will be adequately represented by the use of a 5.0m cell size.

Furthermore, the use of a 2.0m cell size for a 1911 hectare catchment is expected to result in impractical model run times of at least several days or even a full week. The use of a 5.0m cell size would reduce most run times to approximately 10-12 hours, thus allowing model simulations to be performed overnight.

The basis of the topographic grid used in the TUFLOW model is the ALS survey. **Figure 1-3** shows ground elevations within the Wetherill Park catchment based on this data.

3.2.2. Building Polygons

Previous overland flood studies undertaken within the Fairfield LGA considered buildings as solid objects in the floodplain. This means that buildings form impermeable boundaries within the model, and that while water can flow around buildings, it cannot flow across their footprint.

However since this study uses the Direct Rainfall approach, this approach could not be applied because it would exclude all the rainfall volumes which landed within the building footprints. Hence, an alternate approach was applied, by varying Manning's n values with water depth.

Across building footprints, a low value of Manning's n was applied for shallow depths to decrease the response time of the catchment. This accounted for roofs and hardstand surfaces that are directly connected to surface and sub-surface drainage paths. Once depths increase, the effects of obstructions due to buildings are accounted for with much higher values of hydraulic roughness.

It should be noted that given the number of buildings within the floodplain, it was not considered practical to verify whether each building was slab on ground or raised with a clear understorey area. Furthermore, the rate and degree to which floodwaters enter a building will vary depending on factors such as whether doors or windows are open, and whether these openings are exposed to the flows. Hence, an identical approach was taken to the representation of buildings in the floodplain, where a low value of hydraulic roughness was assumed for a nominal flow depth, with a much higher value of hydraulic roughness assumed for greater depths.

A full summary of the values of Manning's n used in the model is provided in **Section 3.2.5**.

3.2.3. Detention Basins

Detention basins in the TUFLOW model were represented as 2D objects in the floodplain. Basin topography was typically derived from the DTM, with the crest of the basin walls being more accurately defined using surveyed break lines. Outlet structures were represented using 1D culvert or pipe elements with their levels and dimensions derived from survey data.

3.2.4. Property Fencelines

Fencelines have not been explicitly represented in the model and floodwaters can flow across them freely. Although fences may obstruct overland flood flows in some parts of the catchment, experience indicates that representing fences in the hydraulic model requires making unvalidated assumptions about depths at which fences overflow or fail. Also, including fence lines would have required on-site identification of fence type, blockage and structural strength for individual properties. This was beyond the scope of this study.

The potential obstruction to flow caused by fences has generally been represented by increasing the cell roughness (Manning's n values) for certain land uses, as described in **Section 3.2.5**. The limitation of this approach is that the flood levels may be slightly overestimated and flow velocities slightly underestimated for flooding within properties depending on the actual locations of obstructions and the interaction of flood flows with these obstructions. However, this approach does preserve the likely typical flooding behaviour, in which floodwaters use the road corridor as the preferential flow path.

3.2.5. Surface Roughness

All parts of the study area within the TUFLOW model were assigned hydraulic roughness values according to land use type and ground cover. These are based on standard reference values for Manning's n in *Open Channel Hydraulics* (Chow, 1959) and typical values used in previous FCC flood studies.

Note that a depth-varying value of Manning's n roughness was applied for Buildings, with a value of 0.03 used for flow depths below 0.03m, a value of 1.00 used for flow depths above 0.10m, and linear interpolation used for depths in between.

Table 3-2 TUFLOW Model Grid Hydraulic Roughness Values

Land Use Type	Assumed Manning's n Roughness
Soil/Short Grass	0.03
Buildings	1.00
Open Concrete (eg driveways, carparks)	0.02
Roads	0.02
Dense Vegetation	0.08
Swimming Pools	0.013

3.3. 1D Domain Setup

3.3.1. Open Channels and Crossings

The three concrete-lined trapezoidal open channels described in Section 1.2.2, along with their associated bridge and culvert crossings were incorporated into the 1D network of the model using the ground survey obtained by FCC staff.

It should be noted that the main channel of Prospect Creek was represented in the 2D domain and was not explicitly modelled as a 1D reach like the three concrete-lined trapezoidal channels. This form of schematisation was chosen since this reach of Prospect Creek has a generally small and poorly defined channel and its flow behaviour along the reach within the study area is dominated by the two detention basins. Since this study aims to simulate flood behaviour in the Wetherill Park overland catchment, and does not attempt to accurately reproduce flood levels in Prospect Creek, this approach is considered appropriate.

Channel crossings were represented as either 1D culvert or 1D bridge elements, with the associate flow over the crossing typically modelled as returning to the 2D domain, however the use of a 1D weir element at a few locations (Hallstrom Place, Durian Place and Private Bridge #2) was preferred since it resulted in improved model stability.

The various railings on culvert and bridge crossings (either hand rails or safety/guard rails) were assumed to be fully blocked by debris if the spacing between the bars was less than 150mm, and a break-line was used in the model to increase the level of the crossing to the top of the rail.

Railings with greater than 150mm bar spacing were assumed unblocked. Refer to Figure 3-1and Figure 3-2 for examples of crossings with unblocked and blocked railings respectively.

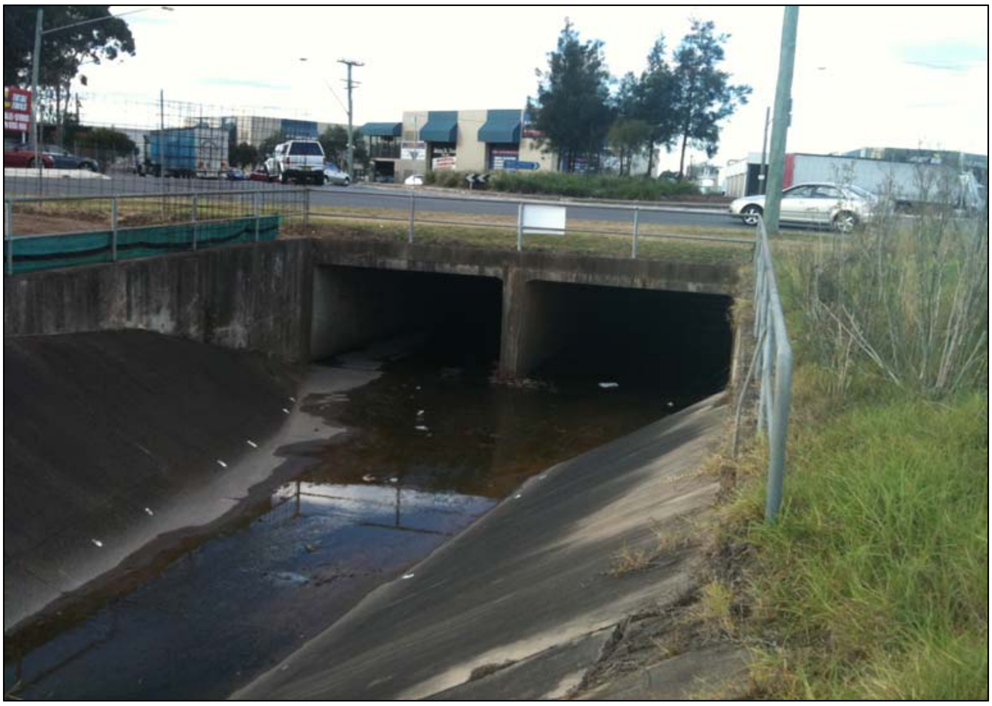


Figure 3-1 View of Railings at Victoria Street/Newton Road Culvert Inlet – Assumed Unblocked

Figure 3-2 View of Railings at Davis Road Bridge – Assumed Blocked

Rails were typically 0.5m to 1.0m high, with two notable exceptions:

- A 2.2 metre high iron fence on the upstream and downstream face of the first private bridge crossing on the Main Channel;
- A 2.0 metre high chain wire fence on the downstream face of Newton Road crossing on Tributary 1.

At both of the above locations, the 1:100 year ARI design flood was well below the deck level and the high fences had therefore no impact on model results.

The Widemere Road crossing of Prospect Creek was modelled as a 2D flow constriction to allow for energy losses at the bridge piers.

3.3.2. Stormwater Pits and Pipes

Stormwater pit and pipe attributes were based on the survey undertaken by Bankstown City Council. The modelled network was typically comprised of pipes with a diameter of 900mm and greater and their associated pits, with smaller pipes included as necessary at the known trouble spots to represent these locations in more detail. This is in keeping with previous overland flood studies undertaken within the Fairfield LGA.

No stormwater pits and pipes within the Holroyd LGA were included in the model since details of the underground stormwater drainage in this area were not available and the accurate representation of flooding in this area was not the purpose of this study in any case.

The surface inflows were separately defined for different pits in the model. Pit inflow relationships were defined in terms of flow depths versus pit inflow.

The kerb inlet pits in the study area are typically Hornsby Council inlet pits (lintel and grate) of inlet lengths of 0.9, 1.2, 1.8, 2.4, 3.0, 3.6 and 4.2m (internal dimensions). For these pit types, the inflow relationships adopted in TUFLOW were extracted from the DRAINS model default database.

For large specialised pit inlets, such as grated surface or letterbox inlets, the pit inflow relationships were derived as a depth versus inflow relationship assuming weir or orifice flow (with the conservative case governing) along the pit grate's perimeter.

Blocking factors adopted in the model were 30% in the calibration event and the 20 year ARI and 50% for the 100 year ARI and larger events.

The model contains a total of 632 lengths of stormwater pipe and a total of 388 inlet pits, comprising 285 kerb inlet pits and 103 grated surface or letterbox inlets. A further 242 junction pits were included in the model.

The modelled 1D drainage network, including concrete-lined, trapezoidal channels and the underground stormwater drainage network is shown in Figure 3-3.

Figure 3-3 Modelled 1D Drainage Network

3.4. Boundary Conditions

3.4.1. Rainfall

As stated earlier, the Direct Rainfall method was used in this study. Rainfall depths were input to the model in 5 minute increments, with a uniform spatial distribution assumed for both the calibration and design events.

The loss rates adopted are provided in Table 3-3 below.

Table 3-3 Adopted Rainfall Losses

	Initial Loss (mm)	**Continuing Loss (mm/h)**
Calibration Event		
Pervious Areas	0	2.5
Impervious Areas	0	0
Design Events		
Pervious Areas	10	2.5
Impervious Areas	0	0

Note that a zero Initial Loss was applied for the calibration model, given the considerable quantity of lead-up rain that fell prior to the start of the event.

For the calibration event, measured rainfall from selected nearby gauging stations were used as input to the model. This process is outlined in Section 0.

For the design events, the 20, 100 and 500 year ARI events were modelled as Australian Rainfall and Runoff 1987 (ARR87) storms. Average rainfall intensities for each design event were obtained from an IFD curve specially derived for the centroid of the Wetherill Park catchment (33.85S,150.90E). Temporal patterns from *Australian Rainfall and Runoff Volume 2* (Institution of Engineers, 1987) were then used to derive design storm series.

The average rainfall intensity for storm events up to and including the 500 year ARI event are presented in **Appendix B.**

Design rainfall time series for the Probable Maximum Precipitation (PMP) events were based on the Generalised Short Duration Method (GSDM) in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method* (BOM, 2003).

The design rainfall time series for the 10,000 year ARI events were derived using the method for determining rainfall from extreme storm events (between 1:500 year ARI and the PMP) in *Australian Rainfall and Runoff – Volume 1 Book 6* (Institution of Engineers, 1998). A notional AEP of 10^{-7} was assumed for the PMP event given the catchment size and based on guidance in the method. A GSDM temporal pattern was adopted for all modelled extreme rainfall events, that is, the extreme rainfall storm events were assumed to have the same temporal pattern as the PMP event.

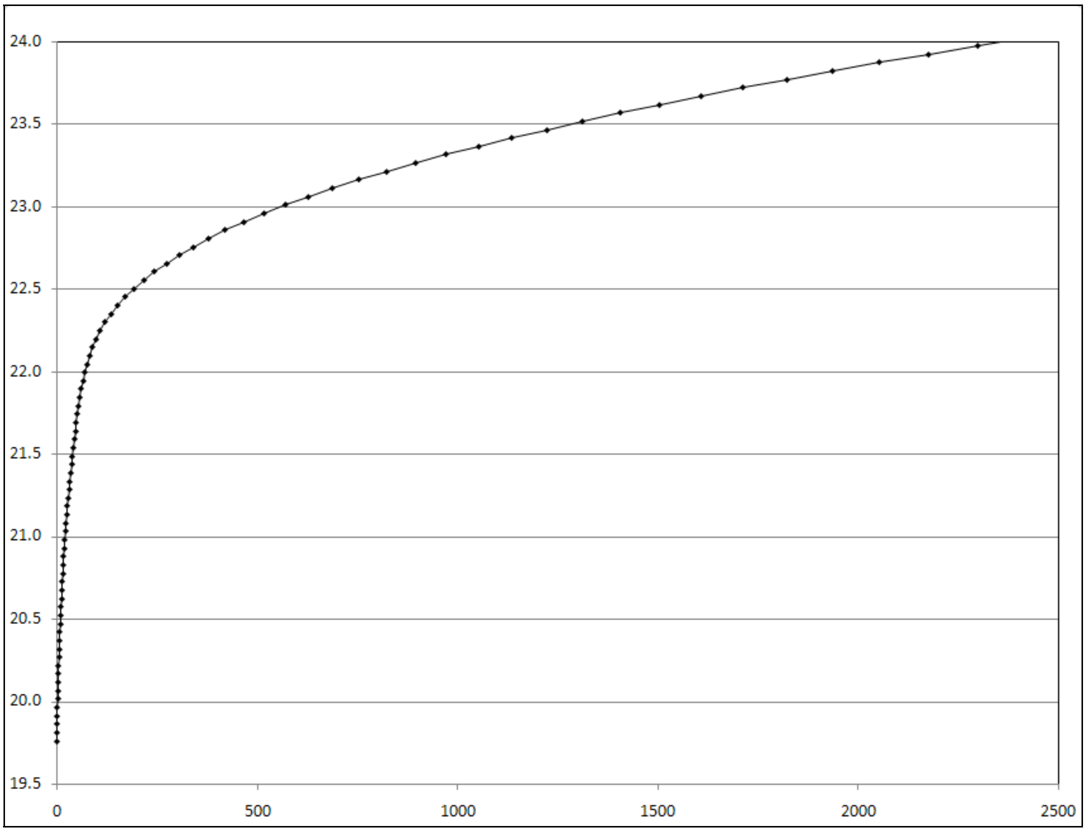
The average rainfall intensity for the extreme storm events is presented in **Appendix B.**

3.4.2. Downstream Boundary

A Stage-Discharge (HQ) boundary condition was adopted at the downstream boundary of the TUFLOW model, at a location on Prospect Creek approximately 70 metres downstream of the outlet of the Rosford Street detention basin. The HQ relation was automatically generated by the model, with an assumed water surface slope of 0.5%, based on the average bed slope of that reach of the creek.

The Stage-Discharge relationship used as the model downstream boundary condition is provided in Figure 3-4.

Figure 3-4 Stage Discharge Relationship used as Model Downstream Boundary Condition



3.4.3. Prospect Reservoir Outflows

The “Review of Prospect Creek Flood Levels” (Cardno & Willing, June 2004) summarised estimates of peak outflows provided by the Sydney Catchment Authority (SCA) from the Prospect Reservoir into Prospect Creek.

The SCA advised that the operating protocol for the reservoir is to maintain the storage drawn down between 0.43m and 0.88m below Full Supply Level (FSL). These peak outflows are presented in the Table 3-4 below:

Table 3-4 Peak Outflow from Prospect Reservoir (m^3/s)

ARI (years)	Peak Outflow From Prospect Reservoir (m^3/s)		
	FSL	FSL – 0.43m	FSL – 0.88m
2	4.2	0	0
5	5.9	0	0
10	6.9	0	0
20	9.6	0	0
50	12.6	2.0	0
100	15.2	4.1	0
2,000	33.0	20.0	2.6
50,000	68.0	48.0	17.0
1,000,000	100.0	69.4	38.5

Hence for the calibration event and for all design events with the exception of the PMF, the Prospect Reservoir outflow was conservatively taken as the higher discharge, i.e. for 0.43m below FSL. For the PMF event, the outflow was taken assuming the reservoir was at its FSL. These discharges, assumed constant in the modelling, are summarised in Table 3-5 below. Note that for the 1:500 year and 1:10,000 year ARI events, the discharge in Prospect Creek was estimated by linear interpolation of a log-normal plot of the Prospect Reservoir Peak Outflows for each storm ARI.

Table 3-5 Adopted Prospect Creek Discharges

Storm Event in Wetherill Park Overland Flow Catchment	Prospect Creek Discharge (m^3/s)
Calibration Event (18th April 2012)	2.0
20 year ARI	0.0
100 year ARI	4.1
500 year ARI	12.6
10,000 year ARI	34.0
PMF	100.0

3.5. Initial Model Runs

3.5.1. Model Stability

TUFLOW models, if configured appropriately, are typically numerically stable. However, models often require "debugging" during their initial development in order to rectify issues in the model which cause model instability and inaccuracy. Several such issues were encountered in the Wetherill Park model. These issues are described below:

- A sharp negative gradient (a rise of approximately 1.0m over a 40m length) at the Main Channel outlet was causing instabilities and poor model convergence. This was addressed by converting the second last channel reach to a weir section, which enhanced model stability while providing a realistic representation of the channel bathymetry.

- Small volumes of additional storage were added to 1d channel nodes, at three (3) locations on the Main Channel, one (1) location on Tributary 1 and eight (8) locations on the Rosford Channel. This additional storage resulted in improved model convergence and was found to have a negligible effect on model results.
- Certain long 1d network reaches were broken up, with the model automatically interpolating a cross section in between. This included four reaches on the Main Channel (from Bentley Street to downstream of Durian Place) and one reach on Tributary 1 (just upstream of the Newton Road crossing). This was found to considerably improve model stability, particularly for extreme events (1:500 year ARI and greater).

The mass balance error in all design model runs were well under +/-1% and hence were considered acceptable.

3.5.2. Quality Assurance

An external technical review of the Wetherill Park Overland Flood Study TUFLOW model was undertaken by BMT-WBM to ensure that the model was configured appropriately. The review considered the 18th April 2012 calibration event and the 1:100 year ARI (2 hour) design event.

The review recommended a number of model changes (Memo "L.19202.002.01" dated 13th December 2012), all of which were adopted. The recommended changes included a number of refinements to improve model stability such as minor adjustments 1D structures and to 1D/2D boundaries, the elimination of several short sections of underground drainage pipes and also change in 1D timestep from 0.15 seconds to 0.10 seconds to be compatible with the 1.0 second timestep used in the 2D domain of the model.

The review concluded with a second Memo ("L.19202.000.01" dated 8th March 2013) which found that the model was adequately calibrated to the 2012 rainfall event, had acceptable convergence and was suitable for use in identifying and mapping overland flooding within the Wetherill Park catchment. Full details of this Memo are provided in **Appendix G**.

A further review of the model was conducted by Cardno Pty Ltd (known as Cardno) in the first stage of producing a Catchment Management Plan for the Wetherill Park catchment. Cardno was engaged by Fairfield City Council through the Local Government Procurement Panel and undertook a review of the existing hydraulic model before beginning the Catchment Management Plan. During this review Cardno did not find that any major rectifications were needed to the model, but they did identify a number of areas where minor model refinements could be undertaken. This included pipe network connections, ID channel schematisation, culvert and bridge modelling methodology, 1D/2D connections, and the inclusion of a noise barrier wall that exists along Cowpasture Road and had not been included in the model.

After these areas had been assessed and minor modifications made to the hydraulic model, slightly altered depth and velocity maps were produced which led to updated flood precinct mapping. In general, the depth results were within +- 0.2m at calibration locations, indicating reasonable correlation with the previous model. Reductions in water levels of up to 0.5m and 0.35m were identified upstream of Toohey Road and Victoria Road, respectively.

The revised mapping has replaced the initial mapping that was created and is now included within this report to be adopted by Council. The Hydraulic Model Update Summary Report produced by Cardno dated 8 January 2015 is included within this report as **Appendix I**.

3.6. Model Calibration

3.6.1. Overview

Calibration and validation against recorded data is an integral step in the flood modelling process and is required to provide a measure of confidence in model results. OEH's generic flood study brief states that where sufficient data is available, hydrologic and hydraulic models should be calibrated and validated using data from at least three historical flood events.

The calibration process generally involves inputting recorded rainfall and/or streamflow data from a flood event into a model and comparing model results against other corresponding recorded data such as water levels and flow rates. In an iterative process the model setup and parameters are then altered, within acceptable bounds, to achieve a good correlation between model results and recorded data. Once calibrated, models should be validated by achieving simulation of a different flood level to an acceptable level of accuracy.

Unfortunately for this study, detailed model calibration and verification could not be carried out since direct measurements of flood levels in the catchment were not available for historical flood events.

Rudimentary model calibration was carried out to high water marks observed following the 18th April 2012 flood event. This model calibration is discussed below.

3.6.2. Flood Event of 18th April 2012

On the 18th April 2012, a moderate flood event occurred in the study area. Rainfall in one-minute increments was recorded at gauging stations at various locations near the Wetherill Park catchment (outlined in red) and in and around the Fairfield LGA (outlined in black) as shown in the figure below.

Figure 3-5 Location of Study Area and Available Rainfall Gauging Stations

An analysis of the rainfall was carried out to determine the duration of the event. A spreadsheet was used for each gauging station to determine the highest rainfall that fell for each standard storm duration, i.e. the highest rainfall depth for any 15 minute period, for any 30 minute period, for any 45 minute period and so on.

These rainfall depths were then converted to average rainfall intensities and then plotted on an Intensity-Frequency-Duration (IFD) Curve. The duration with the highest corresponding Average Recurrence Interval (ARI) was deemed to be the storm duration for that gauging station.

The analysis indicated that the 18th April 2012 event was best described as a 30 to 45 minute duration storm at most gauging stations. A summary of the maximum rainfall depths for the 30 minute and 45 minute periods at each gauging station, along with the approximated ARI of the storm is provided in Table 3-6 below.

Table 3-6 Maximum Rainfall Depths and Approximated ARIs for 18th April 2012 Flood Event

Gauge	Maximum Rainfall Depth (mm)		Approximate ARI (years)
	30 minute duration	45 minute duration	
GS067119 - Horsley Park Equestrian Centre	43.0	50.2	30 – 40
GS567169 - Abbotsbury (Fairfield City Farm)	29.5	40.0	10
GS567154 - Cabramatta Bowling Club	13.0	15.0	1
GS567077 - Fairfield STP	19.0	25.0	2 – 5
GS567083 - Prospect Reservoir	33.0	38.0	10
GS567079 - Guildford (Pipehead)	24.5	31.0	2 - 5

The analysis showed considerable variability in the recorded rainfall at the various gauges. The gauges closer to the study area, such as Horsley Park, Abbotsbury and the Prospect Reservoir, indicated a moderate flood event with the approximated ARI ranging from about 10 to just under 50 years. The gauges more remote from the study area, such as Cabramatta Bowling Club, Fairfield STP and Guildford, all recorded considerably less rainfall indicating a more minor event at these locations.

Due to this variability of rainfall, for the purposes of this calibration it was decided to run the model using an average of the recorded rainfall from the three gauges closest to the study area, namely Horsley Park, Prospect Reservoir and Abbotsbury. The cumulative rainfall recorded at each gauge, along with the average cumulative rainfall, is provided in Figure 3-6 below.

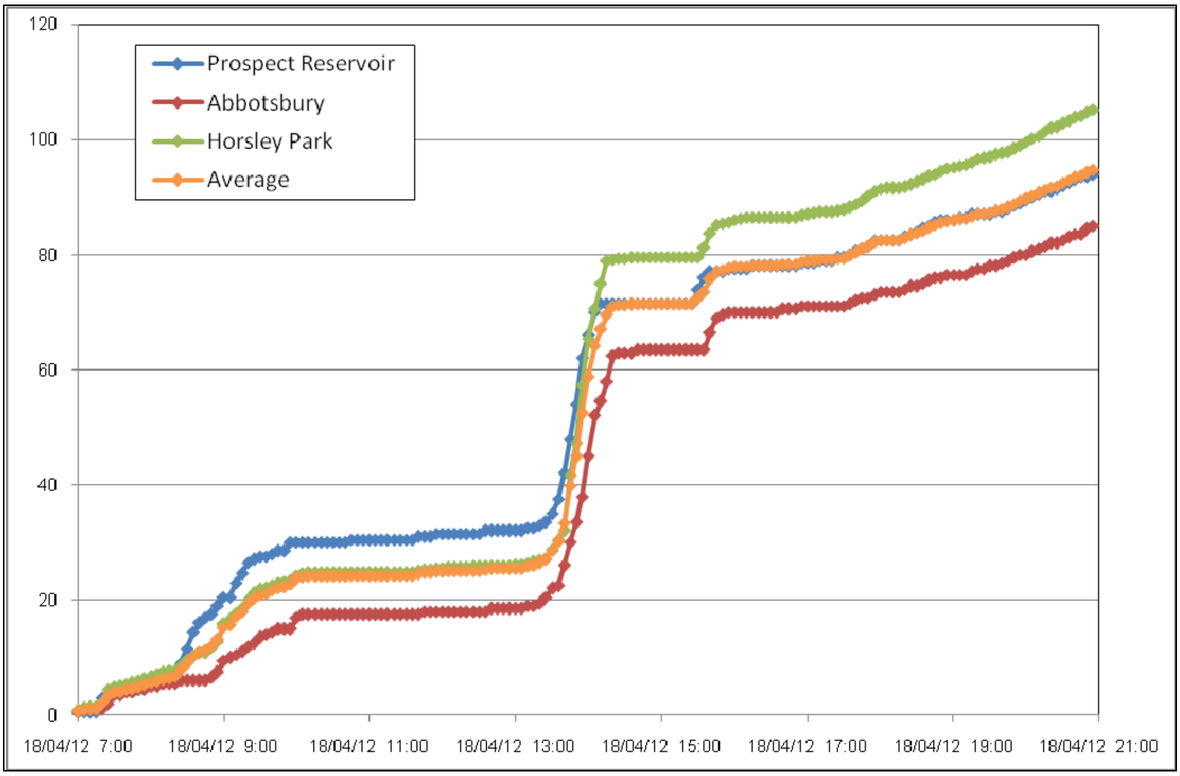


Figure 3-6 Cumulative Recorded Rainfall for 18th April 2012 Calibration Event

The above figure shows the most intense burst of rainfall occurred between approximately 1:20pm and 2:45pm on the afternoon of the 18th April. Hence the simulation was run for a five hour period using rainfall recorded on 18th April from 12 noon until 5pm.

High water marks were recorded and surveyed by Council staff at numerous locations in the study area. These high water marks, usually in the form of debris marks and flattened grass were typically observe0d at locations upstream and downstream of bridge and culvert crossings. The observed and modelled peak water levels at the high water mark locations for rainfall from each of the three gauges are presented and compared in Table 3.7.

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Table 3-7 Observed and Modelled Peak Water Levels for 18th April 2012 Flood Event

Location	High Water Level		
	Observed (m AHD)	Modelled (m AHD)	Difference (m)
Main Channel			
Cowpasture Rd/The Horsley Drive (culvert inlet)	57.07	57.18	0.11
Cowpasture Rd/The Horsley Drive (on roundabout)	57.73	57.71	-0.02
Tooheys Rd (culvert inlet)	51.52	51.43	-0.09
Hallstrom Place (bridge outlet)	48.25	48.15	-0.10
Newton Rd/Victoria Street (culvert outlet)	41.34	40.76	-0.58
Elizabeth St (bridge outlet)	35.86	35.70	-0.16
Snow Confectionary Pty Ltd (34 Davis Road)	32.68	32.66	-0.02
Tributary 1			
Newton Rd (culvert inlet)	42.35	42.37	0.02
Rosford Street			
Hassall St (culvert inlet)	26.23	26.46	0.23
Hassall St (culvert outlet)	25.80	25.89	0.09
Prospect Creek			
Gipps Rd (culvert outlet)	24.67	24.60	-0.07
Rosford Reserve (outlet)	22.53	22.71	0.18
Other Locations			
Emerson Street Reserve (in basin)	37.64	37.79	0.15

Long profiles are provided in Figures 3.7 to 3.9, with the modelled and observed peak water levels for each of the three concrete-lined open channels.

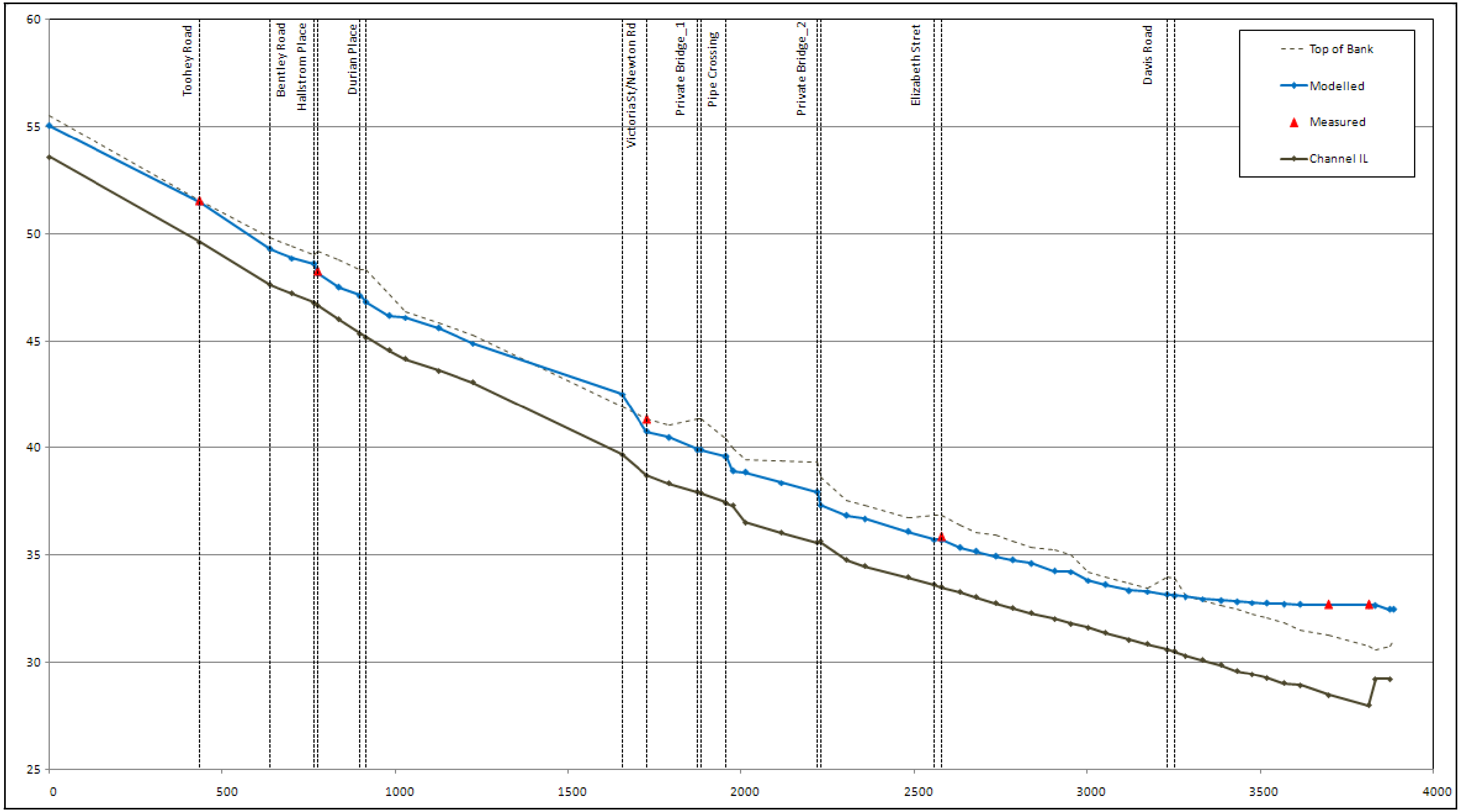


Figure 3-7 Main Channel Long Profile for Measured and Modelled Flood Levels for 18th April 2012 Calibration Event

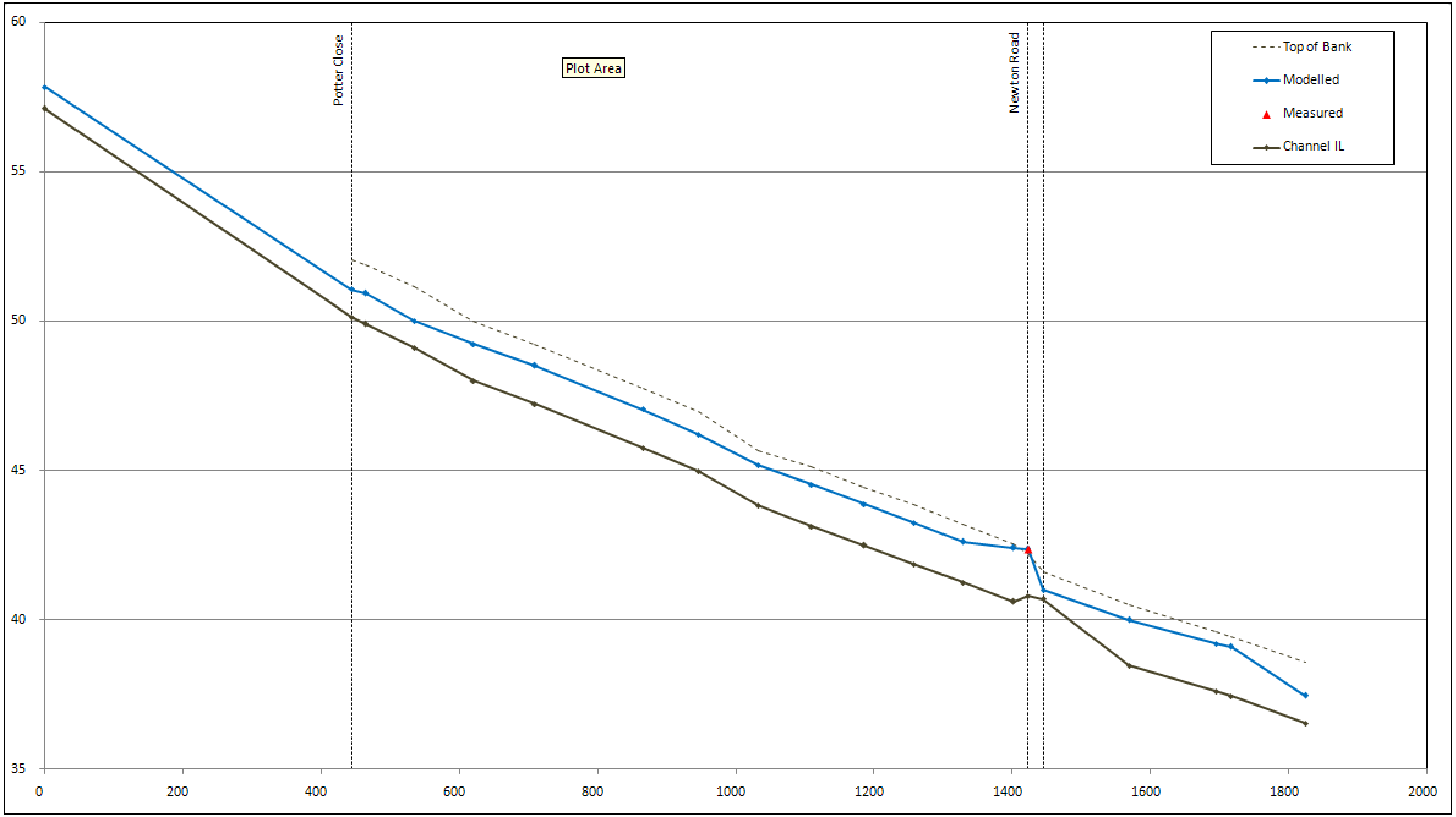


Figure 3-8 Tributary 1 Long Profile for Measured and Modelled Flood Levels for 18th April 2012 Calibration Event

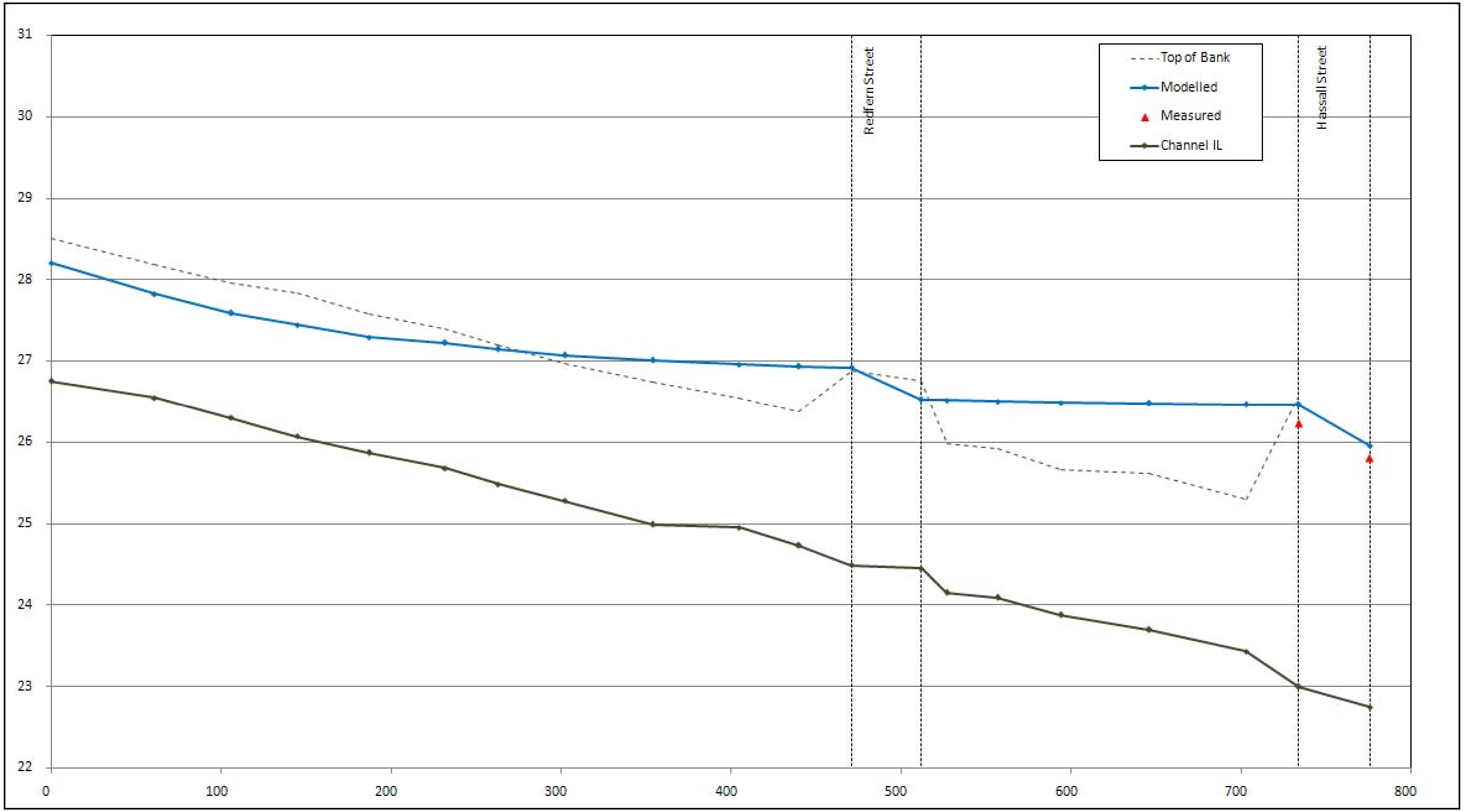


Figure 3-9 Rosford Channel Long Profile for Measured and Modelled Flood Levels for 18th April 2012 Calibration Event

The above table and figures show that the modelled water levels are generally within 200mm of the recorded high water levels, indicating a satisfactory model calibration.

The model could not be calibrated to the high water mark at outlet of the Victoria Street/Newton Road culverts on the Main Channel, where the model was found to under-predict peak water levels by 0.58m. The model predicted extremely high velocities (in the order of 5m/s) both within these culverts and in the channel immediately downstream. These velocities, which were calculated in the 1D component of the model, are a vertically and horizontally averaged velocity in the particular culvert or channel reach. In reality however, higher velocities are expected to occur near the centre of the channel with lower velocities at the edges, and where there are instances of relatively high velocities such as those observed at the outlets of the Victoria Street/Newton Road culverts, this effect is expected to be more pronounced.

A reduction in the velocity near the channel edges would result in a corresponding increase in the peak flood level. For example, if the velocity at the channel edges was, for example 2m/s lower than in the channel centre, the corresponding drop in velocity head ($V^2/2g$) could easily result in an increase in water levels in the order of 0.5m. The 1D model cannot account for these lateral variations in water level and velocity and hence the under-prediction of the peak water level at the outlet of the Victoria Street/Newton Road culverts is considered a reasonable result.

3.7. Comparison with Hydrologic Model

3.7.1. Background

Since detailed calibration and verification could not be carried out for this study, a comparison with an independent hydrologic model was made for a few selected sub-catchments in the study area. As far as practically possible, sub-catchments are selected that satisfy the following requirements:

- A range of different land uses are included;
- A range of different terrain slopes are included;
- Each sub-catchment can be considered in isolation with limited influence from neighbouring sub-catchments. Sub-catchments located in the upper reaches of the study area are more likely to meet this requirement;
- Few or no hydraulic structures or controls such as bridges, culverts and weirs are included, to allow a realistic comparison between hydrologic and hydraulic models.

The hydrographs generated by the independent hydrologic model are compared to those calculated by the hydraulic model for the corresponding catchment. Note that when considering flows from the hydraulic model, both the underground pipe flow and the overland flow component need to be included. The key outputs to be compared are the peak flow, the volume of runoff, timing of the peak flow and the overall shape of the hydrograph.

Note that it is not always expected that the two models will match. However, where there are differences, some interpretation of the results can be made, and the model can be checked as to why this is the case. Engineering judgement and experience should be employed to determine appropriate adjustment of model parameters. Conventional values of hydrologic modelling parameters such as Initial and Continuing Loss may not necessarily be applicable to a corresponding model using the Direct Rain method.

3.7.2. Results

As part of the "*Review of Prospect Creek Flood Levels*" (Cardno Willing, 2004) an XP-RAFTS model of the Prospect Creek catchment was developed based on RAFTS models from previous studies. This model was subsequently reviewed in 2006 by Bewsher Consulting as part of the Prospect Creek Floodplain Management Plan, with certain modelling inputs and assumptions varied and flow estimates subsequently revised.

This latest XP-RAFTS model was used in this study. The average of the recorded rainfall from the gauges at Horsley Park, Prospect Reservoir and Abbotsbury for the 18th April 2012 event was used as input to the model and the calculated discharge hydrographs were compared to those of the TUFLOW model.

The following sections provide details of the sub-catchments selected for analysis and the comparison of the discharge hydrographs calculated by both models.

3.7.2.1. Sub-Catchment #1

The first sub-catchment selected for analysis corresponds to sub-catchment H1 in the RAFTS model. This sub-catchment, of area 49.4ha, is located around The Horsley Drive in the vicinity of Laguna Place and the Rennie Street service road. The sub-catchment drains in a roughly northerly direction, and it contains both industrial land use (north of The Horsley Drive) and residential land use (south of The Horsley Drive).

The location of Sub-Catchment#1 is provided in Figure 3-10 below, with the sub-catchment boundary depicted in red and the underground stormwater pipes in yellow.

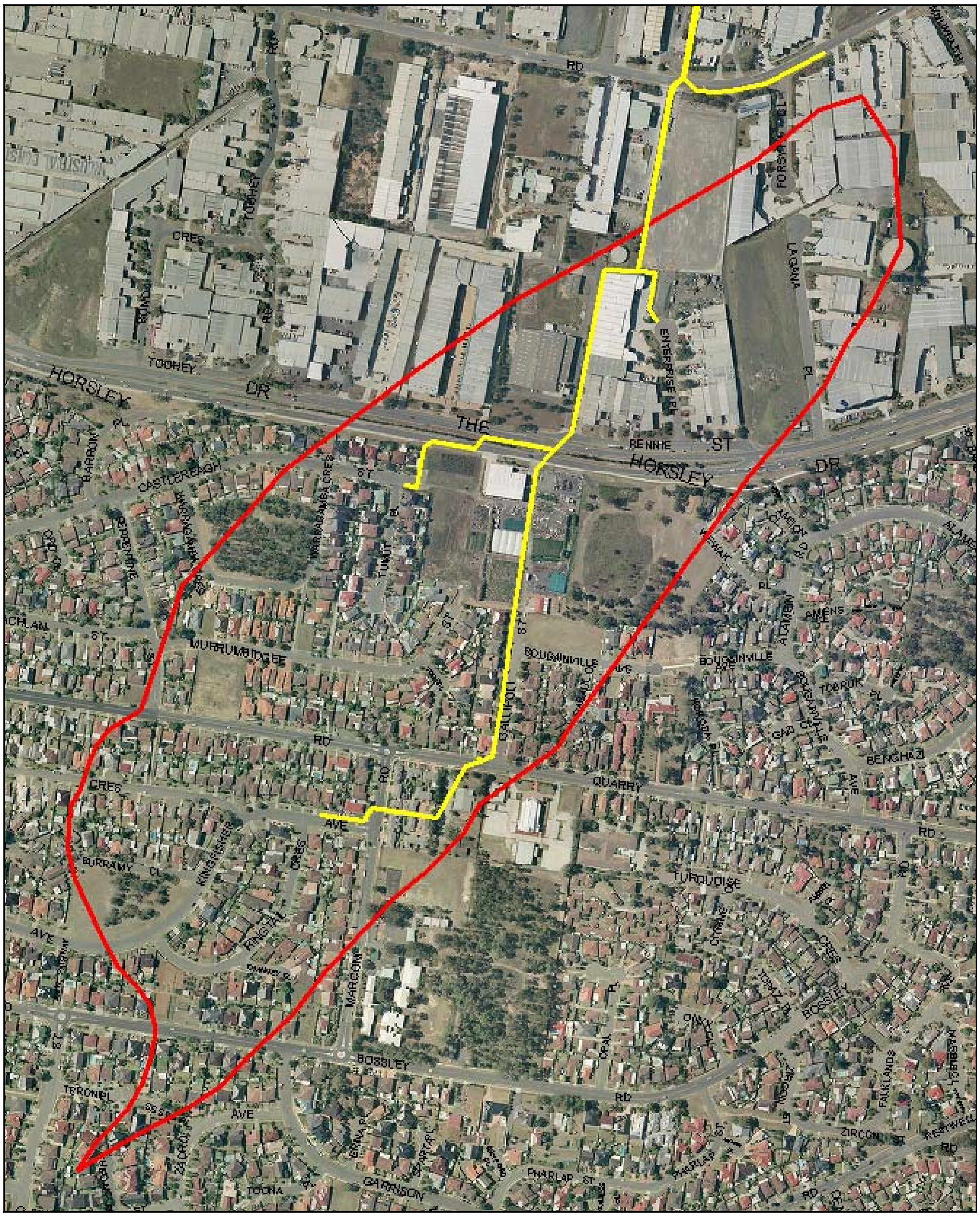


Figure 3-10 Location of Sub-Catchment#1

In order to calculate the discharge hydrograph for the TUFLOW model, flows in the pipe reaches 1581_030-020 and 1500_030-020 (parallel and indiscernible from each other in Figure 3-10) were added to the 2D overland flows crossing the north-west boundary of the sub-catchment.

A comparison of the discharge hydrographs calculated by the XP-RAFTS hydrologic model and the TUFLOW model is provided in Figure 3-11 and Table 3-8 below.

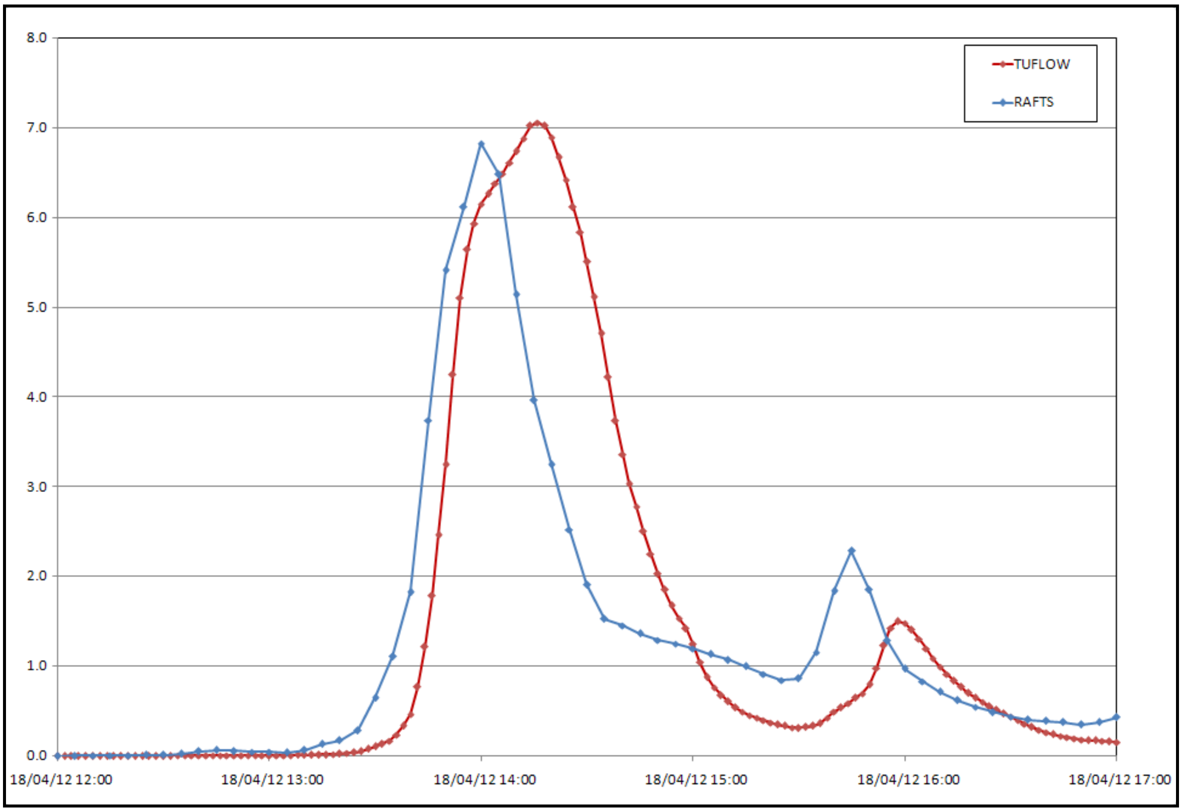


Figure 3-11 Comparison of Model Results for Sub-Catchment #1

Table 3-8 Comparison of Model Results for Sub-Catchment #1

Location	**Model Results**		
	Peak Discharge (m^3/s)	**Time of Peak**	**Volume (m^3)**
RAFTS MODEL			
Sub-catchment H1	**6.8**	**18-Apr-12 14:00**	**23 370**
TUFLOW MODEL			
Overland Flow	2.4		
Pipe 1581_030-020	1.5		
Pipe 1500_030-020	3.3		
TOTAL	**7.1**	**18-Apr-12 14:16**	**25 631**

3.7.2.2. Sub-Catchment #2

The second sub-catchment selected for analysis corresponds to sub-catchment A3 in the RAFTS model. This sub-catchment is located around intersection of The Horsley Drive and Cowpasture Road and drains in a roughly north-easterly direction. The sub-catchment contains mainly industrial land, with small components of residential land in the south and rural land in the west.

The location of Sub-Catchment#2 is provided in Figure 3-12 below, with the sub-catchment boundary depicted in red, the underground stormwater pipes in yellow and the concrete-lined open channel in dark blue.

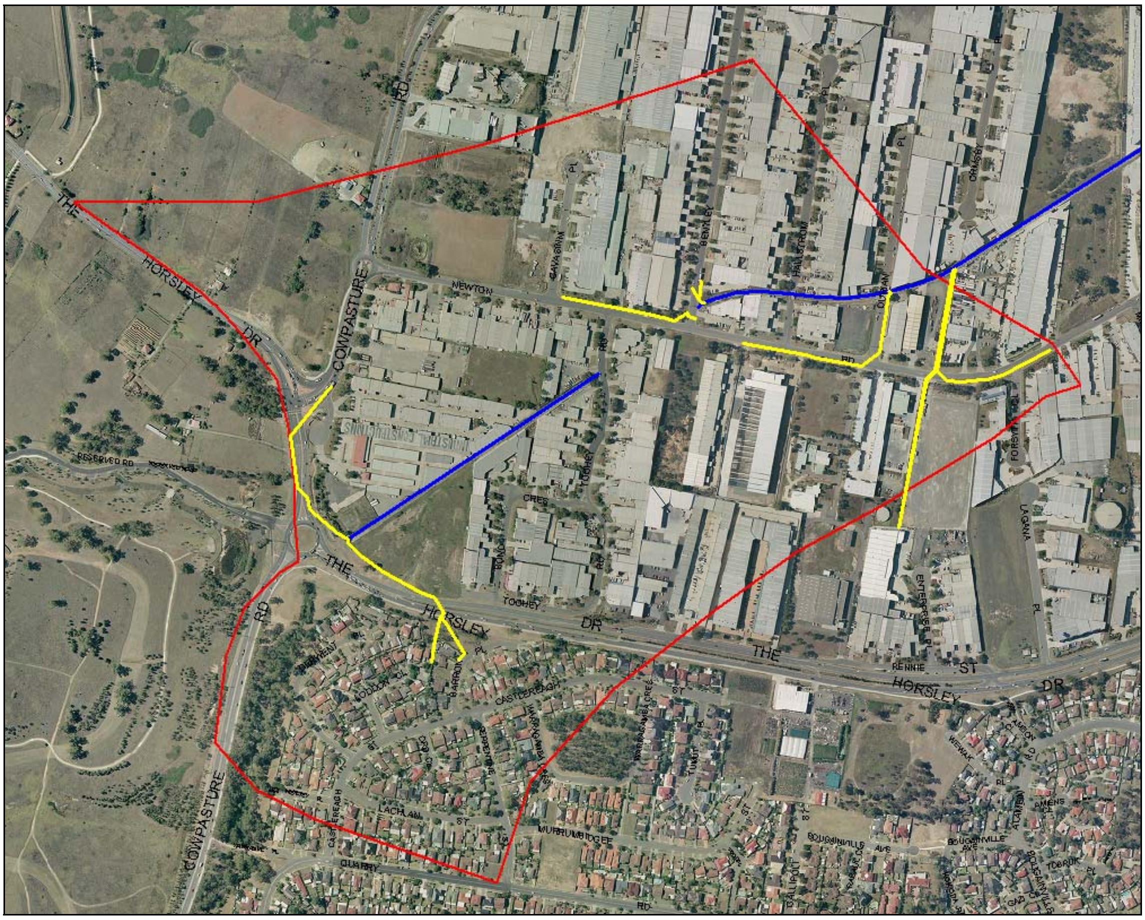


Figure 3-12 Location of Sub-Catchment#2

In order to calculate the discharge hydrograph for the TUFLOW model, flows in the pipe reaches 1500_010-000 and 1510_010-000 (parallel and indiscernible from each other in Figure 3-12) as well as flows in the open channel reach MC_42B were added to the 2D overland flows crossing the north-east boundary of the sub-catchment.

A comparison of the discharge hydrographs calculated by the XP-RAFTS hydrologic model and the TUFLOW model is provided in Figure 3-13 and Table 3-9 below.

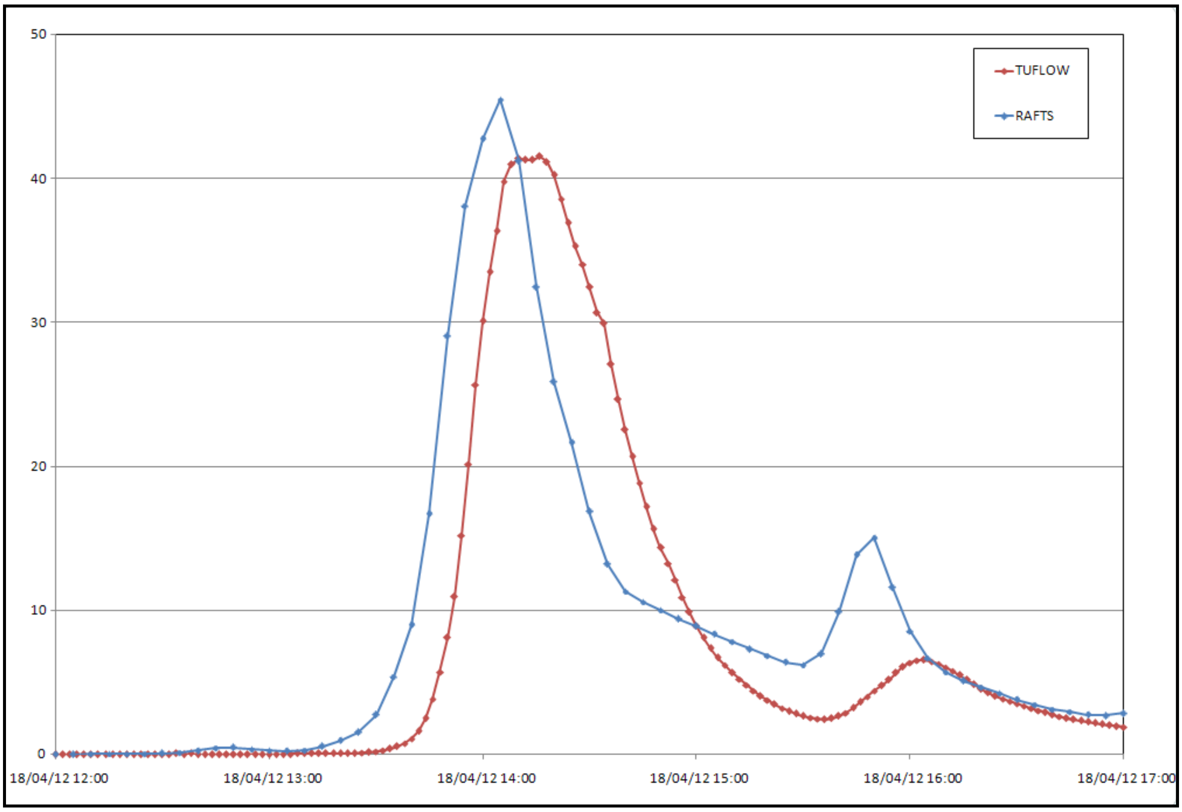


Figure 3-13 Comparison of Model Results for Sub-Catchment #2

Table 3-9 Comparison of Model Results for Sub-Catchment #2

Location	**Model Results**		
	Peak Discharge (m^3/s)	**Time of Peak**	**Volume (m^3)**
RAFTS MODEL			
Sub-catchment A3	**45.2**	**18-Apr-12 14:04**	**162 899**
TUFLOW MODEL			
Overland Flow	2.1		
Pipe 1500_010-000	3.2		
Pipe 1510_010-000	3.0		
Channel MC_42B	34.2		
TOTAL	**41.8**	**18-Apr-12 14:16**	**147 440**

3.7.2.3. Sub-Catchment #3

The third and final sub-catchment selected for analysis is the catchment for the Emerson Street basin, which corresponds to sub-catchment F1-1 in the RAFTS model. This sub-catchment is located in the eastern part of the study area and is roughly bounded by The Horsley Drive in the north, Emerson Street in the east and extends almost to Polding Street in the south and Canley

Vale Road in the west. The sub-catchment contains almost entirely residential land and drains in a roughly north-easterly direction towards the Emerson Street basin.

The location of Sub-Catchment#3 is provided in Figure 3-14 below, with the sub-catchment boundary depicted in red and the underground stormwater pipes in yellow.

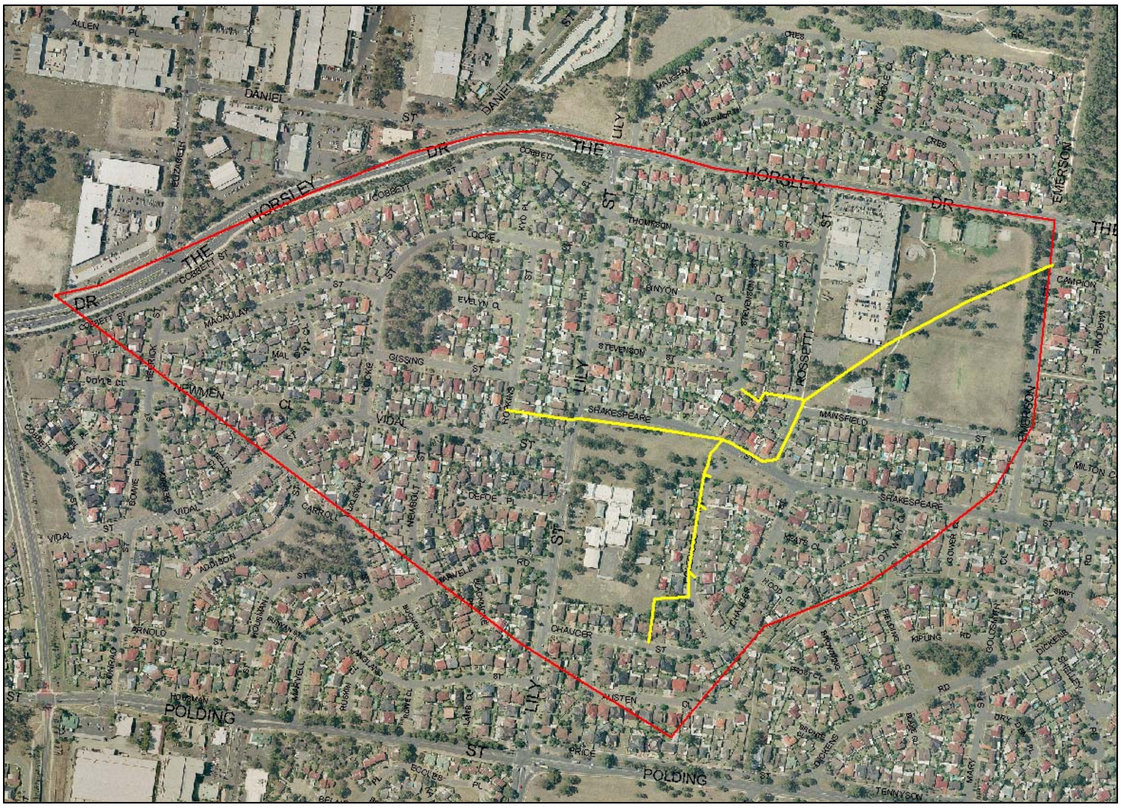


Figure 3-14 Location of Sub-Catchment#3

In order to calculate the discharge hydrograph for the TUFLOW model, flows in the basin outlet pipe 0700_105-090 were extracted from the model. It was not required to consider any 2D overland flows, since the basin did not overtop during the 18th April 2012 event.

It was noted in the received RAFTS model that the basin outlet pipe was assigned a gradient of 0.2%, considerably flatter than the 4% gradient obtained from the survey and used in the TUFLOW model. In order to allow a proper comparison of model results, the RAFTS model was rerun with the outlet pipe gradient adjusted to 4%.

A comparison of the discharge hydrographs calculated by the RAFTS hydrologic model and the TUFLOW model is provided in Figure 3-15 and Table 3-10 below.

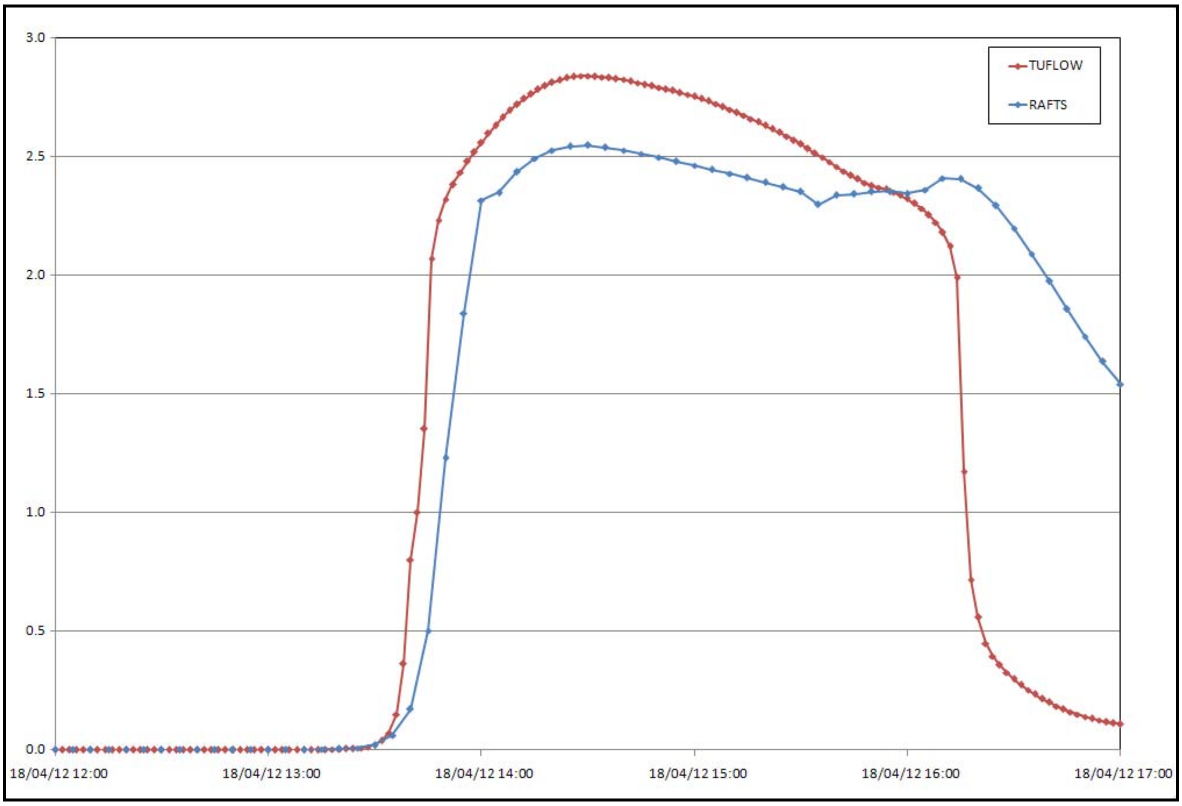


Figure 3-15 Comparison of Model Results for Sub-Catchment #3

Table 3-10 Comparison of Model Results for Sub-Catchment #3

Location	Model Results		
	Peak Discharge (m^3/s)	Time of Peak	Volume (m^3)
RAFTS MODEL			
Sub-catchment F1	2.5	18-Apr-12 14:30	26 476
TUFLOW MODEL			
Pipe 0700_040-030	2.8	18-Apr-12 14:30	24 529

3.7.2.4. Discussion

The above results indicate that for all three sub-catchments considered, good agreement was achieved between the RAFTS and TUFLOW results with respect to the magnitude and timing of the discharge peaks. The hydrograph volumes were also in reasonable agreement, with total volumes calculated by the two models within 15% of each other for all three sub-catchments.

It is therefore considered that reasonable agreement between the TUFLOW model and the independent hydrologic model was achieved and no adjustment of parameters to the TUFLOW model was made.

3.8. Comparison with Previous Flood Studies

A comparison of the results of this study with those from the *Flood Risk Assessment for 449 Victoria Street, Wetherill Park (Cardno 2011)* was undertaken, with peak flood levels for the 1:100 year ARI, 2 hour design event extracted from both models for the reach of the Main Channel between Durian Place and the Victoria Street/Newton Road intersection.

Peak flood levels in Cardno's study were found, on average, to be 0.25m higher than those of this study. However since peak discharges in Cardno's model were also generally slightly higher (approximately 75 m^3/s, compared to approximately 68 m^3/s) and also used a different downstream boundary condition and simplified cross sections, the result is considered reasonable.

3.9. Sensitivity Analysis

3.9.1. Overview

Sensitivity analyses were conducted to determine the sensitivity of the flood behaviour to variations in the adopted model parameters. The following scenarios were assessed for the 1:100 year ARI, 2 hour duration event:

- Catchment surface roughness;
- Varied rainfall losses;
- Stormwater pit blockage; and,
- Varied Water Surface Slope at Downstream Boundary.

The resulting flood depths were compared to the design 100 year ARI flood depths. The results are discussed below.

3.9.2. Impact of Varied Catchment Roughness

The impact of variations in hydraulic roughness was investigated. The value of Manning's *n* roughness was increased and decreased by 5% from the base case in both the 1D domain (roughness of the concrete-lined open channels) as well as in the 2D domain.

Flood depths are not sensitive to a 5% increase in catchment roughness. The change in flood level depth is typically less than 20mm with the exception of a few tiny, isolated locations where the change is marginally higher.

3.9.3. Impact of Varied Rainfall Losses

The impact of variations in rainfall losses was investigated. The rainfall losses adopted for the sensitivity analysis are tabulated below. Note that since zero Initial and Continuing Losses were assigned to impervious areas, only the pervious area losses were varied in this analysis.

Table 3-11 Rainfall Losses Adopted in Sensitivity Analysis

	Initial Loss (mm)	Continuing Loss (mm/hr)
Base Case	10	2.5
Increased Loss	15	3.5
Decreased Loss	5	1.5

Flood behaviour displayed relatively low sensitivity to the choice of Initial and Continuing Loss. The largest variations in peak waterlevel were noted in the vicinity of the concrete-lined open channels, particularly at the inlets of both the Main Channel and Tributary 1, where peak water levels varied by 0.10-0.20m and by 0.05-0.10m respectively. Smaller variations of 0.05m-0.10m were noted at a few other locations such as near the Toohey Road/Newton Road intersection and the Victoria Street/Newton Road intersection on the Main Channel and near Potter Close on Tributary 1. At almost all other locations in the study area, the variation in peak water level was less than 0.05m.

3.9.4. Impact of Increased Stormwater Pit Blockage

The impact of increased stormwater pit blockage was investigated. The blockage factor was increased from the design value of 50% blocked in the 100 year ARI event to 75% blocked (i.e. the pit inlet has 25% capacity of an unblocked pit inlet).

Flood behaviour was found to be largely insensitive to increased pit inlet blockage in the catchment, with peak waterlevels not changed by more than 0.10m with the exception of a few isolated locations where very slightly higher increases were observed.

3.9.5. Impact of Varied Water Surface Slope at Downstream Boundary

As stated earlier, the Stage-Discharge (HQ) curve used as the downstream boundary condition was derived using an assumed water surface slope of 0.5%. The impact of varying the assumed water surface slope was investigated by performing sensitivity runs using a HQ curve based on an assumed water slope of 0.25% and 1.0%.

The varied water surface slope resulted in differences in peak water level from the base case of 20-30cm at the downstream boundary of the model. These differences became considerably smaller at relatively short distances upstream and were negligible at the outlet of the Rosford detention basin.

It was therefore concluded that the impact of the choice of water surface slope for the HQ boundary condition is limited to a very short reach of Prospect Creek downstream of the Rosford detention basin and had no impact on flood levels in the other parts of the study area.

3.9.6. Conclusions from Sensitivity Analyses

In summary, flood behaviour in the model is not sensitive to variations in the selected parameters such as Manning’s n, rainfall losses, degree of pit blockage and tailwater conditions. Uncertainties about these parameters therefore are not likely to affect the outcomes of any overland floodplain management measures which are implemented.

4. Flood Model Results

4.1. Processing of Results

Once calibration and validation was complete, the TUFLOW model was run for the 20, 100, 500, and 10,000 year ARI and PMF events for a range of storm durations from 25 minutes to 6 hours. The peak water level for each storm duration at each grid point in the model of the catchment was extracted and used to form a 'peak of peaks' grid of flood depth and velocity.

The initial flood depth maps were then refined in order to provide the most relevant and useful information. Isolated patches and minor fingers of shallow depth flooding of less than 150mm were manually removed. The rationale for this is that such areas could be considered as areas of "nuisance" or "localised" flooding caused by local drainage rather than actual overland flooding. For example, ponding of stormwater within the roadway may not be a part of the main body of overland flows.

Further refinements were then made to the grid to address the following issues:

- The application of the Direct Rainfall method had necessitated the use of a cut-off depth of 100mm for mapping purposes. This resulted in small discontinuities in flood levels in some locations, where depths of greater than 100mm were seen to reduce to a zero depth (i.e. to the ground surface elevation) over a very small distance;
- The use of a 5m cell size in the 2D domain had resulted in somewhat coarse, granular flood extents.

The interpolation function in Vertical Mapper was used to restore the removed 100mm depth of flooding by extrapolating the flood grids. The "Inverse Distance Weighting" tool was used with a cell size of 1m and a search/display radius of 10m. This resulted in more complete flood extents with many areas of less than 100mm flood depth now included in the mapping, as well as a considerably finer resolution (reduced from 5m to 1m) in the flood extents.

Detailed flood depth and velocity mapping for the 1:20 and 1:100 year ARI and PMF events are included in **Appendix C** and **Appendix D.** This mapping is the revised mapping produced by Cardno.

4.2. Overview of Flood Behaviour

The following findings on flood behaviour in the study area have been drawn from analysis of the model results and flood depth and velocity mapping. The discussion focuses largely on the flood extents of the 1:100 year ARI extent with some details of other flood events provided where appropriate.

4.2.1. Draining into the Main Channel

Residential Area South of The Horsley Drive

- An overland flow path traverses in a roughly north-westerly direction along Tallowood Crescent, Kanuka Street, Bossley Road, Quarry Road and then in a westerly direction parallel to and along Mulgara Place and into the north-flowing gully alongside Cowpasture Road. The

1:100 year ARI flow largely follows the road layout up to the Bossley Road/Quarry Road roundabout. North-west of the roundabout, numerous properties are inundated, with depths of up to 0.5m observed for the 1:100 year ARI event and depths over 1.0m for the PMF event.

- The Cardno review of the model also found that a sound barrier wall at the rear of properties along Cowplasture Road has a significant effect on overland flow. The wall effectively blocks overland flow from the branch originating east of Jarrah Place resulting in increased ponding at the properties on Mulgara Place adjacent to the wall (up to 0.44m). This water effectively drains away via the existing pipe network. As the noise wall temporarily retains a portion of the overland flow, decreases in 100 yr water levels downstream are evident. More details can be found in **Appendix I.**
- For extreme events, an overland flowpath originates near the northwest corner of Bettong Crescent and follows the underground drainage line north-west to Quarry Road and then to the Cowpasture Road gully. Depths of up to 1.0m were observed at some properties for the PMF event.
- An overland flow path traverses in a roughly northerly direction along Serpentine Street and Barron Place, before flowing overland to The Horsley Drive. The flow is largely confined to the roads for the 1:100 year ARI event, with velocities of up to 2.0m/s occurring. Some flooding to adjacent properties on Serpentine Street and Barron Place was evident for the PMF event.
- An overland flow path traverses in an easterly direction along Kingfisher Avenue, over Marconi Road and through residential properties with depths of up to 1.0m observed for the 1:100 year ARI event. The flow path then continues north along Gallipoli Street and overland through Wewak Place Park to The Horsley Drive.
- North of The Horsley Drive, the path continues to follow the underground drainage line, inundating a number of industrial properties and over Newton Road before continuing overland and inundating further industrial properties between Newton Road and the Main Channel. Model results indicated that the bulk of the 1:100 flows in this location did not directly discharge to the Main Channel, but ponded in this area before being collected by the inlet pits on Newton Road and discharging via the twin 1.65m diameter pipes to the Main Channel.

Industrial Area Upstream of Tributary 1 Confluence

- Modelling results indicated that the reach of the Main Channel between The Horsley Drive/Cowpasture Road roundabout and the Tributary 1 confluence can largely convey the 1:100 year ARI flow within its banks, with the exception of surcharging at the inlets of the Toohey Road culverts and the Victoria Street/Newton Road culverts.
- Considerable surcharging was noted at the inlet of the Toohey Road/Bentley Road culvert, with flood depths for the 1:100 ARI event of up to 1m observed at the road crossing. Floodwaters surcharging from the culvert inlet flowed north along Toohey Road and then east along Newton Road to meet the overland flow from south of The Horsley Drive (the Kingfisher Avenue/Marconi Road/Gallipoli Street flowpath). Flow velocities along Newton Road between Toohey Road and Durian Place were up to 2.0m/s for the 1:100 year ARI event and were reasonably contained to the road boundary, but this was not the case for the PMF, where depths of greater than 1.0m occur in properties on both sides of the street.
- An overland flow path traverses west along Allen Place to the cul-de-sac, where it continues through an industrial property to meet the flow path traversing north along Canley Vale Road. The flow path continues north of Victoria Street and through industrial properties towards the

Main Channel. Model results indicated that the bulk of the 1:100 flows in this location did not directly discharge to the Main Channel, but ponded in this area before being collected by the inlet pits on Victoria Street and discharging via the 1.80m diameter pipe to the Main Channel.

- Considerable surcharging was noted at the inlet of the Victoria Street/Newton Road culverts, with the bulk of the flow for the 1:100 year ARI event breaking out on the left bank, flowing over Victoria Street and continuing north, inundating industrial properties on both the west and east side of Newton Road. Flow depths of up to 0.5m and velocities in excess of 1.0m/s were noted for the 1:100 year ARI event in this area. For the PMF event, severe flooding was noted at this intersection, with extensive areas having depths well in excess of 1.0m.

Industrial Area Downstream of Tributary 1 Confluence

- Modelling results indicated that the reach of the Main Channel from the confluence with Tributary 1 to the Davis Road crossing could largely convey the 1:100 year ARI flow within its banks. Downstream of Davis Road and towards the confluence with Prospect Creek, large areas on both banks are inundated.
- An overland flow path traverses in a northerly direction from Victoria Street just east of its intersection with Elizabeth Street, and follows the underground drainage line through a number of industrial properties and crossing Centre Place and Frank Street before draining into the Main Channel. Model results indicated that the bulk of the 1:100 flows in this location did not directly discharge to the Main Channel, but ponded in Frank Street before being collected by the inlet pits on Frank Street and discharging via the 2.1m diameter pipe to the Main Channel. Ponding depths of up to 0.5m were noted in Frank Street for the 1:100 year ARI event.
- An overland flow path traverses in from beyond the western end of Davis Road in an easterly direction to its crossing with the Main Channel. The flow largely follows the road with depths generally between 0.3m and 0.5m and velocities of up to 2.0m/s for the 1:100 year ARI event and some inundation noted to adjacent industrial properties. Model results indicated that the bulk of the 1:100 flows from this flowpath did not directly discharge to the Main Channel, but ponded near the eastern end of Davis Road before being collected by the nearby inlet pits and discharging via the 1.8m diameter pipe to the Main Channel. Ponding depths of up to 1.0m were noted at the eastern end of Davis Road for the 1:100 year ARI event.

4.2.2. Draining into Tributary 1

Industrial Area Draining to Tributary 1

- Modelling results indicated that Tributary 1 could largely convey the 1:100 year ARI flow within its banks, with the exception of some surcharging at the inlet of the Newton Road culverts.
- Considerable surcharging was noted at the inlet of the Cowpasture Road culverts, with flow overtopping Cowpasture Road and inundating the industrial property downstream before continuing east to Potter Close where it discharges to Tributary 1. Depths on Potter Close were generally less than 0.5m for the 1:100 year ARI event but exceeded 1.0m for the PMF event.
- An overland flow path traverses from near the northern end of Cowpasture Road, following the underground drainage line in a roughly south-easterly direction through industrial properties and over Newton Road, before continuing to follow the drainage line in a south-easterly

direction through further industrial properties to Coates Place. At this point, it is joined by an east-flowing overland flow path from Sleigh Place and then continues south before eventually draining to Tributary 1. Depths on the flow path for the 1:100 year ARI event are generally below 0.3m, except for a few locations where depths upwards of 0.5m occur and velocities of up to 1.5m/s were observed.

4.2.3. Draining into the Rosford Channel

Residential Area South of Victoria Street

- An overland flowpath traverses along Coleridge Road to its northern end where it is joined by an east flowing overland flow path from Shakespeare Street. Both overland flow paths follow the main drainage lines and are largely confined to the road for the 1:100 year ARI event. The flowpath then continues overland in a roughly north-easterly direction, inundating a number of residential properties between Shakespeare Street and the intersection of Mansfield Street and Rossetti Street. Flood depths of up to 1.0m for the 1:100 year ARI event were noted in this area. The flow path then continues north-east before discharging into the Emerson Street basin. For the PMF event, the flowpaths along Coleridge Road and Shakespeare Street extend upstream to Chaucer Street and upstream of Hopkins Street respectively, with the latter resulting in inundation to numerous residential properties on Vidal Street, Gissing Street and Hopkins Street. Flood depths in excess of 1.0m at the inlet pits on Hopkins Street for the PMF event were observed.
- The Emerson Street basin collects discharge from the flowpath described in the previous paragraph. Modelling results indicate that the basin can contain flows for floods up to and including the 1:500 year ARI events, but overtops for the 1:10,000 year ARI and PMF events.
- An overland flowpath originates from the northern end of Maugham Crescent and traverses in a roughly south-easterly direction through a number of residential properties to Ainsworth Crescent. Depths of up to 0.50m for the 1:100 year ARI event and up to 1.0m for the PMF event were noted in this area. The flowpath then continues along Ainsworth Crescent in an easterly direction, with velocities exceeding 2.0m/s for the 1:100 year ARI event, before continuing overland through a number of residential properties between Ainsworth Crescent and Emerson Street before reaching The Horsley Drive, where is joins flows discharged from the Emerson Street basin.
- An overland flowpath originates from near the southern end of Dickens Road and continues largely within the road extents in a roughly north-easterly direction to its intersection with Shakespeare Street where a number of residential properties are inundated. North of Shakespeare Street, the path continues north-east along Wordsworth Street where it is joined by another overland flowpath traversing north along Swinborne Crescent. Velocities for the 1:100 year ARI event exceed 2.0m/s at some locations on Wordsworth Street. This flowpath then joins with the flowpath from Ainsworth Crescent and the flows discharging from the Emerson Street basin and inundate numerous residential properties between Campion Street/Ibsen Place and The Horsley Drive. Depths were typically up to 0.3m for the1:100 year ARI event and for the PMF were generally greater than 0.5m and exceeded 1.0m in some areas.
- After crossing The Horsley Drive, the flowpath continued north inundating numerous residential properties on both sides of Haywood Close, with flow depths of up to 0.5m for the 1:100 year ARI event and well in excess of 1.0m for the PMF event. From the northern end of

Haywood Close, the flowpath crossed a small reserve before flowing over Victoria Street and then discharging to the Rosford Channel.

Residential Area North of Victoria Street

- Modelling results indicated that the reach of the Rosford Channel between Victoria Street and Redfern Street can convey the 1:100 year ARI flow with only minor inundation on either bank. Some flooding was observed at the western end of Chifley Street, with depths of up to 0.3m for the 1:100 year ARI flood. Severe inundation occurred for the PMF event, particularly on the channel's eastern bank, where flood depths of up to 1.0m were observed on residential properties between Galton and Chifley Street and to industrial properties between Chifley Street and Redfern Street.
- An overland flowpath originates from near the western end of Redfern Road for the 1:100 year ARI event and flows east following the underground drainage line. At a point approximately 300m west of the Rosford Channel where the drainage line turns north, ponding occurs on the road and within the industrial property to the immediate north. Flood waters are stored here before eventually being collected in the underground drainage and discharging to the Rosford Channel.
- Surcharging was noted at the inlet of the Redfern Street culverts, resulting in flow breakouts over Victoria Street and inundation of industrial properties north of Redfern Street. Flow depths of up to 0.3m occurred for the 1:100 year ARI event.
- Considerable surcharging was noted at the inlet of the Hassall Street culverts, resulting in flow breakouts and inundation of industrial properties on both channel banks. Flow depths of up to 0.5m occurred for the 1:100 year ARI event.
- An overland flow path traverses east along the north-east section of Hassall Street and joins with another overland flow path traversing north along Blackstone Street, then north-east along Lennox Place and then overland to Hassall Street. Numerous industrial properties between Lennox Place and Hassall Street are inundated, with depths of up to 0.5m observed for the 1:100 year ARI event.

4.2.4. Peak Flood Levels

The design peak water levels for the critical storm duration for each ARI flood are provided as long profiles for each of the three concrete-lined open channels in Figures 4-1 to 4-3 and in tabular form in Appendix E.

The critical storm duration for the concrete-lined open channels was generally found to be the 2 hour duration storm for the design events up to the PMF and the 45 minute duration for the PMF storm.

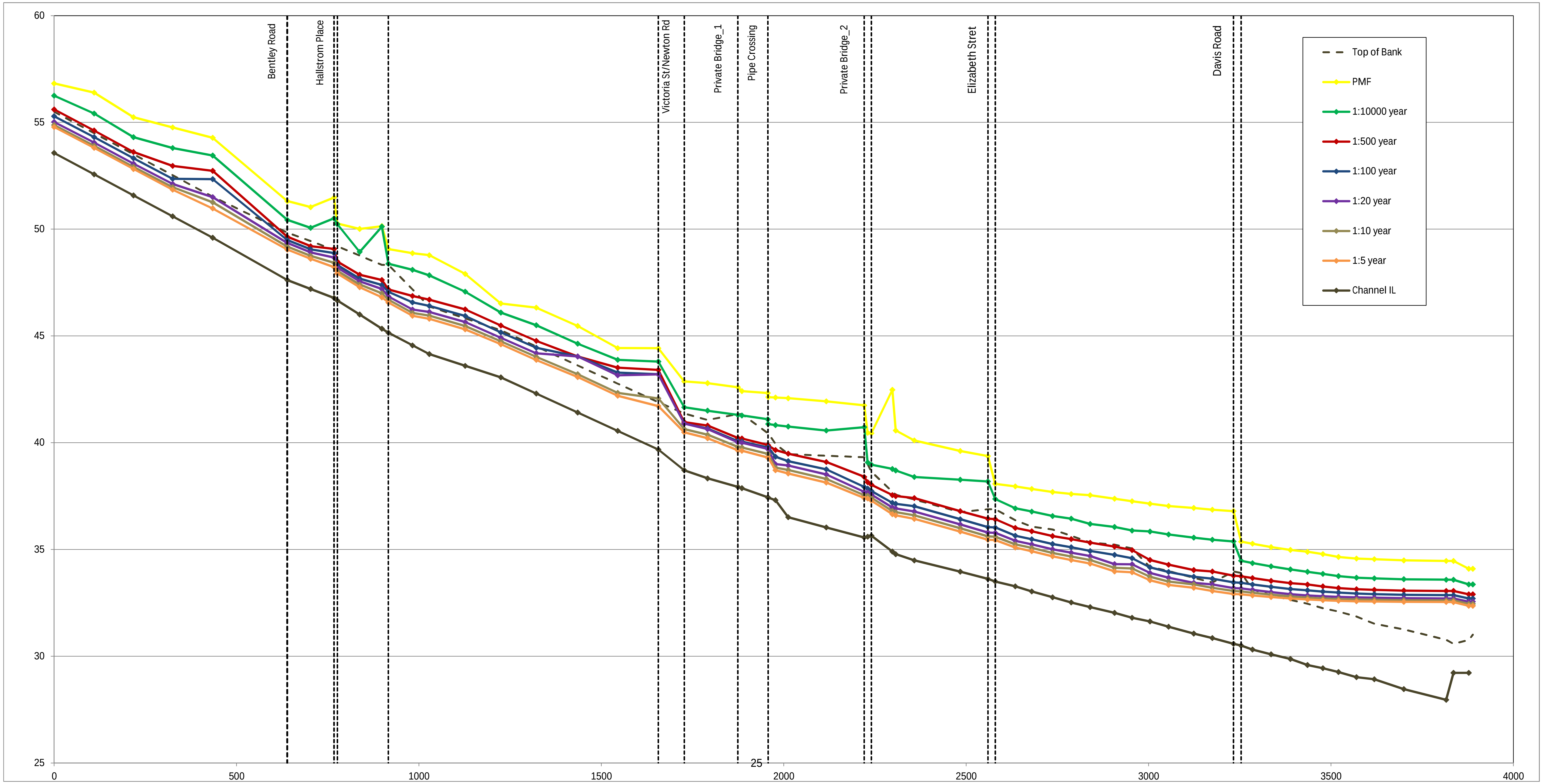


Figure 4-1 Design Flood Levels - Main Channel Long Profile

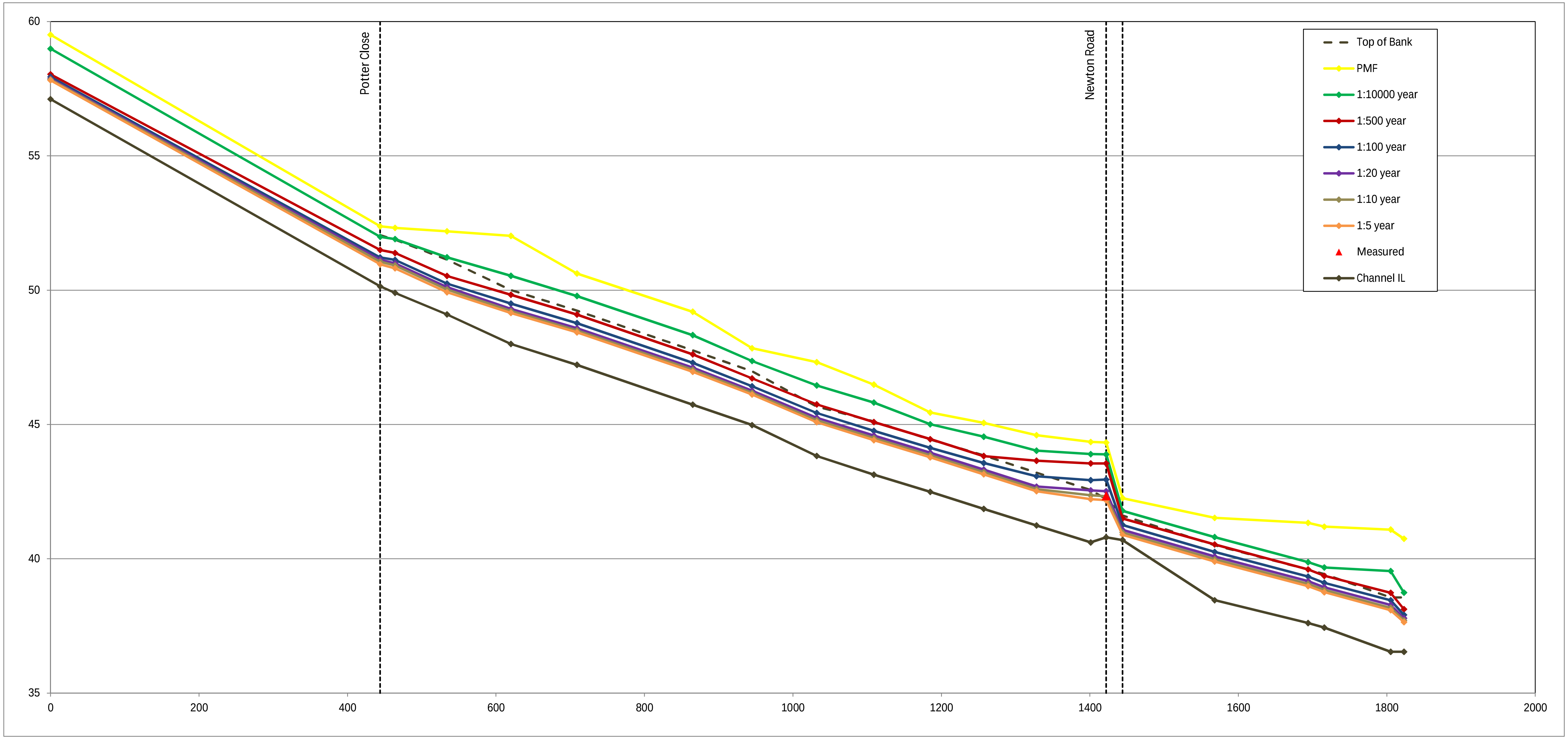


Figure 4-2 Design Flood Levels – Tributary 1 Long Profile

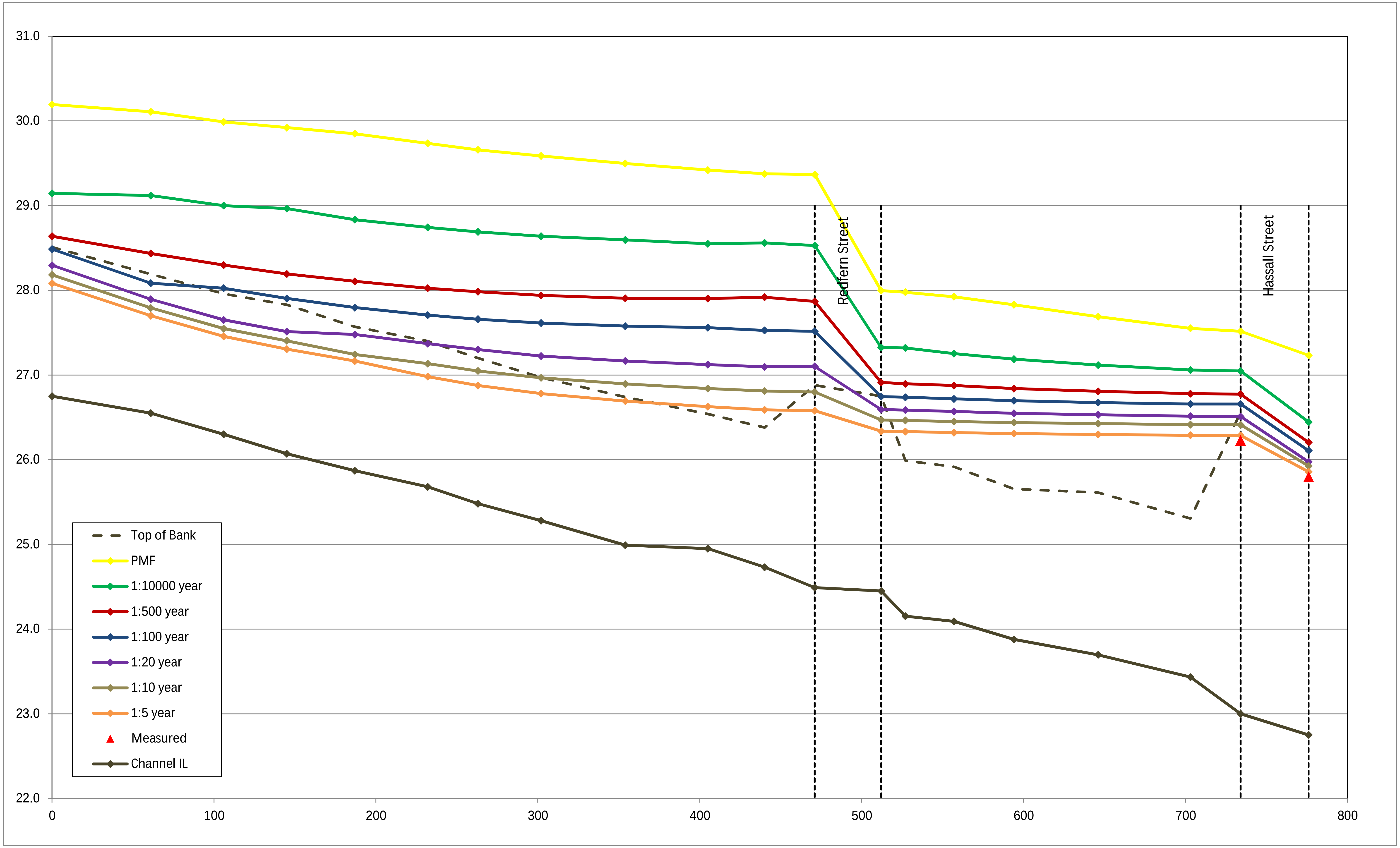


Figure 4-3 Design Flood Levels - Rosford Channel Long Profile

4.3. Flood Risk Precincts

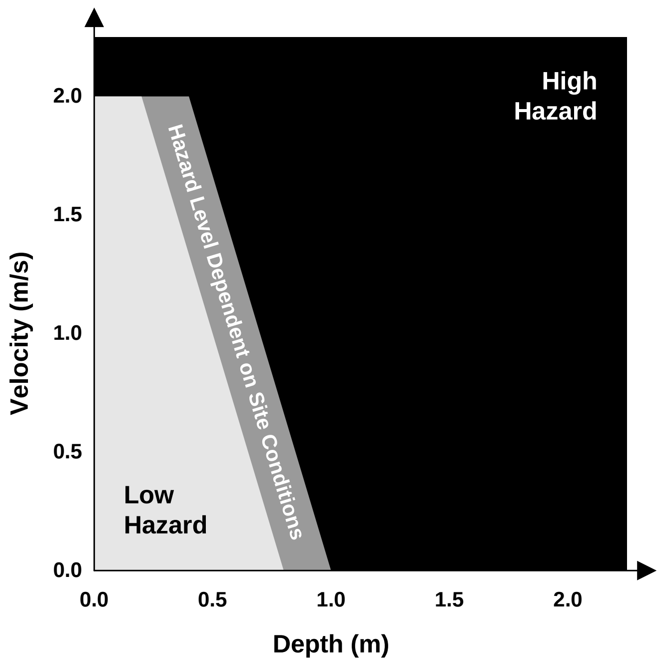
Flood risk precinct mapping has been prepared for the Wetherill Park catchment and is included in **Appendix F.** The flood risk precinct mapping contained within this report is the revised mapping produced by Cardno. The Cardno summary report can be found in **Appendix I.** The flood risk maps were developed from GIS analysis and interpretation of the 1:100 year ARI and PMF event peak depth and velocity grids, based on the FCC flood risk precinct categories described in **Table 4-1.** The flood risk precinct definitions were derived from the hydraulic hazard category diagram presented in the FDM, shown in **Figure 4-4**.

Table 4-1 FCC Flood Risk Precincts (Fairfield City Wide DCP, 2006)

Risk Precinct	Description
High	The area of land below the 100 year ARI flood outline that is subject to high hydraulic hazard (for preparation of the draft flood risk precincts, this has been taken as the provisional 'High Hazard' zone Figure L2 of Appendix L in the NSW Floodplain Development Manual (2005) as reproduced in **Figure 4-4**.
Medium	Land below the 100 year ARI flood outline that is not in the High Risk Flood Precinct
Low	All other land within the floodplain (i.e. within the extent of the PMF) but not identified within either the High Risk or Medium Risk Precincts.

Figure 4-4 Hydraulic Hazard Category Diagram

(reproduced from Figure 6-1 in NSW Floodplain Development Manual)



The flood risk precinct maps show solid precinct outlines, which have been reviewed and refined with consideration of flood evacuation requirements and other floodplain risk management issues. This has included some smoothing of the flood extent to account for local irregularities in the modelled ground surface, and street and property outlines.

The Fairfield City Wide DCP requires areas which were initially assigned a medium flood risk rating but are surrounded by the high risk precinct to also be upgraded to a high flood risk. Issues relating to the evacuation of these areas, which may become cut off during flood events, necessitates that they be allocated a high flood risk. Such areas were not identified in the Wetherill Park catchment and hence no properties required their flood risk to be upgraded in this manner.

The flood risk of islands of low or no flood risk are not required to be upgraded, in accordance with the DCP.

The flood risk mapping identified the following about the extents of the precincts:

- Approximately 1489 properties are within the floodplain outline defined by the PMF event. This includes:
  - 138 parcels in the high (or partially high) risk precinct;
  - 431 parcels in the medium (or partially medium) risk precinct;
  - 920 parcels in the low (or partially low) risk precinct.

- The significant areas of high flood risk occur in the following locations:
  - along the lengths of all three concrete-lined open channels;
  - in the natural gully parallel to Cowpasture Road between Bossley Road and The Horsley Drive;
  - in the natural gully flowing east from the rural area, draining to the Cowpasture Road/The Horsley Drive culverts;
  - in the natural gully flowing south-east from the rural area, draining to the Cowpasture Road/Victoria Street culverts;
  - in the Emerson Street basin;
  - on Wordsworth Street, between Shakespeare Street and Milton Close, and also between Campion Street and The Horsley Drive;
  - north of The Horsley Drive, adjacent to Haywood Close;
  - the northern half of Haywood Close and over the open space to the inlet of the Rosford Channel;

- In the rural areas, the medium flood risk precinct follows the numerous natural gullies and drainage flow-paths leading to the inlets of the Main Channel and Tributary 1.
- In the urban and industrial areas, the medium flood risk precinct largely follows the pattern of the underground drainage system.
- The low flood risk precinct follows the outline of the medium flood risk precinct reasonably closely in most parts of the rural areas, owing to the relatively steep gullies and channels.
- For the remainder of the study area, the low flood risk precinct widens considerably from the outline of the medium flood risk precinct, and numerous additional overland flow paths and inundated areas appear. This is particularly pronounced in the industrial zone, where extensive flood-prone areas are noted around the Victoria Street/Newton Road roundabout and along Davis Road and Redfern Road.

The extent of the flood risk precincts reflects the topography of the catchment. That is, the precincts are relatively narrow in the upper parts of the catchment to the south and west and spread out across the flatter, lower parts of the catchment in the north and east. These features of the topography explain the close similarity of the medium and low risk precincts in the upper catchment and why the low risk precinct outline spreads comparatively further in the lower parts of the catchment.

5. Conclusions

The Wetherill Park Overland Flood Study has been successful in achieving its objectives, which were to:

- Define flood behaviour and identify the major overland flow paths within the Wetherill Park catchment; and
- Identify properties at risk of local overland flooding and to prepare flood risk precinct maps.

The study's modelling approach consisted the development of a TUFLOW model to represent catchment hydrology and the 2D floodplain including topography, surface roughness and boundary conditions, then further development of the TULFOW model to represent the open channels and selected pits and pipes of the drainage network.

Rudimentary calibration to the high water marks observed for the 18^{th} April 2012 flood event was successfully carried out, as was a comparison against an independent hydrologic model which gave greater confidence in model results.

Sensitivity analysis indicated that the overland flow behaviour is typically not sensitive to variation in floodplain roughness, downstream tailwater conditions, varied rainfall losses or the degree of drainage pit blockage. Hence, overland flow estimates are not expected to be significantly impacted by uncertainties in these parameters.

The flood extent and risk precinct mapping were prepared to present only the areas which are affected by significant levels of overland flooding. Raw model results were refined by removing isolated patches and minor fingers of shallow flooding depth deemed to be "nuisance" or "localised" flooding caused by local drainage rather than actual overland flooding. The merit of this approach is that properties which are within or adjacent to these areas are not unduly coded with a flood risk.

The overland flood risk precinct delineation process clearly and objectively defines the level of flood affectation of each part of the study area. Consideration of the flood event ARI in determining the flood risk, in addition to the hydraulic hazard posed by flood events to life and property, is particularly appropriate for the industrial and urban setting of much of the study area. By definition it provides an indication of the probability of a property being flood affected during a given time frame, in addition to the degree of hazard that it would experience.

The study has ultimately provided a good foundation from which to prepare the floodplain risk management study and plan as the next step in the floodplain risk management process.

6. References

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Appendix A Model Stormwater Pit and Pipe Data

Table A-1 Wetherill Park Catchment Stormwater Pit Data

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
0070_180	Sag	GP 0.60m x 0.60m	45.80	43.23
0070_170	Sag	GP 0.60m x 0.60m	45.27	41.67
0070_160	OnGrade	Hornsby 3.0m lintel	43.98	41.13
0070_150	OnGrade	Hornsby 1.8m lintel	42.07	40.67
0070_140	OnGrade	Hornsby 1.8m lintel	41.30	39.75
0070_130	OnGrade	Hornsby 1.8m lintel	41.19	38.34
0070_120	OnGrade	Hornsby 1.8m lintel	39.56	37.36
0070_100	OnGrade	GP 0.40m x 0.80m	38.45	36.40
0070_050	Sag	GP 0.60m x 0.60m	36.11	31.94
0070_030	Sag	Hornsby 1.8m lintel	26.63	24.43
0070_020	Sag	Hornsby 1.8m lintel	26.57	23.84
0070_010	OnGrade	Hornsby 1.8m lintel	25.31	22.79
1770_070	OnGrade	GP 0.50m x 0.90m	36.11	34.21
1770_050	OnGrade	Hornsby 1.8m lintel	34.01	31.59
1770_040	OnGrade	Hornsby 1.8m lintel	33.08	31.18
1770_030	OnGrade	Hornsby 1.8m lintel	32.19	30.42
1770_020	OnGrade	Hornsby 1.8m lintel	31.37	29.07
1770_010	OnGrade	GP 0.40m x 0.90m	30.75	28.90
0400_030	OnGrade	Hornsby 1.8m lintel	36.34	34.32
0400_025	OnGrade	Hornsby 1.8m lintel	35.81	33.93
0400_020	Sag	Hornsby 1.8m lintel	35.39	33.49
0410_040	OnGrade	Hornsby 1.8m lintel	33.19	32.09
0630_310	OnGrade	Hornsby 1.8m lintel	40.72	38.91
0630_300	OnGrade	Hornsby 1.8m lintel	40.14	37.88
0630_290	OnGrade	GP 0.85m x 0.40m	39.68	37.68
0630_280	OnGrade	Hornsby 1.8m lintel	38.99	37.01
0630_270	OnGrade	Hornsby 1.8m lintel	38.82	36.77
0630_260	OnGrade	Hornsby 1.8m lintel	38.50	36.47
0630_250	OnGrade	Hornsby 1.8m lintel	37.72	36.00
0630_240	OnGrade	Hornsby 1.8m lintel	37.17	34.85
0630_230	OnGrade	Hornsby 1.8m lintel	35.95	34.20
0630_220	Sag	Hornsby 1.8m lintel	35.81	34.01
0630_210	OnGrade	Hornsby 1.8m lintel	35.65	33.67
0630_200	OnGrade	Hornsby 1.8m lintel	35.45	33.47
0630_190	OnGrade	Hornsby 1.8m lintel	35.08	33.06
0630_180	Sag	Hornsby 2.4m lintel	34.97	32.97
0630_170	OnGrade	Hornsby 1.8m lintel	34.86	32.79
0630_160	OnGrade	Hornsby 1.8m lintel	34.53	32.49

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
0630_150	OnGrade	GP 0.40m x 0.80m	34.01	32.06
0630_140	OnGrade	Hornsby 1.8m lintel	33.47	31.61
0630_100	Sag	Hornsby 1.8m lintel	33.64	29.62
0630_070	OnGrade	Hornsby 1.8m lintel	31.83	28.32
0630_040	Sag	Hornsby 1.8m lintel	31.02	27.38
0630_030	Sag	Hornsby 4.2m lintel	30.02	27.08
0630_020	Sag	Hornsby 4.2m lintel	30.01	27.07
0630_010	Sag	Hornsby 4.2m lintel	29.99	27.05
1390_120	Sag	Hornsby 1.8m lintel	38.22	36.04
1390_110	OnGrade	Hornsby 1.8m lintel	35.88	34.68
1390_100	Sag	Hornsby 2.4m lintel	35.68	34.40
1390_090	OnGrade	Hornsby 1.8m lintel	35.81	34.26
1390_080	OnGrade	Hornsby 1.8m lintel	35.05	33.47
1390_070	Sag	Hornsby 2.4m lintel	34.67	33.20
1390_060	Sag	Hornsby 2.4m lintel	34.64	33.19
1390_050	OnGrade	Hornsby 1.8m lintel	34.72	33.00
1390_040	OnGrade	Hornsby 1.8m lintel	34.17	32.55
1390_020	OnGrade	Hornsby 1.8m lintel	33.61	31.74
1390_010	Sag	Hornsby 1.8m lintel	33.38	31.44
1390_005	OnGrade	Hornsby 1.8m lintel	33.61	29.66
0700_220	Sag	Hornsby 2.4m lintel	46.54	45.16
0700_210	Sag	Hornsby 1.8m lintel	46.54	44.90
0700_200	Sag	Hornsby 1.8m lintel	45.74	44.10
0700_190	OnGrade	Hornsby 2.4m lintel	45.63	43.89
0700_180	Sag	GP 0.80m x 1.00m	45.09	42.95
0700_170	Sag	GP 0.40m x 0.90m	43.40	41.30
0700_160	Sag	Hornsby 3.6m lintel	41.70	39.39
0700_150	OnGrade	GP 0.45m x 0.85m	42.76	38.96
0700_140	OnGrade	Hornsby 2.4m lintel	40.28	38.22
0700_120	OnGrade	GP 4.00m x 5.00m	38.09	36.42
0700_110	Sag	GP 0.90m x 0.90m	36.39	35.33
0700_105	OnGrade	GP 5.45m wide	35.70	34.52
0700_090	OnGrade	Hornsby 1.8m lintel	35.53	32.93
0700_080	OnGrade	Hornsby 1.8m lintel	35.63	32.95
0700_070	OnGrade	Hornsby 1.8m lintel	35.75	32.85
0700_060	OnGrade	Hornsby 1.8m lintel	35.52	32.46
0700_050	OnGrade	Hornsby 1.8m lintel	34.64	31.84
0700_040	OnGrade	Hornsby 1.8m lintel	33.74	31.08
0700_030	OnGrade	Hornsby 1.8m lintel	33.13	30.23
0700_020	OnGrade	Hornsby 1.8m lintel	32.91	30.01
0700_010	OnGrade	Hornsby 1.8m lintel	33.06	29.86
1170_090	Sag	Hornsby 3.0m lintel	48.24	46.94

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
1170_060	OnGrade	Hornsby 1.8m lintel	45.00	43.25
1170_050	OnGrade	Hornsby 1.8m lintel	44.47	42.87
1170_040	OnGrade	Hornsby 1.8m lintel	42.97	41.75
1170_030	OnGrade	Hornsby 1.8m lintel	42.11	40.86
1170_010	Sag	Hornsby 3.6m lintel	41.82	39.88
1020_030	Sag	GP 0.40m x 0.90m	42.35	40.77
1020_020	Sag	Hornsby 3.6m lintel	39.74	38.09
1020_010	OnGrade	Hornsby 2.4m lintel	39.80	38.14
0750_080	OnGrade	Hornsby 1.8m lintel	38.92	36.91
0750_065	Sag	Hornsby 1.8m lintel	38.08	36.26
0750_060	OnGrade	Hornsby 1.8m lintel	38.18	36.14
0750_020	Sag	GP 0.80m x 1.00m	37.19	34.24
0750_010	Sag	GP 0.80m x 1.00m	36.26	33.31
0760_040	Sag	Hornsby 1.8m lintel	43.41	41.56
0760_020	OnGrade	Hornsby 1.8m lintel	40.21	38.17
0760_016	OnGrade	GP 0.45m x 0.85m	39.06	37.34
0760_015	OnGrade	GP 0.45m x 0.85m	38.88	37.03
0770_020	OnGrade	GP 0.45m x 0.85m	42.54	41.19
0770_010	OnGrade	GP 0.45m x 0.85m	40.83	39.50
0950_010WP	OnGrade	Hornsby 1.8m lintel	35.90	35.00
1180_010	OnGrade	GP 0.85m x 0.40m	42.20	41.55
1190_010WP	OnGrade	Hornsby 1.8m lintel	42.98	42.08
1200_010WP	OnGrade	Hornsby 1.8m lintel	44.56	43.52
2060_040	Sag	Hornsby 2.4m lintel	33.48	31.83
2060_030	OnGrade	Hornsby 1.8m lintel	33.83	31.20
2060_010	OnGrade	GP 0.35m x 0.85m	34.79	30.39
0010_090	OnGrade	Hornsby 1.8m lintel	37.59	35.49
0010_080	OnGrade	Hornsby 1.8m lintel	36.68	34.90
0010_070	OnGrade	Hornsby 1.8m lintel	35.70	33.63
0010_050	OnGrade	Hornsby 1.8m lintel	35.12	32.99
0010_048	OnGrade	Hornsby 1.8m lintel	35.04	32.79
0010_030	OnGrade	Hornsby 1.8m lintel	34.31	31.77
0010_020	OnGrade	Hornsby 1.8m lintel	33.88	31.35
0010_010	Sag	Hornsby 2.4m lintel	33.64	31.08
15200_050	OnGrade	Hornsby 1.8m lintel	35.85	34.43
15200_040	OnGrade	Hornsby 1.8m lintel	35.16	33.34
15200_030	OnGrade	GP 0.35m x 0.35m	34.54	32.98
15200_020	OnGrade	Hornsby 1.8m lintel	34.00	32.23
15200_010	Sag	Hornsby 1.8m lintel	33.70	32.07
0100_030	OnGrade	Hornsby 1.8m lintel	36.20	35.20
0100_020	OnGrade	GP 0.40m x 0.80m	35.59	34.52

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
0100_010	Sag	Hornsby 1.8m lintel	34.96	33.73
0160_040	Sag	GP 0.45m x 0.45m	36.89	36.09
0160_030	Sag	GP 0.45m x 0.45m	36.78	36.01
0160_010	OnGrade	Hornsby 1.8m lintel	36.34	35.34
0180_020	Sag	GP 0.90m x 0.90m	38.10	36.84
0180_010	OnGrade	Hornsby 1.8m lintel	37.21	36.42
0130_010	Sag	Hornsby 1.8m lintel	35.18	33.57
0140_010	Sag	Hornsby 1.8m lintel	35.15	34.12
0120_010	OnGrade	Hornsby 1.8m lintel	35.19	34.37
0150_010	OnGrade	GP 0.90m x 0.60m	36.18	35.03
0200_010	OnGrade	GP 0.85m x 0.40m	38.95	37.07
15100_010	OnGrade	Hornsby 1.8m lintel	34.31	32.78
2100_120	OnGrade	GP 1.00m x 1.00m	48.60	42.46
2100_115	Sag	GP 0.80m x 0.70m	43.91	39.35
2100_085	OnGrade	Hornsby 1.8m lintel	41.19	37.78
2100_080	Sag	Hornsby 1.8m lintel	40.04	37.24
2100_070	OnGrade	GP 0.50m x 0.90m	40.61	36.58
2100_035	Sag	GP 0.90m x 0.90m	38.10	35.41
2100_030	OnGrade	Hornsby 0.9m lintel	38.35	34.78
2100_010	Sag	GP 0.45m x 0.45m	37.44	33.37
17000_010	Sag	Hornsby 3.0m lintel	40.06	38.13
2090_020	Sag	Hornsby 3.0m lintel	35.97	34.22
2090_015	Sag	GP 0.90m x 0.90m	35.80	33.36
0230_190	OnGrade	Hornsby 1.8m lintel	56.22	53.12
0230_180	OnGrade	Hornsby 2.4m lintel	52.96	51.74
0230_170	OnGrade	Hornsby 1.8m lintel	52.74	50.63
0230_160	Sag	GP 1.20m x 1.20m	52.25	49.14
0230_110	OnGrade	GP 0.60m x 0.60m	46.87	42.57
0230_105	Sag	Hornsby 1.8m lintel	45.97	42.41
0230_065	Sag	GP 0.70m x 0.70m	42.99	39.29
0230_060	Sag	GP 0.70m x 0.70m	42.98	39.19
0230_056	Sag	GP 0.70m x 0.70m	42.94	39.00
0230_030	OnGrade	GP 0.80m x 0.80m	38.34	35.00
0230_020	Sag	Hornsby 1.8m lintel	37.69	34.34
0230_010	Sag	GP 0.40m x 0.90m	37.70	34.09
0240_040	OnGrade	Hornsby 1.8m lintel	38.64	36.82
0240_030	Sag	Hornsby 3.0m lintel	37.32	35.60
0240_020	Sag	Hornsby 1.8m lintel	37.28	35.35
0240_010	OnGrade	Hornsby 1.8m lintel	37.61	34.86
17100_030	Sag	GP 0.80m x 1.00m	41.70	39.92
17100_020	Sag	GP 0.80m x 1.00m	41.49	39.53

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
17100_010	OnGrade	GP 0.85m x 0.40m	41.07	39.07
0350_010	Sag	Hornsby 2.4m lintel	52.29	50.99
0470_050	OnGrade	Hornsby 1.8m lintel	45.25	43.28
0470_040	OnGrade	Hornsby 1.8m lintel	44.50	42.86
0470_035	Sag	GP 0.40m x 0.90m	44.51	42.79
0470_026	Sag	Hornsby 1.8m lintel	44.22	41.98
0470_025	Sag	Hornsby 1.8m lintel	44.26	41.80
0480_010	OnGrade	Hornsby 1.8m lintel	43.56	40.79
0490_010	Sag	Hornsby 1.8m lintel	43.95	43.41
0510_050	OnGrade	Hornsby 1.8m lintel	49.83	47.58
0510_040	OnGrade	Hornsby 1.8m lintel	49.02	46.72
0510_030	OnGrade	Hornsby 1.8m lintel	48.00	45.37
0510_007	Sag	GP 0.90m x 0.60m	46.40	44.04
0540_010	OnGrade	Hornsby 1.8m lintel	49.25	46.95
1890_025	OnGrade	Hornsby 3.6m lintel	48.20	45.31
1890_020	Sag	GP 1.75m x 0.40m	47.66	45.28
1510_130	Sag	GP 0.90m x 0.60m	58.54	57.13
1510_120	OnGrade	Hornsby 1.8m lintel	58.45	56.29
1510_110	OnGrade	Hornsby 3.0m lintel	57.46	55.01
1510_100	OnGrade	Hornsby 3.0m lintel	55.91	52.77
1510_090	OnGrade	Hornsby 3.0m lintel	53.74	51.16
1510_080	OnGrade	Hornsby 1.8m lintel	51.36	48.92
1510_050	Sag	Hornsby 1.8m lintel	49.40	47.65
1510_045	OnGrade	Hornsby 1.8m lintel	48.94	46.80
1510_040	OnGrade	Hornsby 1.8m lintel	48.62	46.54
1510_035	OnGrade	GP 0.40m x 0.90m	48.53	46.50
1510_020	OnGrade	Hornsby 1.8m lintel	47.97	45.89
1510_010	Sag	GP 0.40m x 0.90m	47.84	45.16
1500_170	OnGrade	Hornsby 0.9m lintel	62.18	60.58
1500_160	OnGrade	Hornsby 0.9m lintel	61.92	60.32
1500_150	Sag	Hornsby 1.8m lintel	61.92	60.12
1500_140	OnGrade	GP 0.40m x 0.90m	62.02	59.82
1500_130	Sag	Hornsby 1.8m lintel	59.92	57.40
1500_120	OnGrade	Hornsby 1.8m lintel	59.87	57.24
1500_110	OnGrade	GP 0.40m x 0.90m	59.68	56.88
1500_100	OnGrade	GP 0.40m x 0.90m	57.54	55.63
1500_095	OnGrade	Hornsby 1.8m lintel	57.01	54.20
1500_090	OnGrade	Hornsby 2.4m lintel	56.27	53.35
1500_075	OnGrade	Hornsby 1.8m lintel	54.51	51.09
1500_050	Sag	Hornsby 3.0m lintel	53.64	50.42
1500_040	Sag	GP 1.10m x 0.75m	51.31	48.68

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
1500_030	Sag	GP 1.10m x 0.75m	50.92	47.72
1580_030	Sag	Hornsby 1.8m lintel	51.15	50.11
1620_020	OnGrade	Hornsby 0.9m lintel	62.58	61.03
1620_010	OnGrade	GP 0.60m x 0.60m	62.05	60.45
1620_007	OnGrade	Hornsby 2.4m lintel	58.93	57.45
1620_006	OnGrade	Hornsby 2.4m lintel	56.58	54.98
1620_005	Sag	GP 0.80m x 0.70m	56.61	54.69
1680_010	Sag	Hornsby 1.8m lintel	59.84	58.38
1581_020	OnGrade	GP 0.90m x 0.90m	48.39	46.09
1581_010	Sag	GP 0.90m x 0.60m	47.85	45.45
0860_060	Sag	Hornsby 2.4m lintel	41.27	39.19
0860_050	OnGrade	GP 0.40m x 0.90m	41.60	39.42
0860_040	OnGrade	Hornsby 2.4m lintel	41.58	38.95
0860_030	Sag	Hornsby 2.4m lintel	41.59	38.92
0870_020	Sag	Hornsby 2.4m lintel	41.27	39.49
0890_020	OnGrade	Hornsby 2.4m lintel	42.00	39.89
0890_010	OnGrade	Hornsby 1.8m lintel	42.08	39.88
0850_150	OnGrade	Hornsby 1.8m lintel	51.94	48.79
0850_130	OnGrade	Hornsby 1.8m lintel	51.72	48.72
0850_095	Sag	GP 0.90m x 0.90m	45.76	42.66
0850_092	OnGrade	Hornsby 3.0m lintel	45.68	42.08
0850_080	OnGrade	Hornsby 3.0m lintel	44.17	41.35
0850_070	OnGrade	Hornsby 3.0m lintel	43.50	40.32
0850_060	OnGrade	Hornsby 3.0m lintel	42.71	39.39
0850_050	OnGrade	Hornsby 3.0m lintel	42.24	38.58
0850_040	Sag	Hornsby 3.0m lintel	42.01	38.39
0850_020	Sag	Hornsby 1.8m lintel	41.06	37.25
1050_090	OnGrade	Hornsby 1.8m lintel	54.77	51.87
1050_086	OnGrade	Hornsby 1.8m lintel	54.41	51.37
1050_085	OnGrade	Hornsby 1.8m lintel	53.86	50.85
1050_084	OnGrade	Hornsby 1.8m lintel	53.23	50.13
1050_080	OnGrade	Hornsby 3.0m lintel	52.74	49.63
1050_078	OnGrade	GP 0.50m x 0.85m	52.23	49.47
1050_077	OnGrade	GP 0.50m x 0.85m	51.72	49.20
1050_076	OnGrade	GP 0.50m x 0.85m	51.54	49.09
1050_075	OnGrade	GP 0.50m x 0.85m	51.41	48.96
1050_070	OnGrade	Hornsby 3.0m lintel	51.33	48.95
1050_060	OnGrade	Hornsby 3.0m lintel	50.62	48.21
1050_050	OnGrade	GP 0.50m x 0.90m	49.92	47.32
1050_040	OnGrade	Hornsby 2.4m lintel	48.32	45.25
1050_030	OnGrade	GP 0.75m x 0.90m	47.81	44.71

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
1050_020	OnGrade	Hornsby 3.0m lintel	47.43	44.36
1450_030	OnGrade	Hornsby 1.8m lintel	55.95	53.01
1450_010	OnGrade	Hornsby 1.8m lintel	55.27	52.77
0950_010FH	OnGrade	Hornsby 1.8m lintel	43.32	42.53
0960_010	OnGrade	Hornsby 1.8m lintel	46.69	45.83
0990_010	OnGrade	Hornsby 2.4m lintel	42.92	41.67
1010_010	OnGrade	Hornsby 2.4m lintel	43.70	42.46
1020_010FH	OnGrade	Hornsby 1.8m lintel	44.26	42.68
1040_010	OnGrade	Hornsby 2.4m lintel	46.18	44.80
0910_010	OnGrade	Hornsby 1.8m lintel	41.23	37.50
0920_030	Sag	GP 0.80m x 0.70m	41.02	39.46
0920_020	Sag	GP 0.80m x 0.70m	41.01	39.44
16800_020	OnGrade	Hornsby 2.4m lintel	41.98	38.18
1250_060	OnGrade	Hornsby 1.8m lintel	47.22	45.54
1250_050	OnGrade	Hornsby 1.8m lintel	46.29	44.36
1250_040	OnGrade	Hornsby 1.8m lintel	46.04	44.14
1250_030	OnGrade	Hornsby 1.8m lintel	43.51	41.92
1250_020	Sag	Hornsby 1.8m lintel	42.55	40.12
1250_010	Sag	Hornsby 2.4m lintel	42.62	39.96
1330_030	OnGrade	Hornsby 1.8m lintel	49.45	47.09
1330_020	OnGrade	Hornsby 1.8m lintel	48.80	46.31
1250_150	Sag	Hornsby 1.8m lintel	55.86	54.54
1250_140	OnGrade	Hornsby 1.8m lintel	55.97	54.13
1250_120	Sag	GP 1.20m x 1.20m	55.35	52.49
1250_080	Sag	GP 0.90m x 0.60m	49.46	47.36
16600_010	OnGrade	Hornsby 1.8m lintel	56.94	55.20
14600_030	Sag	Hornsby 3.0m lintel	47.36	45.46
0560_010	Sag	Hornsby 3.6m lintel	46.46	45.32
0560_008	OnGrade	GP 0.90m x 0.60m	47.05	45.35
0560_005	OnGrade	Hornsby 1.2m lintel	46.76	44.96
0570_060	OnGrade	Hornsby 1.8m lintel	56.17	53.91
0570_050	Sag	Hornsby 1.8m lintel	55.41	52.73
0570_045	OnGrade	Hornsby 1.8m lintel	55.63	52.75
0570_040	OnGrade	Hornsby 1.8m lintel	56.04	51.04
0610_010	Sag	GP 0.90m x 0.60m	55.72	53.47
16500_010	Sag	Hornsby 2.4m lintel	54.48	53.13
0600_002	Sag	GP 0.60m x 0.60m	55.93	52.75
0580_020	OnGrade	GP 0.80m x 0.80m	57.21	53.41
0580_010	OnGrade	Hornsby 1.8m lintel	55.68	53.01
0590_050	Sag	GP 0.90m x 0.90m	60.40	59.23
0590_040	Sag	GP 0.90m x 0.90m	57.50	55.75

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
0590_030	Sag	GP 0.90m x 0.90m	57.15	55.59
0590_020	OnGrade	GP 1.40m x 0.85m	56.46	52.29
0590_010	Sag	Hornsby 1.8m lintel	55.53	52.12
16300_150	Sag	GP 0.90m x 0.90m	62.29	60.18
16300_140	Sag	GP 0.90m x 0.90m	60.89	59.33
16300_130	Sag	GP 0.90m x 0.90m	60.26	58.60
16300_110	OnGrade	GP 0.90m x 0.90m	59.00	57.07
16300_100	Sag	GP 1.50m x 1.00m	58.80	57.00
16300_090	OnGrade	GP 1.35m x 0.90m	57.95	55.82
0640_030	Sag	Hornsby 1.8m lintel	64.10	61.95
0640_020	Sag	Hornsby 1.8m lintel	63.99	61.19
0650_010	OnGrade	GP 0.50m x 0.90m	64.25	61.55
2150_030	Sag	Hornsby 1.8m lintel	61.35	59.95
2150_020	OnGrade	Hornsby 0.9m lintel	61.73	59.90
2150_015	Sag	GP 0.80m x 0.70m	60.87	59.17
2150_009	OnGrade	Hornsby 1.8m lintel	61.40	57.90
2150_008b	OnGrade	GP 0.80m x 1.00m	61.16	57.42
2150_008a	OnGrade	Hornsby 1.8m lintel	60.75	57.16
2150_005b	Sag	GP 0.40m x 0.90m	57.72	55.64
0730_760	OnGrade	GP 0.80m x 1.00m	58.01	57.19
0730_090	OnGrade	Hornsby 1.8m lintel	61.73	59.90
0730_080	OnGrade	Hornsby 1.8m lintel	60.87	59.17
0730_060	OnGrade	Hornsby 1.8m lintel	61.40	57.90
0730_050	Sag	Hornsby 1.8m lintel	61.16	57.42
0730_040	Sag	Hornsby 1.8m lintel	60.75	57.16
0730_030	OnGrade	Hornsby 1.8m lintel	59.41	56.75
0730_020	OnGrade	Hornsby 1.8m lintel	57.72	55.64
0730_010	Sag	Hornsby 1.8m lintel	57.57	54.82
0750_010FH	OnGrade	Hornsby 1.8m lintel	54.20	52.15
1190_050	OnGrade	Hornsby 1.8m lintel	45.97	44.83
1190_040	OnGrade	Hornsby 1.8m lintel	45.15	43.61
1190_030	OnGrade	Hornsby 1.8m lintel	44.31	42.35
1190_020	OnGrade	Hornsby 1.8m lintel	43.25	40.99
1190_010	Sag	Hornsby 2.4m lintel	42.31	40.61
1200_010	OnGrade	Hornsby 1.8m lintel	42.34	41.37
1210_010	OnGrade	Hornsby 1.8m lintel	43.64	41.64
1220_010	OnGrade	Hornsby 1.8m lintel	44.49	43.04
1230_010	OnGrade	Hornsby 1.8m lintel	45.30	43.97
1240_010	OnGrade	Hornsby 1.8m lintel	46.18	45.28
1820_100	Sag	Hornsby 1.8m lintel	38.45	36.45
1820_090	Sag	Hornsby 1.8m lintel	38.38	36.23

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
1820_070	OnGrade	Hornsby 1.8m lintel	36.57	33.67
1820_060	OnGrade	GP 0.40m x 0.90m	35.76	33.28
1820_050	OnGrade	Hornsby 1.8m lintel	34.96	32.71
1820_040	OnGrade	GP 0.40m x 0.90m	34.42	32.23
1820_030	OnGrade	Hornsby 1.8m lintel	33.77	31.59
1820_020	OnGrade	Hornsby 1.8m lintel	32.74	30.34
1820_005	OnGrade	Hornsby 1.8m lintel	31.57	30.07
0300_030	Sag	Hornsby 1.8m lintel	26.61	25.25
0440_040	Sag	GP 1.20m x 1.20m	44.00	41.83
0440_030	Sag	GP 1.20m x 1.20m	43.70	41.86
0440_020	Sag	GP 1.20m x 1.20m	43.07	41.37
1800_020	OnGrade	Hornsby 1.8m lintel	35.40	33.80
1800_010	OnGrade	Hornsby 2.4m lintel	35.23	33.43
2310_040	Sag	Hornsby 1.8m lintel	66.97	65.93
2310_038	Sag	Hornsby 1.8m lintel	66.86	65.56
2310_030	OnGrade	Hornsby 2.4m lintel	64.56	63.16
2310_020	Sag	Hornsby 1.8m lintel	64.42	62.82
2360_070	OnGrade	Hornsby 1.8m lintel	69.28	67.58
2360_060	Sag	Hornsby 1.8m lintel	68.16	66.56
2360_050	Sag	Hornsby 3.0m lintel	68.13	66.06
2390_010	Sag	Hornsby 1.8m lintel	66.05	64.75
2400_010	Sag	Hornsby 1.8m lintel	68.78	67.08
2380_030	Sag	Hornsby 1.8m lintel	67.51	66.55
2380_020	OnGrade	Hornsby 1.8m lintel	67.37	66.17
1960_020	OnGrade	Hornsby 1.8m lintel	51.52	49.25
1960_010	OnGrade	Hornsby 1.8m lintel	50.66	48.24
1970_010	OnGrade	Hornsby 1.8m lintel	51.30	50.36
17500_010	Sag	Hornsby 2.4m lintel	66.25	65.76
13600_050	OnGrade	GP 0.80m x 1.00m	66.99	64.77
13600_040	OnGrade	Hornsby 2.4m lintel	61.37	59.62
13600_030	OnGrade	Hornsby 2.4m lintel	60.55	58.68
13600_010	OnGrade	Hornsby 1.8m lintel	59.00	56.27
0700FH_50	OnGrade	Hornsby 1.8m lintel	70.72	69.43
0700FH_40	OnGrade	Hornsby 1.8m lintel	69.53	68.36
0700FH_30	OnGrade	Hornsby 1.8m lintel	69.10	67.47
0700FH_20	OnGrade	Hornsby 1.8m lintel	64.74	63.05
0700FH_10	OnGrade	GP 0.80m x 1.00m	61.35	59.27
0700FH_09	OnGrade	GP 0.50m x 0.90m	60.24	58.29
0700FH_08	OnGrade	Hornsby 1.8m lintel	59.10	57.48
0700FH_07	Sag	Hornsby 1.8m lintel	58.31	56.21
0700FH_06	Sag	GP 0.40m x 0.90m	58.41	55.06

Pit Name (Line No_Pit No)	Pit Type	Pit Size	Surface Elevation (m AHD)	Bottom Level (m AHD)
0710FH_10	OnGrade	Hornsby 1.8m lintel	70.88	69.98
0720FH_10	OnGrade	Hornsby 1.8m lintel	69.92	68.97
13300_010	OnGrade	Hornsby 1.8m lintel	59.14	58.00
2140_030	Sag	Hornsby 1.2m lintel	57.03	55.53
2140_025	Sag	Hornsby 1.2m lintel	57.08	55.48
2140_020	OnGrade	Hornsby 1.2m lintel	57.25	55.31
0330_020	OnGrade	Hornsby 1.8m lintel	27.60	26.55
0330_010	Sag	Hornsby 0.9m lintel	27.52	25.92
0800_010	OnGrade	GP 0.80m x 1.00m	57.45	55.51
0800_008	Sag	Hornsby 2.4m lintel	57.10	54.63
0800_003	Sag	Hornsby 3.0m lintel	57.18	53.79
0800_002	Sag	Hornsby 2.4m lintel	53.57	51.67
0800_001	Sag	Hornsby 2.4m lintel	53.50	51.06
20000_10	Sag	Hornsby 1.8m lintel	57.85	55.92
2010_050	OnGrade	Hornsby 2.4m lintel	54.65	52.00
2010_040	OnGrade	Hornsby 3.0m lintel	52.41	50.11
2010_030	OnGrade	Hornsby 1.8m lintel	51.71	49.58
2010_020	Sag	GP 0.80m x 0.70m	51.24	49.09
2010_010	Sag	Hornsby 2.4m lintel	50.85	48.45
2000_020	Sag	Hornsby 1.8m lintel	50.32	49.42
2000_010	Sag	Hornsby 1.8m lintel	50.24	48.10

Table A-2 Wetherill Park Catchment Stormwater Pipe Data

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
0010_110-100	36.96	36.26	113.1	0.61	1.05
0010_100-095	36.26	35.94	52.2	0.62	1.05
0010_095-090	35.94	35.49	73.2	0.61	1.05
0010_090-080	35.49	34.90	96.6	0.61	1.20
0010_080-070	34.90	33.63	97.6	1.30	1.20
0010_070-060	33.63	33.13	73.7	0.68	1.35
0010_060-050	33.13	32.99	20.8	0.68	1.35
0010_050-048	32.99	32.79	22.1	0.88	1.50
0010_048-047	32.79	32.22	73.9	0.78	1.50
0010_047-040	32.22	31.98	30.3	0.78	1.50
0010_040-030	31.98	31.77	27.4	0.78	1.50
0010_030-020	31.77	31.35	92.2	0.45	1.80
0010_020-010	31.35	31.08	60.3	0.45	1.80
0010_010-008	31.08	30.77	38.2	0.81	1.80
0010_008-000	30.77	30.35	53.0	0.80	1.80
0070_180-170	43.23	41.67	63.3	2.46	1.20
0070_170-160	41.67	41.13	14.0	3.87	1.20
0070_160-150	41.13	40.67	40.8	1.13	0.90
0070_150-140	40.67	39.75	69.6	1.32	0.90
0070_140-130	39.75	38.34	17.5	8.09	0.90
0070_130-120	38.34	37.36	74.0	1.32	1.05
0070_120-100	37.36	36.40	49.6	1.94	1.20
0070_100-090	36.40	36.07	22.1	1.52	1.20
0070_090-080	36.07	34.77	85.4	1.53	1.20
0070_080-070	34.77	33.42	88.2	1.52	1.50
0070_070-060	33.42	33.23	12.6	1.53	1.50
0070_060-050	33.23	31.94	84.5	1.52	1.50
0070_050-040	31.94	31.16	47.1	1.67	1.50
0070_040-035	31.16	28.89	135.6	1.67	1.50
0070_035-034	28.89	28.63	15.6	1.67	1.50
0070_034-030	28.63	24.43	251.2	1.67	1.65
0070_030-020	24.43	23.84	12.8	4.59	1.80
0070_020-015	23.84	23.19	131.8	0.50	1.80
0070_015-014	23.19	22.94	49.8	0.50	1.80
0070_014-010	22.94	22.79	30.1	0.50	1.80
0070_010-000	22.79	22.79	19.3	0.01	1.80
0100_030-020	35.20	34.52	91.4	0.75	0.45
0100_020-010	34.52	33.73	81.5	0.97	0.53
0100_010-000	33.73	32.99	30.6	2.44	0.60
0120_010-000	34.37	32.99	22.4	6.17	0.38
0130_010-000	33.57	33.13	10.8	4.08	0.83

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
0140_010-010	34.12	33.57	13.1	4.21	0.45
0150_010-000	35.03	33.63	4.7	29.87	0.30
0160_040-030	36.09	36.01	14.2	0.57	0.23
0160_030-020	36.01	35.37	25.6	2.52	0.23
0160_020-010	35.37	35.34	1.0	2.50	0.53
0160_010-000	35.34	34.90	12.9	3.44	0.53
0180_020-010	36.84	36.42	19.6	2.14	0.45
0180_010-000	36.42	35.49	13.0	7.19	0.38
0200_010-000	37.07	36.26	9.8	8.19	0.53
0230_190-180	53.12	51.74	114.1	1.21	0.45
0230_180-170	51.74	50.64	16.0	6.90	0.53
0230_170-160	50.64	49.15	114.7	1.30	0.68
0230_160-150	49.15	48.22	41.3	2.25	0.75
0230_150-140	48.22	47.37	37.3	2.25	0.75
0230_140-130	47.37	46.39	43.5	2.25	0.83
0230_130-120	46.39	45.68	31.6	2.25	0.83
0230_120-110	45.68	42.57	138.0	2.25	1.05
0230_110-105	42.57	42.41	20.7	0.77	1.20
0230_105-100	42.41	41.90	49.7	1.03	1.20
0230_100-090	41.90	41.74	15.7	1.03	1.50
0230_090-080	41.74	40.40	130.0	1.03	1.50
0230_080-070	40.40	39.36	100.6	1.03	1.65
0230_070-065	39.36	39.29	7.0	1.03	1.65
0230_065-060	39.29	39.20	5.7	1.60	1.65
0230_060-056	39.20	39.01	11.8	1.61	1.65
0230_056-055	39.01	38.67	25.8	1.29	1.65
0230_055-050	38.67	38.36	24.3	1.29	1.80
0230_050-040	38.36	38.26	7.6	1.29	1.80
0230_040-035	38.26	36.65	125.0	1.29	1.80
0230_035-030	36.65	35.00	127.3	1.29	1.95
0230_030-020	35.00	34.34	9.2	7.21	2.10
0230_020-010	34.34	34.10	12.6	1.94	2.10
0230_010-005	34.10	33.43	77.2	0.87	2.10
0230_005-000	33.43	33.38	3.9	1.29	2.10
0240_040-030	36.82	35.60	68.4	1.78	0.75
0240_030-020	35.60	35.35	14.3	1.72	1.05
0240_020-010	35.35	34.86	73.7	0.67	1.35
0240_010-000	34.86	34.10	6.0	12.78	1.35
0300_030-020	25.25	25.01	34.7	0.68	0.83
0300_020-015	25.01	24.86	32.2	0.47	0.90
0300_015-010	24.86	23.97	54.6	1.63	0.90
0300_010-000	23.97	23.40	34.6	1.63	0.90

Line No_	**Invert Level (m AHD)**		**Length**	**Slope**	**Diameter**
u/s pit_d/s pit	**Upstream**	**Downstream**	**(m)**	**(%)**	**(m)**
0310_010-000	24.73	23.97	10.4	7.37	0.90
0330_020-010	26.55	25.92	20.6	3.06	1.50
0330_010-000	25.92	25.80	46.8	0.26	0.90
0350_010-000	50.99	50.64	24.0	1.47	0.68
0400_070-060	36.72	36.61	96.8	0.11	1.05
0400_060-050	36.61	35.80	100.7	0.80	1.05
0400_050-040	35.80	34.86	74.9	1.26	1.05
0400_040-030	34.86	34.32	17.9	3.01	1.05
0400_030-025	34.32	33.93	23.9	1.62	1.05
0400_025-020	33.93	33.50	17.3	2.54	1.05
0400_020-015	33.50	30.00	194.1	1.80	1.05
0400_015-014	30.00	28.67	73.9	1.80	1.05
0400_014-010	28.67	28.15	28.8	1.80	1.05
0400_010-000	28.15	26.66	82.7	1.80	1.35
0410_040-010	32.09	30.29	100.1	1.80	0.60
0410_010-000	30.29	28.15	5.2	41.18	1.20
0440_040-030	41.83	41.86	3.8	-0.79	0.83
0440_030-020	41.86	41.37	80.9	0.60	0.83
0440_020-010	41.37	40.91	77.3	0.60	0.90
0440_010-000	40.91	40.83	12.8	0.60	0.90
0470_050-040	43.28	42.86	70.8	0.59	0.83
0470_040-035	42.86	42.79	8.9	0.72	0.83
0470_035-030	42.79	42.23	84.5	0.67	1.05
0470_030-026	42.23	41.98	36.8	0.67	1.05
0470_026-025	41.98	41.80	13.2	1.32	1.05
0470_025-024	41.80	41.35	23.6	1.91	1.05
0470_024-020	41.35	40.97	19.9	1.92	1.05
0470_020-010	40.97	39.83	59.6	1.91	1.50
0470_010-000	39.83	39.50	17.4	1.92	1.50
0480_010-000	40.79	40.97	17.7	-1.02	1.20
0490_010-000	43.41	42.23	25.7	4.60	0.30
0510_050-040	47.59	46.72	34.0	2.55	0.83
0510_040-030	46.72	45.37	53.4	2.53	1.05
0510_030-020	45.37	45.31	8.2	0.72	1.05
0510_020-015	45.31	45.28	4.1	0.71	1.05
0510_015-010	45.28	44.94	47.0	0.72	1.05
0510_010-009	44.94	44.18	105.1	0.72	1.50
0510_009-008	44.18	44.15	4.0	0.72	1.50
0510_008-007	44.15	44.04	15.5	0.72	1.50
0510_007-000	44.04	43.83	7.4	2.82	1.50
0540_010-000	46.95	46.72	13.1	1.75	0.68
0560_010-008	45.32	45.35	7.5	-0.45	0.90

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
0560_008-005	45.35	44.96	49.0	0.80	0.90
0560_005-000	44.96	44.91	3.7	1.28	0.90
0570_610-060	53.48	53.92	12.9	-3.41	0.60
0570_060-050	53.92	52.74	68.5	1.72	1.05
0570_050-045	52.74	52.75	10.9	-0.14	1.65
0570_045-040	52.75	51.04	47.2	3.63	1.65
0570_040-030	51.04	50.49	84.3	0.64	1.65
0570_030-025	50.49	50.39	16.4	0.64	1.65
0570_025-020	50.39	49.81	90.6	0.64	1.65
0570_020-010	49.81	49.51	46.5	0.64	1.20
0570_010-008	49.51	48.76	115.2	0.64	1.20
0570_008-007	48.76	48.16	93.3	0.64	1.20
0570_007-000	48.16	48.13	4.6	0.82	1.80
0570_001-000	48.16	48.06	4.1	2.42	1.80
0580_020-010	53.42	53.02	5.3	7.60	0.60
0580_010-000	53.02	51.04	13.0	15.20	0.60
0590_060-050	63.11	59.23	85.0	4.56	0.75
0590_050-040	59.23	55.75	76.5	4.56	0.75
0590_040-030	55.75	55.59	3.5	4.57	0.75
0590_030-020	55.59	52.29	72.3	4.56	0.75
0590_020-010	52.29	52.12	11.2	1.53	1.65
0590_010-000	52.12	52.75	15.9	-3.95	1.65
0600_008-005	52.81	52.80	4.0	0.07	1.05
0600_005-002	52.80	52.75	65.4	0.08	1.05
0600_002-000	52.75	52.74	18.5	0.08	1.05
0630_310-300	38.91	37.89	35.2	2.92	0.90
0630_300-290	37.89	37.68	41.9	0.48	1.05
0630_290-280	37.68	37.01	47.3	1.42	1.05
0630_280-270	37.01	36.78	23.0	1.02	1.05
0630_270-260	36.78	36.47	28.7	1.06	1.05
0630_260-250	36.47	36.00	56.2	0.84	1.05
0630_250-240	36.00	34.85	47.1	2.45	1.05
0630_240-230	34.85	34.21	47.6	1.36	1.05
0630_230-220	34.21	34.02	13.1	1.46	1.05
0630_220-210	34.02	33.68	20.5	1.66	1.05
0630_210-200	33.68	33.48	19.5	1.02	1.20
0630_200-190	33.48	33.06	40.4	1.03	1.20
0630_190-180	33.06	32.97	11.2	0.82	1.20
0630_180-170	32.97	32.79	18.4	0.99	1.20
0630_170-160	32.79	32.50	29.8	0.97	1.20
0630_160-150	32.50	32.06	67.7	0.64	1.20
0630_150-140	32.06	31.61	44.5	1.02	1.20

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
0630_140-130	31.61	31.32	12.7	2.28	1.20
0630_130-120	31.32	30.79	23.3	2.28	1.20
0630_120-100	30.79	29.62	50.9	2.28	1.50
0630_100-090	29.62	29.00	85.2	0.73	1.80
0630_090-080	29.00	28.95	6.7	0.74	1.80
0630_080-070	28.95	28.33	86.2	0.73	2.40
0630_070-065	28.33	28.15	33.6	0.52	2.40
0630_065-064	28.15	27.97	34.2	0.52	2.40
0630_064-060	27.97	27.85	24.0	0.52	2.40
0630_060-050	27.85	27.51	64.7	0.52	2.40
0630_050-040	27.51	27.39	24.8	0.52	2.40
0630_040-030	27.39	27.08	67.6	0.45	2.40
0630_030-020	27.08	27.07	3.9	0.18	2.40
0630_020-000	27.07	27.05	4.1	0.54	2.40
0630_010-000	27.05	26.70	45.6	0.78	2.40
0640_040-030	62.40	61.95	7.5	6.00	0.60
0640_030-020	61.95	61.19	12.6	6.00	0.60
0640_020-010	61.19	60.98	13.6	1.56	0.60
0640_010-000	60.98	60.18	51.4	1.56	0.90
0650_010-000	61.55	60.98	19.3	2.95	0.60
0700FH_50-40	69.43	68.36	26.9	3.99	0.45
0700FH_40-30	68.36	67.47	22.8	3.88	0.45
0700FH_30-20	67.47	63.05	52.6	8.42	0.45
0700FH_20-10	63.05	59.27	48.9	7.73	0.45
0700FH_10-09	59.27	58.29	40.3	2.42	0.60
0700FH_09-08	58.29	57.49	46.4	1.75	0.68
0700FH_08-07	57.49	56.21	41.1	3.10	0.68
0700FH_07-06	56.21	55.06	6.2	18.57	0.68
0700FH_06-00	55.06	54.32	21.6	3.46	0.68
0700_220-210	45.16	44.90	9.9	2.59	0.90
0700_210-200	44.90	44.10	79.9	1.00	1.05
0700_200-190	44.10	43.89	13.6	1.57	1.05
0700_190-180	43.89	42.96	21.8	4.29	1.05
0700_180-170	42.96	41.30	87.9	1.89	1.05
0700_170-160	41.30	39.39	93.2	2.05	1.05
0700_160-155	39.39	39.32	13.3	0.52	1.20
0700_155-154	39.32	39.06	50.5	0.52	1.20
0700_154-150	39.06	38.96	19.7	0.52	1.20
0700_150-140	38.96	38.22	62.0	1.20	1.20
0700_140-130	38.22	37.90	29.5	1.06	1.20
0700_130-120	37.90	36.42	139.7	1.06	1.35
0700_120-110	36.42	35.34	125.7	0.86	0.60

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
0700_110-105	35.34	34.52	80.4	1.01	0.60
0700_105-090	34.52	32.93	39.7	4.00	0.90
0700_090-080	32.93	32.95	11.0	-0.15	1.05
0700_080-070	32.95	32.85	46.8	0.21	1.50
0700_070-060	32.85	32.46	31.4	1.25	1.50
0700_060-050	32.46	31.84	82.5	0.75	1.65
0700_050-040	31.84	31.09	88.9	0.85	1.65
0700_040-030	31.09	30.23	94.1	0.91	1.65
0700_030-020	30.23	30.01	21.8	1.03	1.65
0700_020-010	30.01	29.86	23.6	0.61	1.65
0700_010-000	29.86	28.95	10.2	8.91	1.65
0710FH_10-00	69.98	63.05	69.3	10.01	0.38
0720FH_10-00	68.97	68.36	17.9	3.42	0.38
0730_760-090	57.19	59.91	22.3	-12.17	0.45
0730_090-080	59.91	59.17	84.3	0.87	0.75
0730_080-070	59.17	58.25	21.3	4.35	0.75
0730_070-060	58.25	57.90	37.2	0.92	0.90
0730_060-050	57.90	57.42	17.8	2.71	0.90
0730_050-040	57.42	57.17	92.5	0.27	0.90
0730_040-030	57.17	56.75	100.0	0.41	0.90
0730_030-020	56.75	55.64	23.3	4.79	1.05
0730_020-010	55.64	54.82	40.4	2.02	1.05
0730_010-000	54.82	54.41	92.1	0.44	1.20
0750_10-00FH	52.15	57.90	55.5	-10.36	0.38
0750_080-070	36.91	36.60	19.6	1.56	0.90
0750_070-065	36.60	36.26	21.9	1.56	0.90
0750_065-060	36.26	36.15	8.6	1.31	0.90
0750_060-050	36.15	35.95	14.7	1.37	0.90
0750_050-040	35.95	35.49	33.1	1.37	1.05
0750_040-030	35.49	34.44	76.4	1.37	1.05
0750_030-020	34.44	34.24	14.5	1.37	1.05
0750_020-010	34.24	33.31	31.6	2.96	1.05
0750_010-000	33.31	32.46	30.8	2.75	1.05
0760_050-040	44.46	41.56	33.4	8.68	0.53
0760_040-035	41.56	40.41	40.8	2.81	0.53
0760_035-030	40.41	39.98	15.4	2.81	0.53
0760_030-025	39.98	39.01	34.3	2.82	0.60
0760_025-020	39.01	38.17	29.9	2.81	0.60
0760_020-016	38.17	37.34	34.6	2.42	0.60
0760_016-015	37.34	37.03	11.8	2.57	0.60
0760_015-010	37.03	36.85	7.2	2.58	0.60
0760_010-000	36.85	34.44	23.2	10.38	0.75

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
0770_020-010	41.20	39.50	32.8	5.17	0.23
0770_010-000	39.50	38.17	11.1	11.95	0.30
0800_010-008	55.52	54.63	27.0	3.26	0.90
0800_008-003	54.63	53.79	22.1	3.82	0.90
0800_003-002	53.79	51.67	77.0	2.76	1.20
0800_002-001	51.67	51.53	16.6	0.81	1.20
0850A_15-01	36.93	36.86	19.9	0.33	1.80
0850_150-140	48.80	48.75	15.9	0.27	0.83
0850_140-130	48.75	48.73	9.8	0.28	0.83
0850_130-120	48.73	44.84	222.3	1.75	1.05
0850_120-110	44.84	44.67	9.9	1.75	1.05
0850_110-100	44.67	43.16	86.1	1.75	1.35
0850_100-095	43.16	42.66	28.8	1.75	1.35
0850_095-092	42.66	42.08	104.4	0.55	1.35
0850_092-090	42.08	41.84	22.0	1.12	1.65
0850_090-080	41.84	41.35	43.3	1.13	1.65
0850_080-070	41.35	40.33	54.1	1.89	1.80
0850_070-060	40.33	39.39	70.8	1.32	1.80
0850_060-050	39.39	38.59	41.7	1.93	1.80
0850_050-040	38.59	38.40	21.9	0.87	1.80
0850_040-035	38.40	38.20	15.6	1.22	1.80
0850_035-030	38.20	37.55	53.8	1.22	1.80
0850_030-020	37.55	37.26	23.9	1.22	1.80
0850_020-015	37.26	36.93	198.0	0.17	1.80
0850_015-000	36.93	36.89	22.0	0.16	1.80
0860_060-050	39.19	39.43	12.7	-1.87	0.83
0860_050-040	39.43	38.95	8.2	5.81	0.83
0860_040-030	38.95	38.92	13.0	0.18	0.90
0860_030-020	38.92	38.32	95.2	0.64	1.20
0860_020-010	38.32	37.72	93.1	0.64	1.35
0860_010-000	37.72	37.66	9.2	0.64	1.50
0870_020-010	39.50	39.31	12.7	1.42	0.83
0870_010-000	39.31	38.92	27.4	1.43	0.90
0890_020-010	39.90	39.88	13.0	0.12	0.38
0890_010-000	39.88	38.95	73.1	1.28	0.83
0900_010-000	40.00	39.88	4.9	2.45	0.75
0910_010-000	37.50	37.26	32.3	0.76	0.38
0920_030-020	39.46	39.44	5.6	0.34	0.90
0920_020-000	39.44	37.55	5.3	36.06	0.90
0930_010-009	44.19	43.72	22.8	2.06	1.80
0930_009-008	43.72	40.80	80.2	3.64	1.80
0930_008-007	40.80	38.57	99.6	2.25	1.80

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
0930_007-000	38.57	37.26	3.0	43.61	1.80
0950_010-0FH	42.54	40.80	2.0	85.11	0.60
0950_010-0WP	35.00	32.95	32.3	6.36	0.38
0960_010-000	45.84	44.19	2.0	83.68	0.60
0980_010-000	39.50	39.39	5.9	1.89	0.53
0990_010-000	41.67	39.39	20.4	11.20	0.53
1000_010-000	40.40	40.33	5.9	1.26	0.45
1010_010-000	42.46	40.33	18.2	11.74	0.38
1020_030-025	40.77	39.83	29.4	3.19	0.53
1020_025-024	39.83	39.39	13.9	3.18	0.60
1020_024-020	39.39	38.09	40.9	3.18	0.60
1020_020-010	38.09	38.14	9.9	-0.51	0.75
1020_010-000	38.14	37.90	6.0	3.93	0.75
1020_10-00FH	42.69	41.35	17.8	7.52	0.68
1030_010-000	41.90	41.84	22.4	0.29	1.65
1040_020-010	44.90	44.80	8.7	1.13	0.38
1040_010-000	44.80	42.08	19.4	14.02	0.38
1050_100-090	52.44	51.87	23.9	2.36	1.05
1050_090-086	51.87	51.37	14.6	3.49	1.05
1050_086-085	51.37	50.85	19.5	2.65	1.05
1050_085-084	50.85	50.13	20.3	3.55	1.05
1050_084-080	50.13	49.64	16.2	3.03	1.05
1050_080-078	49.64	49.48	18.6	0.86	1.05
1050_078-077	49.48	49.20	20.7	1.32	1.05
1050_077-076	49.20	49.09	9.2	1.18	1.05
1050_076-075	49.09	48.96	10.4	1.27	1.05
1050_075-070	48.96	48.95	6.4	0.23	1.05
1050_070-060	48.95	48.21	70.2	1.05	1.05
1050_060-050	48.21	47.32	75.2	1.18	1.05
1050_050-040	47.32	45.25	65.3	3.17	1.05
1050_040-030	45.25	44.71	21.3	2.52	1.05
1050_030-020	44.71	44.36	14.5	2.42	1.05
1050_020-015	44.36	43.34	26.9	3.82	1.05
1050_015-000	43.34	42.08	32.7	3.83	1.05
1170_090-080	46.94	46.23	37.8	1.88	0.45
1170_080-070	46.23	45.24	24.0	4.15	0.45
1170_070-065	45.24	44.49	18.0	4.15	0.45
1170_065-060	44.49	43.25	29.7	4.16	0.45
1170_060-050	43.25	42.87	35.3	1.10	0.60
1170_050-040	42.87	41.75	94.1	1.18	0.60
1170_040-030	41.75	40.86	41.3	2.16	0.60
1170_030-025	40.86	40.83	1.8	1.73	0.60

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
1170_025-020	40.83	40.53	16.9	1.75	0.60
1170_020-015	40.53	40.19	14.6	2.36	0.75
1170_015-010	40.19	39.88	12.9	2.37	0.75
1170_010-000	39.88	39.39	13.3	3.69	0.75
1180_010-000	41.55	40.83	9.6	7.53	0.38
1190_000-050	58.25	44.83	146.3	9.17	0.38
1190_050-040	44.83	43.61	69.7	1.75	0.60
1190_040-030	43.61	42.35	62.5	2.02	0.83
1190_030-020	42.35	40.99	63.0	2.16	1.05
1190_020-010	40.99	40.61	63.0	0.60	1.20
1190_010-005	40.61	40.22	45.0	0.88	1.20
1190_010-000	42.08	41.75	10.6	3.09	0.38
1190_005-000	40.22	39.97	3.4	7.13	1.20
1200_010-000	41.37	40.61	22.1	3.42	0.38
1200_10-00WP	43.52	42.87	11.8	5.55	0.38
1210_010-000	41.64	40.99	31.5	2.06	0.38
1220_010-000	43.05	42.35	31.2	2.24	0.38
1230_010-000	43.97	43.61	30.8	1.18	0.53
1240_010-00	45.28	44.83	30.8	1.47	0.38
1250_150-140	54.54	54.13	13.0	3.17	0.45
1250_140-135	54.13	53.99	5.1	2.76	1.05
1250_135-130	53.99	53.70	10.6	2.76	1.05
1250_130-120	53.70	52.50	43.5	2.76	1.20
1250_120-010	52.50	52.29	8.5	2.39	1.20
1250_010-090	52.29	47.49	201.8	2.38	1.20
1250_090-080	47.49	47.37	5.3	2.38	1.05
1250_080-030	47.37	47.10	13.6	1.98	1.20
1330_030-020	47.10	46.31	40.3	1.94	1.35
1330_020-010	46.31	45.38	48.1	1.94	1.35
1330_010-005	45.38	44.36	52.4	1.95	1.35
1330_005-004	44.36	43.73	32.4	1.94	1.35
1330_004-010	43.73	43.49	12.5	1.94	1.35
1270_010-000	43.49	40.01	178.2	1.95	1.35
1250_060-050	45.54	44.36	42.1	2.81	0.83
1250_050-040	44.36	44.14	15.0	1.48	0.83
1250_040-030	44.14	41.92	103.4	2.14	0.90
1250_030-025	41.92	41.71	7.7	2.79	0.90
1250_025-020	41.71	40.12	57.1	2.78	1.05
1250_020-015	40.12	40.01	12.0	0.92	1.05
1250_015-010	40.01	39.96	5.2	0.92	1.20
1250_010-005	39.96	39.76	28.1	0.72	1.20
1250_005-000	39.76	39.74	2.7	0.71	1.20

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
13300_10-00	58.00	57.49	13.7	3.80	0.38
13600_50-40	64.77	59.62	69.6	7.40	0.68
13600_40-30	59.62	58.68	25.3	3.69	0.90
13600_30-20	58.68	57.38	21.2	6.14	0.90
13600_20-10	57.38	56.27	18.1	6.14	0.90
13600_10-00	56.27	55.31	15.6	6.14	0.90
1390_120-110	36.04	34.68	69.2	1.96	0.60
1390_110-100	34.68	34.40	23.3	1.21	0.60
1390_100-090	34.40	34.26	13.4	1.05	0.60
1390_090-080	34.26	33.47	59.2	1.34	0.60
1390_080-070	33.47	33.20	11.3	2.33	0.60
1390_070-060	33.20	33.19	9.9	0.17	0.75
1390_060-050	33.19	33.00	11.6	1.58	0.75
1390_050-040	33.00	32.56	50.3	0.89	0.75
1390_040-030	32.56	32.24	21.1	1.51	0.90
1390_030-020	32.24	31.74	32.7	1.52	1.05
1390_020-010	31.74	31.45	30.3	0.97	1.05
1390_010-005	31.45	29.66	15.4	11.59	1.05
1390_005-000	29.66	29.62	9.3	0.38	1.05
1450_050-040	53.27	53.17	25.9	0.38	0.90
1450_040-031	53.17	53.02	40.8	0.38	0.90
1450_031-030	53.02	53.01	2.3	0.40	0.90
1450_030-020	53.01	52.85	44.1	0.38	1.05
1450_020-010	52.85	52.77	19.2	0.38	1.05
1450_010-100	52.77	52.44	14.3	2.36	1.05
14600_50-40	46.89	46.24	33.9	1.91	0.83
14600_40-30	46.24	45.46	37.2	2.12	0.90
14600_30-20	45.46	45.37	4.4	1.92	0.90
14600_20-10	45.37	44.18	58.1	2.05	1.05
1500_170-160	60.58	60.32	52.3	0.50	1.05
1500_160-150	60.32	60.12	18.1	1.09	1.05
1500_150-140	60.12	59.82	12.8	2.35	1.05
1500_140-135	59.82	59.85	66.5	-0.04	1.05
1500_135-130	59.85	57.40	71.7	3.41	1.20
1500_130-125	57.40	57.43	1.9	-1.27	1.20
1500_125-124	57.43	57.25	23.9	0.74	1.20
1500_124-120	57.25	57.24	1.0	0.72	1.20
1500_120-110	57.24	56.88	9.4	3.83	1.20
1500_110-100	56.88	55.64	69.9	1.79	1.20
1500_100-095	55.64	54.20	70.2	2.05	1.20
1500_095-090	54.20	53.35	36.3	2.34	1.35
1500_090-080	53.35	51.26	173.1	1.21	1.35

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
1500_080-075	51.26	51.09	14.1	1.20	1.35
1500_075-070	51.09	50.84	18.8	1.36	1.50
1500_070-060	50.84	50.46	27.7	1.36	1.65
1500_060-050	50.46	50.42	3.0	1.35	1.65
1500_050-040	50.42	48.68	199.7	0.87	1.65
1500_040-030	48.68	47.72	41.0	2.34	1.65
1500_030-020	47.72	45.76	226.1	0.87	1.65
1500_020-010	45.76	45.61	17.5	0.87	1.65
1500_010-000	45.61	44.29	151.9	0.87	1.65
15100_10-00	32.79	31.98	9.6	8.34	0.83
1510_130-120	57.14	56.30	13.2	6.35	0.83
1510_120-110	56.30	55.01	41.9	3.07	0.83
1510_110-100	55.01	52.77	28.0	7.99	0.83
1510_100-090	52.77	51.16	25.4	6.34	0.83
1510_090-080	51.16	48.93	25.8	8.68	0.83
1510_080-070	48.93	48.84	9.4	0.89	1.05
1510_070-060	48.84	48.62	24.3	0.90	1.05
1510_060-050	48.62	47.65	108.8	0.90	1.20
1510_050-045	47.65	46.80	78.8	1.08	1.35
1510_045-040	46.80	46.54	25.5	1.01	1.35
1510_040-035	46.54	46.51	7.4	0.47	1.35
1510_035-030	46.51	46.17	28.7	1.16	1.35
1510_030-020	46.17	45.90	24.0	1.16	1.35
1510_020-010	45.90	45.16	20.6	3.55	1.35
1510_010-000	45.16	44.32	153.8	0.55	1.65
15200_50-40	34.43	33.34	88.5	1.22	0.83
15200_40-30	33.34	32.98	83.9	0.43	0.83
15200_30-20	32.98	32.24	21.8	3.43	0.83
15200_20-10	32.24	32.08	20.8	0.77	1.05
15200_10-08	32.08	32.06	1.5	0.79	1.05
15200_08-00	32.06	31.35	27.9	2.55	1.05
1580_030-020	50.11	49.62	17.2	2.86	0.68
1580_020-010	49.62	48.33	45.2	2.86	1.80
1580_010-000	48.33	47.72	21.3	2.85	1.80
1581_030-020	47.72	46.09	216.9	0.75	0.90
1581_020-010	46.09	45.45	9.5	6.83	0.90
1581_010-000	45.45	45.16	16.8	1.68	1.65
1620_020-010	61.03	60.45	13.0	4.50	0.53
1620_010-009	60.45	60.37	1.8	4.66	0.30
1620_009-008	60.37	58.05	49.5	4.67	0.53
1620_008-007	58.05	57.45	12.9	4.67	0.53
1620_007-006	57.45	54.98	51.3	4.80	0.53

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
1620_006-005	54.98	54.69	17.8	1.63	0.60
1620_005-004	54.69	52.86	41.4	4.43	0.60
1620_004-003	52.86	52.02	18.8	4.43	0.60
1620_003-000	52.02	50.84	26.8	4.43	0.60
16300_150-14	60.18	59.33	30.9	2.74	0.90
16300_140-13	59.33	58.60	32.5	2.24	1.05
16300_130-12	58.60	58.32	10.8	2.64	1.05
16300_120-11	58.32	57.07	47.2	2.64	1.05
16300_110-10	57.07	57.00	5.8	1.20	1.05
16300_100-09	57.00	55.83	68.6	1.71	1.05
16300_090-00	55.83	52.29	94.3	3.74	1.05
16400_40-30	52.50	52.02	18.3	2.64	0.38
16400_30-20	52.02	49.64	43.6	5.45	0.83
16400_20-10	49.64	48.74	34.0	2.65	0.83
16400_10-00	48.74	45.61	16.3	19.24	0.83
16500_10-08	53.13	49.93	37.4	8.57	0.90
16500_08-07	49.93	49.55	71.8	0.53	1.05
16500_07-00	49.55	49.51	8.3	0.47	1.05
16600_010-00	55.21	54.13	55.6	1.93	0.83
16700_10-00	52.65	52.02	50.0	1.26	0.83
16800_40-20	38.40	38.18	15.4	1.39	1.80
16800_20-10	38.18	37.62	54.3	1.03	1.80
16800_10-00	37.62	37.50	11.7	1.03	1.80
16800_00-00	37.50	37.26	23.7	1.03	1.80
1680_010-000	58.38	57.25	9.7	11.62	0.53
16900_40-20	38.40	38.24	15.0	1.06	1.80
16900_20-10	38.24	37.64	56.4	1.06	1.80
16900_10-00	37.64	37.50	12.3	1.12	1.80
17000_10-00	38.13	37.24	12.8	6.96	0.83
17100_30-20	39.92	39.53	20.9	1.87	0.90
17100_20-10	39.53	39.07	43.0	1.07	1.05
17100_10-00	39.07	38.67	2.6	15.36	1.05
17500_01-00	65.76	64.97	8.9	8.85	1.05
1770_070-060	34.21	33.43	13.7	5.74	1.05
1770_060-050	33.43	31.59	32.0	5.74	1.05
1770_050-040	31.59	31.18	21.9	1.91	1.05
1770_040-030	31.18	30.42	20.9	3.63	1.05
1770_030-020	30.42	29.07	21.2	6.37	1.05
1770_020-010	29.07	28.90	22.6	0.73	1.20
1770_010-000	28.90	26.66	21.1	10.60	1.20
1800_020-010	33.80	33.43	14.1	2.59	0.75
1800_010-000	33.43	32.70	28.1	2.59	0.75

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
1820_100-090	36.45	36.23	12.7	1.72	1.20
1820_090-080	36.23	34.76	88.0	1.67	1.20
1820_080-070	34.76	33.67	64.8	1.68	0.90
1820_070-060	33.67	33.28	34.2	1.14	1.20
1820_060-050	33.28	32.71	36.1	1.57	1.20
1820_050-040	32.71	32.23	29.6	1.64	1.20
1820_040-030	32.23	31.60	30.6	2.07	1.20
1820_030-020	31.60	30.34	62.6	2.00	1.20
1820_020-010	30.34	30.29	24.9	0.21	1.20
1820_010-006	30.29	30.11	88.9	0.21	1.35
1820_006-005	30.11	30.08	14.2	0.21	1.35
1820_005-000	30.08	29.89	8.7	2.13	1.35
1890_040-030	46.72	45.37	94.4	1.43	0.75
1890_030-025	45.37	45.31	17.7	0.33	0.90
1890_025-020	45.31	45.28	69.1	0.04	0.90
1890_020-010	45.28	44.94	32.7	1.04	1.05
1890_010-008	44.94	44.18	32.9	2.31	1.05
1890_008-007	44.18	44.15	29.7	0.10	1.05
1890_007-006	44.15	44.04	19.1	0.59	1.05
1890_006-000	44.04	43.83	10.2	2.06	1.05
1960_020-010	49.26	48.24	29.0	3.50	0.90
1960_010-001	48.24	47.81	6.1	7.06	1.05
1970_010-000	50.36	48.24	22.1	9.58	0.38
20000_10-00	55.92	54.63	50.7	2.54	0.83
2000_020-010	49.42	48.10	13.0	10.18	0.38
2000_010-000	48.10	48.07	5.4	0.51	0.45
2010_050-040	52.00	50.11	50.3	3.75	0.83
2010_040-030	50.11	49.59	29.3	1.79	1.20
2010_030-020	49.59	49.10	42.7	1.15	1.20
2010_020-010	49.10	48.45	37.6	1.71	1.20
2010_010-000	48.45	48.35	9.5	1.05	0.90
2060_040-030	31.83	31.20	67.0	0.93	0.90
2060_030-010	31.20	30.39	84.4	0.97	1.05
2060_010-000	30.39	30.32	40.4	0.16	1.05
2080_020-005	46.80	33.74	376.4	3.47	0.90
2080_005-004	33.74	33.50	13.8	1.74	0.90
2080_004-000	33.50	31.70	10.5	17.17	1.20
2090_030-020	34.50	34.23	25.1	1.09	0.53
2090_020-015	34.23	33.36	79.1	1.10	0.60
2090_015-010	33.36	33.13	21.0	1.09	0.60
2090_010-000	33.13	33.50	11.1	-3.32	0.60
2100_120-115	42.46	39.35	182.0	1.71	1.35

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
2100_115-110	39.35	38.54	47.0	1.71	1.35
2100_110-100	38.54	38.50	2.6	1.68	1.35
2100_100-090	38.50	38.17	19.3	1.71	1.35
2100_090-085	38.17	37.78	22.6	1.71	1.35
2100_085-080	37.78	37.24	53.2	1.02	1.50
2100_080-070	37.24	36.58	67.4	0.98	1.80
2100_070-060	36.58	36.49	7.6	1.25	1.80
2100_060-050	36.49	35.82	53.5	1.24	1.80
2100_050-040	35.82	35.55	21.4	1.25	1.80
2100_040-035	35.55	35.41	11.5	1.24	1.80
2100_035-030	35.41	34.78	42.1	1.49	1.80
2100_030-020	34.78	33.67	69.0	1.62	1.80
2100_020-010	33.67	33.38	17.9	1.62	1.80
2100_010-000	33.38	32.29	67.0	1.62	1.80
2140_050-040	59.83	57.25	95.8	2.68	0.75
2140_040-030	57.25	55.53	76.9	2.24	0.90
2140_030-025	55.53	55.48	21.4	0.26	0.90
2140_025-020	55.48	55.31	18.3	0.91	1.05
2140_020-000	55.31	54.82	54.2	0.91	1.05
2150_030-020	59.95	59.91	14.1	0.30	0.83
2150_020-015	59.91	59.17	51.8	1.42	0.83
2150_015-010	59.17	58.25	14.5	6.38	0.90
2150_010-009	58.25	57.90	5.4	6.37	1.20
2150_009-008	57.90	57.42	13.7	3.52	1.20
2150_008-008	57.42	57.17	14.2	1.77	1.20
2150_008-007	57.17	56.75	23.3	1.78	1.20
2150_007-005	56.75	55.64	62.7	1.78	1.35
2150_005-005	55.64	54.82	46.0	1.78	1.35
2150_005-000	54.82	54.41	23.0	1.78	0.90
2160_020-010	63.02	61.21	34.7	5.22	0.53
2160_010-000	61.21	58.25	36.9	8.01	0.53
2310_040-038	65.94	65.56	15.4	2.42	0.38
2310_038-037	65.56	64.33	43.0	2.86	0.60
2310_037-030	64.33	63.16	41.1	2.86	0.75
2310_030-020	63.16	62.82	18.5	1.85	0.75
2310_020-010	62.82	62.15	24.7	2.71	0.90
2310_010-000	62.15	61.58	20.8	2.71	0.90
2360_070-060	67.58	66.56	53.3	1.92	0.75
2360_060-050	66.56	66.06	124.6	0.40	1.05
2360_050-045	66.06	65.79	15.0	1.83	1.05
2360_045-040	65.79	65.41	20.8	1.83	1.05
2360_040-035	65.41	64.41	54.4	1.83	1.20

Line No_	Invert Level (m AHD)		Length	Slope	Diameter
u/s pit_d/s pit	Upstream	Downstream	(m)	(%)	(m)
2360_035-030	64.41	63.56	46.5	1.83	1.20
2360_030-020	63.56	61.81	95.0	1.83	1.20
2380_030-020	66.56	66.17	33.8	1.14	0.38
2380_020-010	66.17	65.76	36.0	1.14	0.38
2380_010-000	65.76	64.97	69.3	1.14	0.38
2390_010-000	64.75	63.56	45.8	2.61	0.45
2400_010-000	67.08	65.41	44.5	3.77	0.90

Appendix B IFD and Design Rainfall Intensity Data

Table B-1 Average Rainfall Intensities for Storm Events up to 500 year ARI (mm/hr)

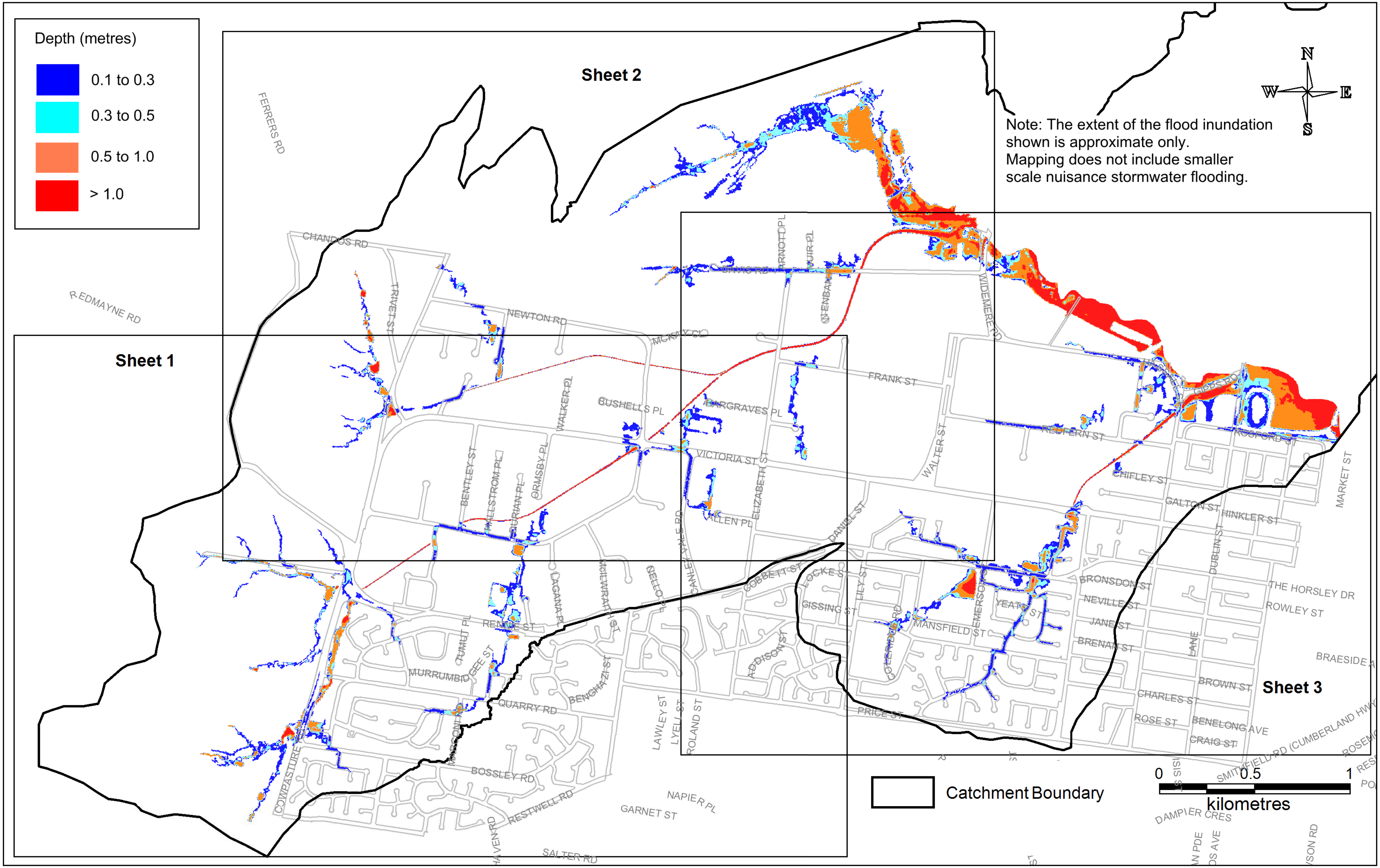
Duration (min)	Event ARI		
	20 year	100 year	500 year
15	107.8	138.7	170.8
25	83.7	107.5	132.5
30	76.1	97.7	120.3
45	60.9	78.2	96.2
60	51.6	66.4	81.5
90	40.6	52.4	64.6
120	34.1	44.1	54.5
180	26.6	34.5	42.9
270	20.7	26.9	33.7
360	17.4	22.6	28.4
540	13.6	17.8	22.3

Table B-2 Average Rainfall Intensities for Extreme Storm Events (mm/hr)

Duration (min)	Event ARI	
	10,000 year	PMP
15	273.1	542.8
25	212.9	428.2
30	195.4	399.6
45	160.2	340.3
60	138.0	299.7
90	107.6	228.6
120	90.5	192.1
180	69.6	142.9
360	45.2	90.5

Appendix C Flood Depth Mapping

- Flood depths for 20 and 100 year ARI and PMF events presented. This mapping is the revised mapping produced by Cardno.

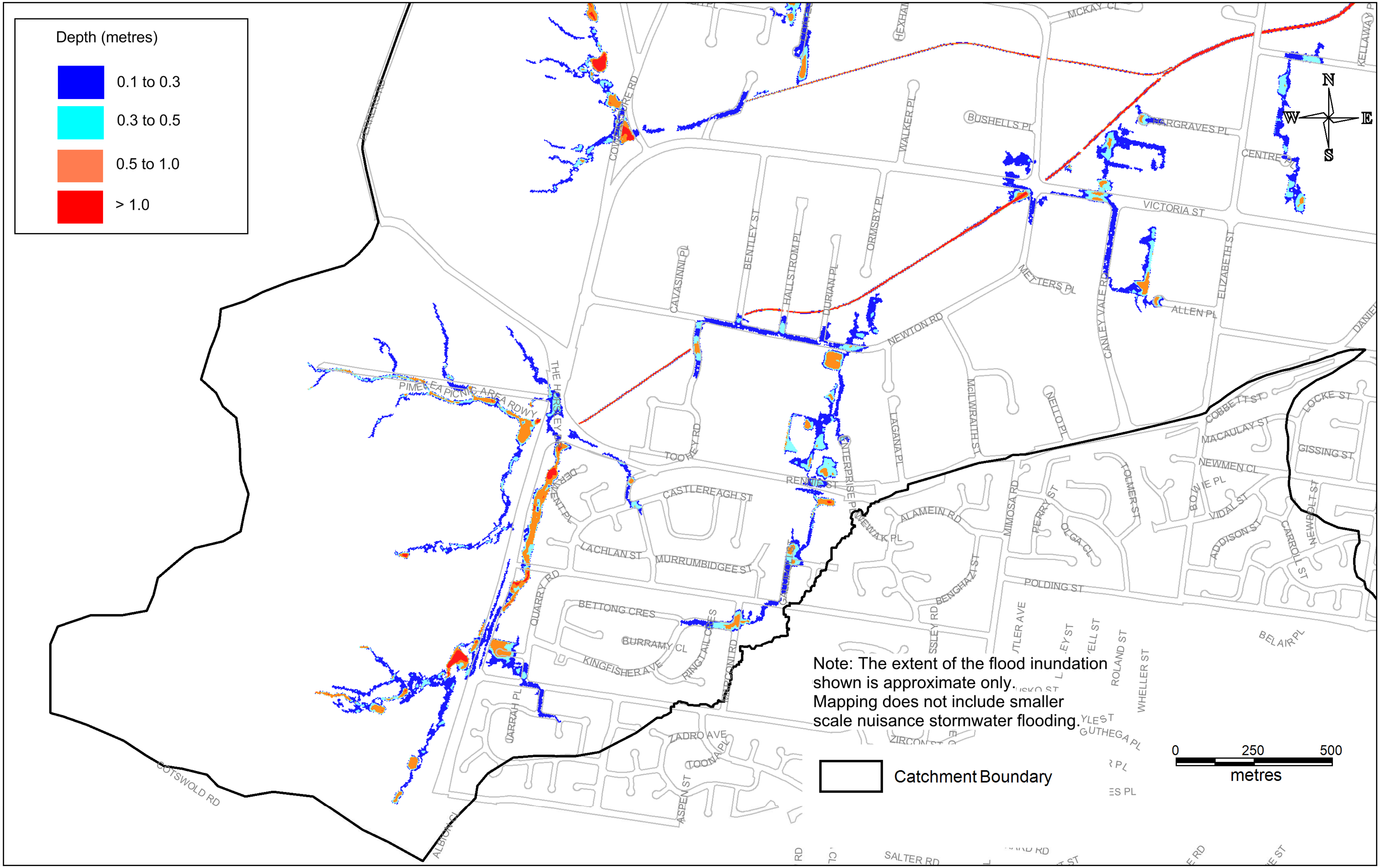


20Year ARI Flood Depths

Cardno

59914102
October 2014

Wetherill Park Catchment Management Plan



20Year ARI Flood Depths
Sheet 1

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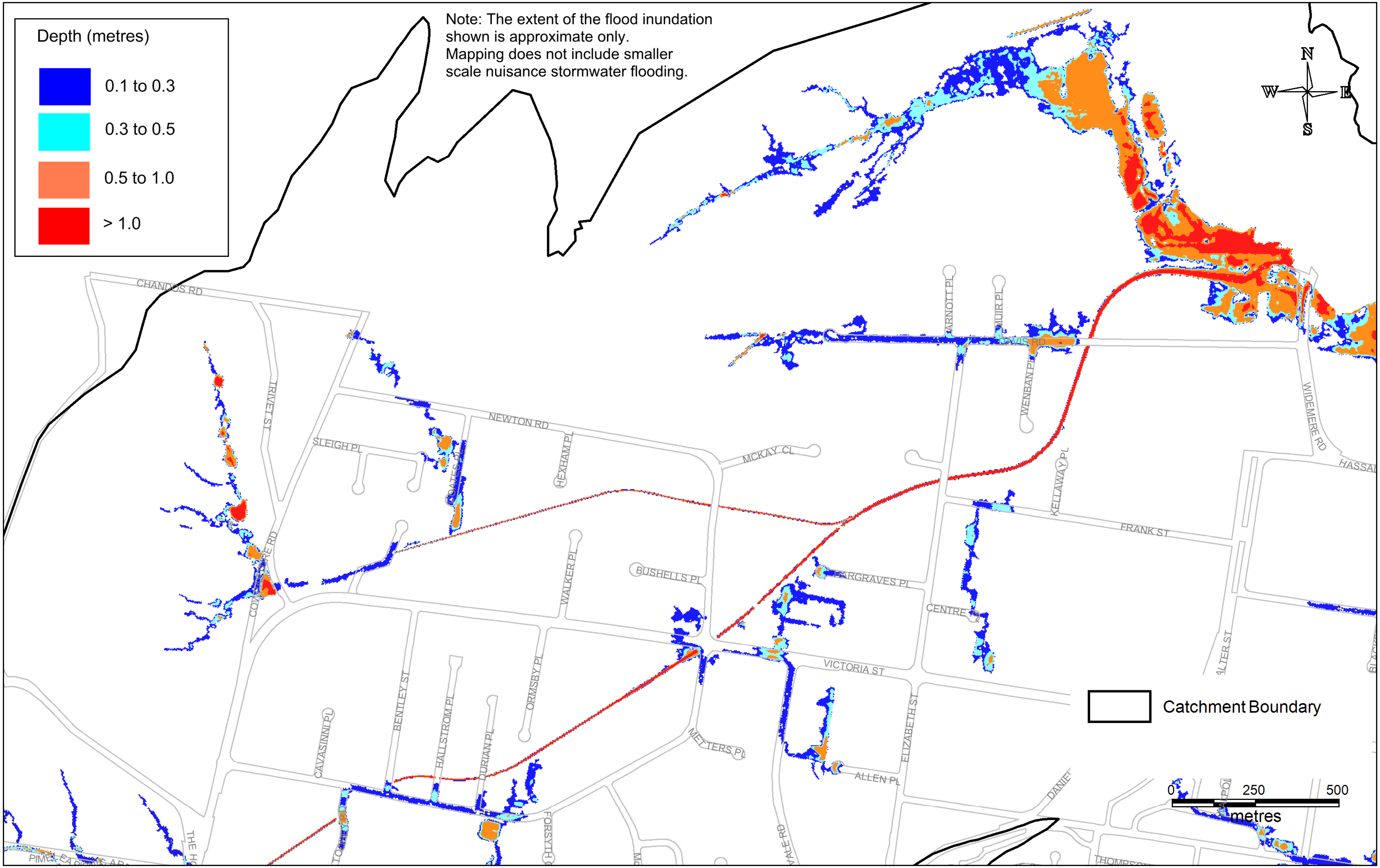
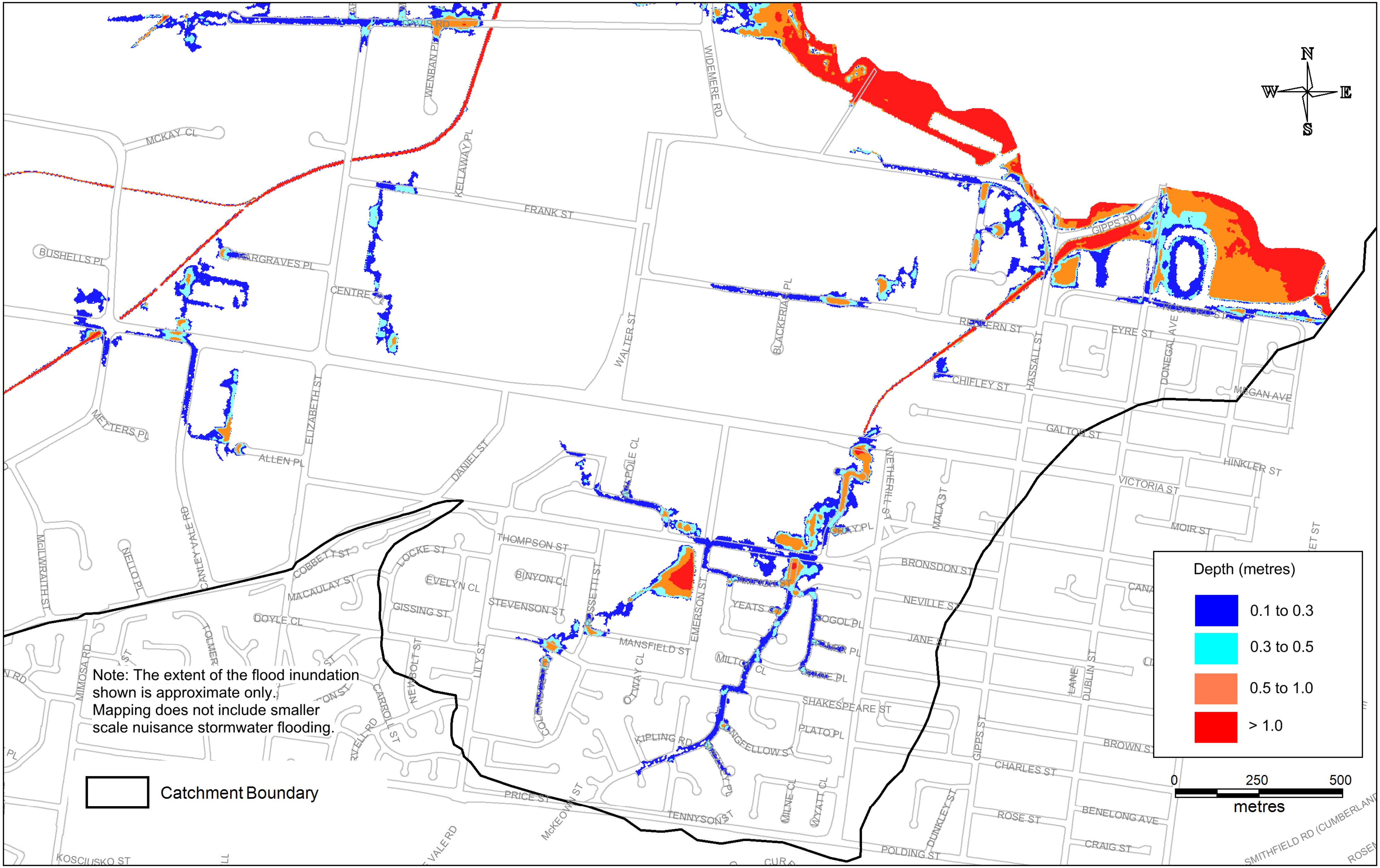
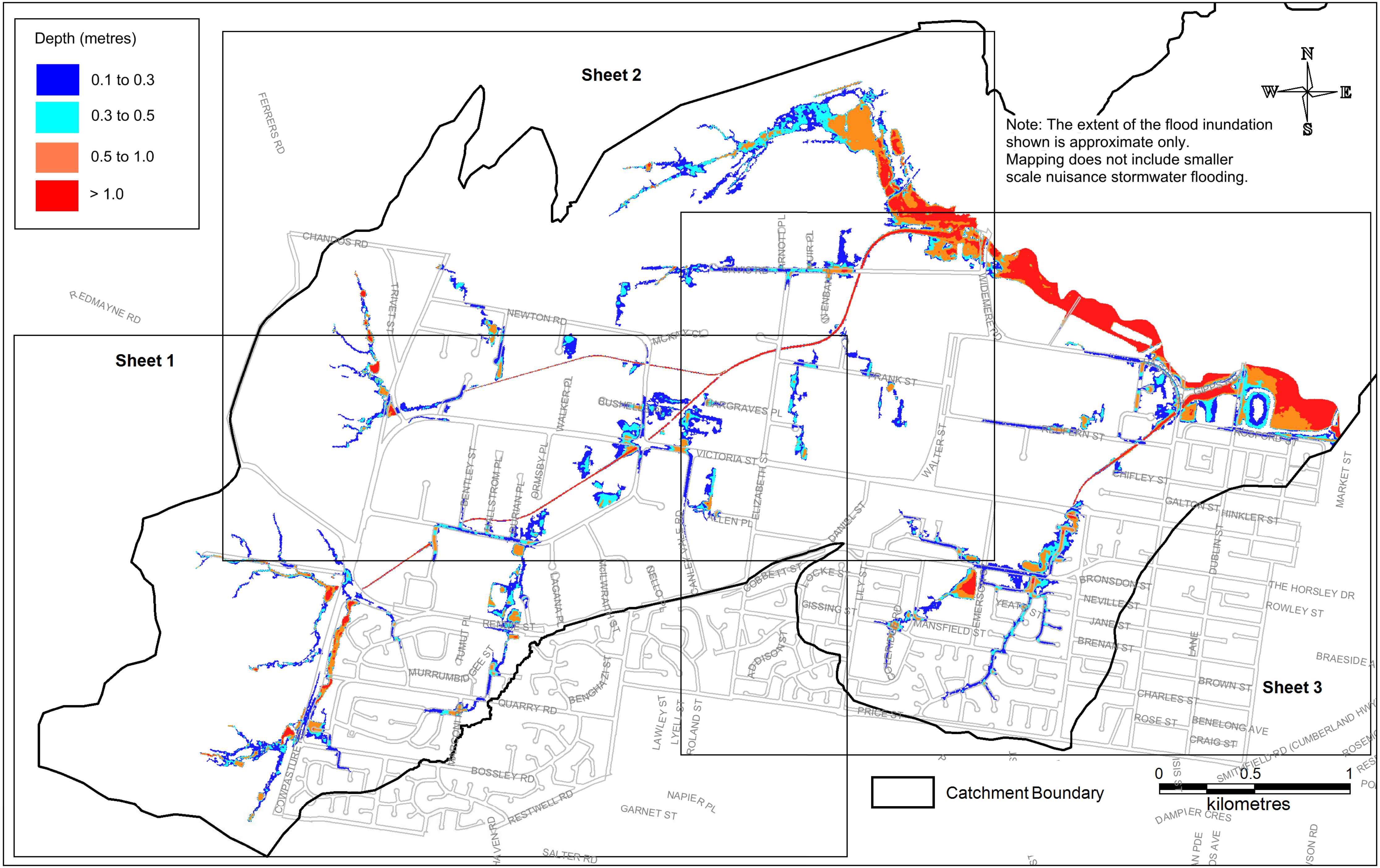


20Year ARI Flood Depths
Sheet 2



20Year ARI Flood Depths
Sheet 3



100Year ARI Flood Depths

Cardno

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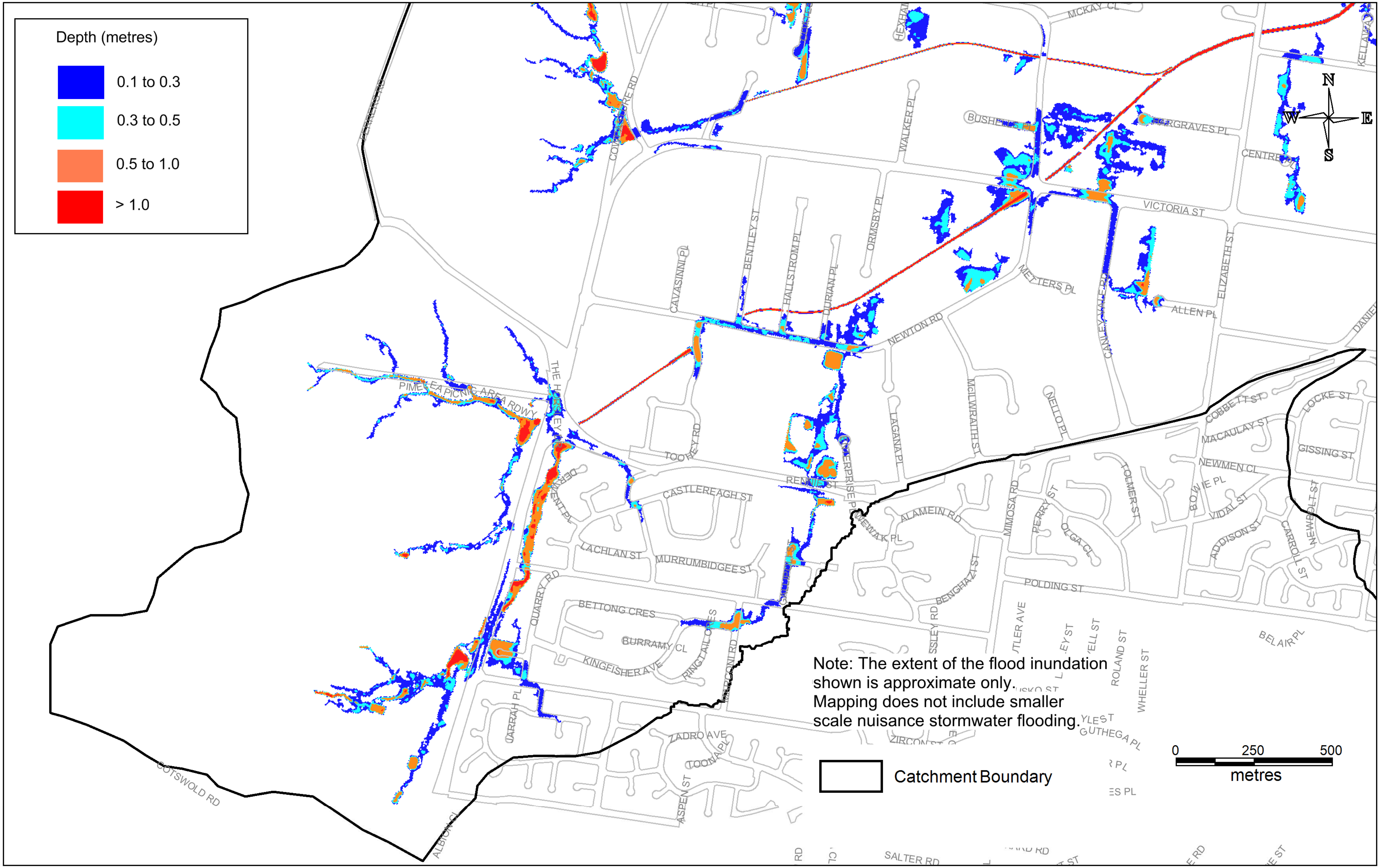
October 2014

Wetherill Park Catchment Management Plan



100Year ARI Flood Depths Sheet 1

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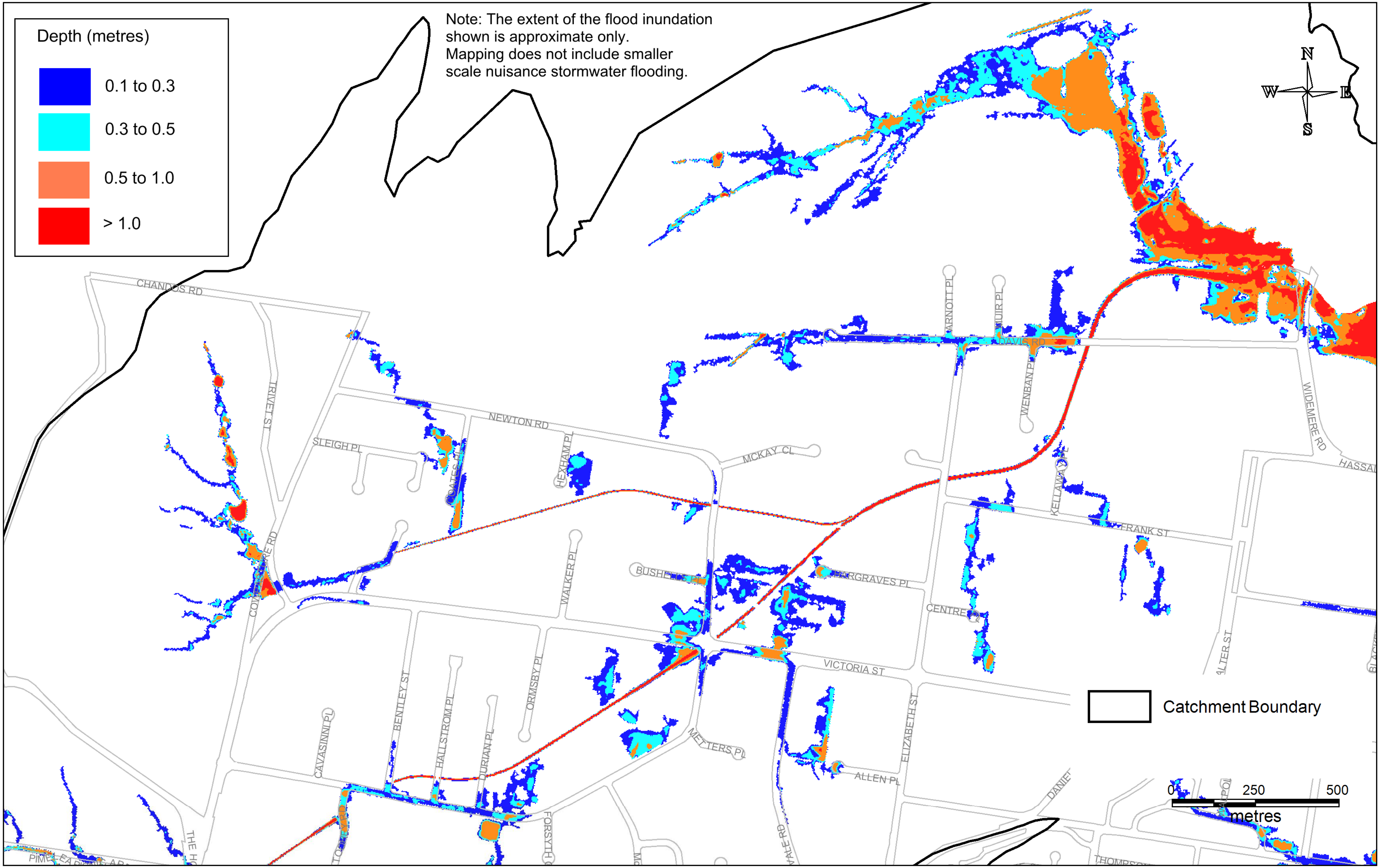

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100Year ARI Flood Depths
Sheet 2

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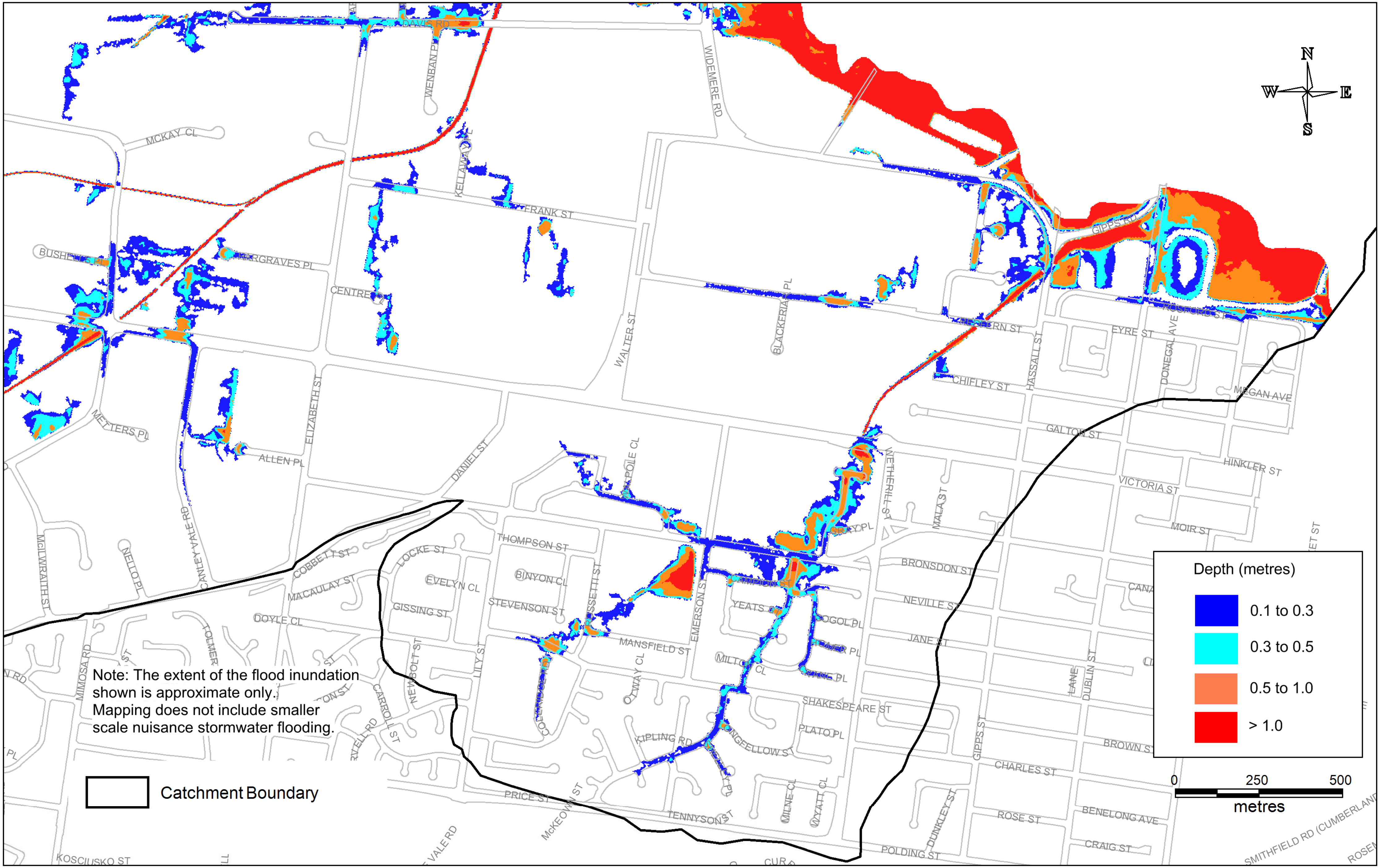
Wetherill Park Catchment Management Plan



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100Year ARI Flood Depths
Sheet 3

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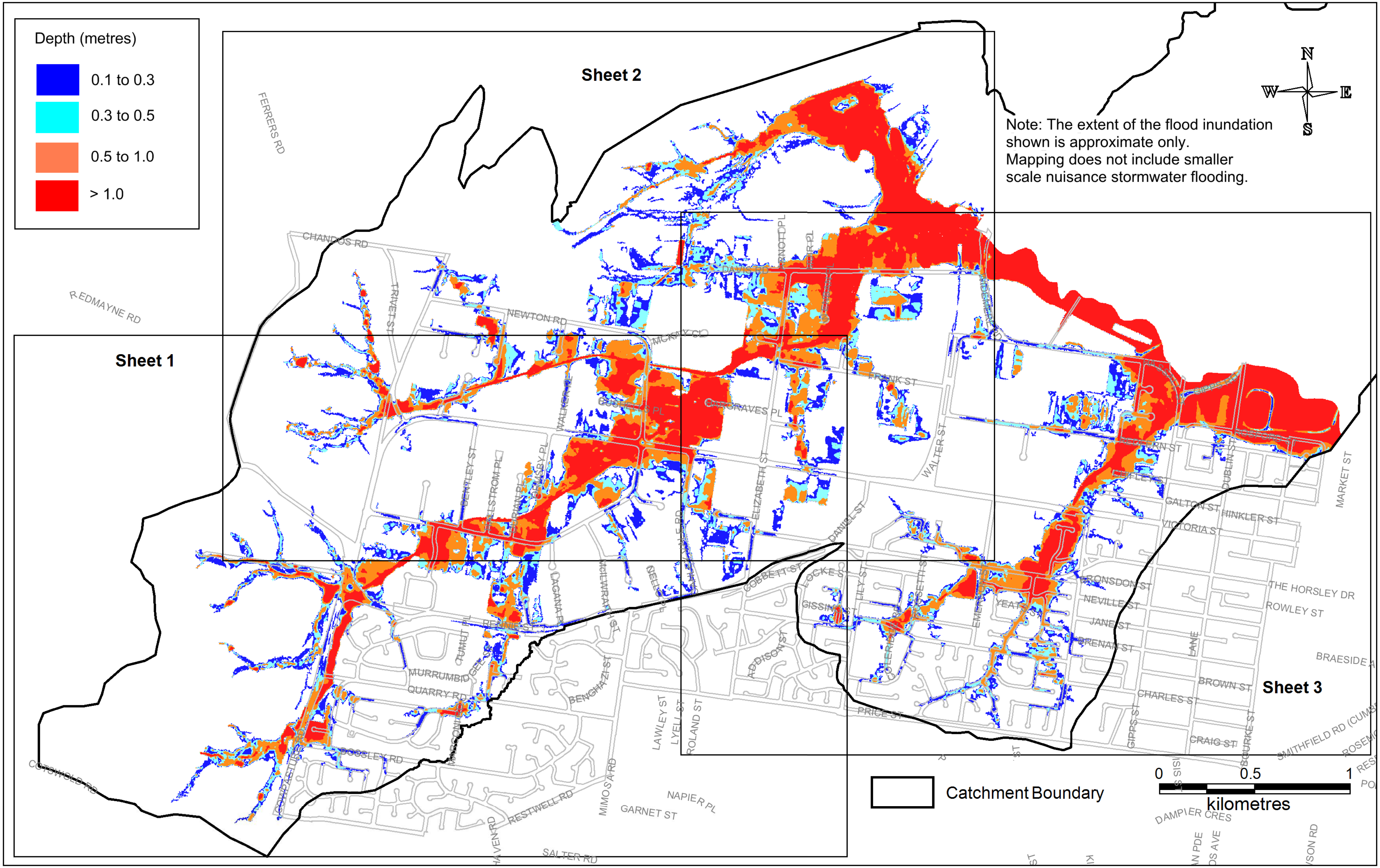
October 2014



PMF Flood Depths

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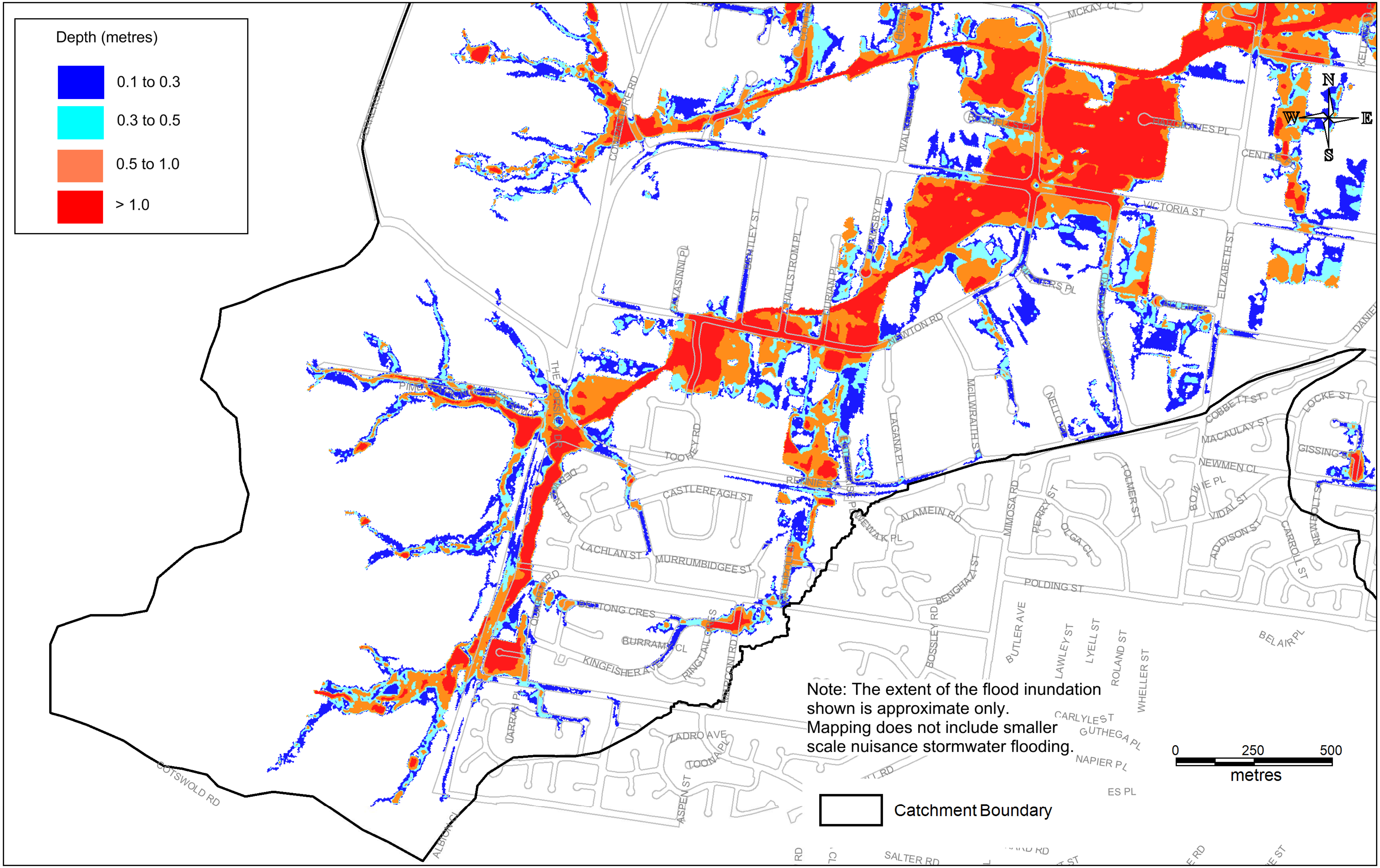
October 2014

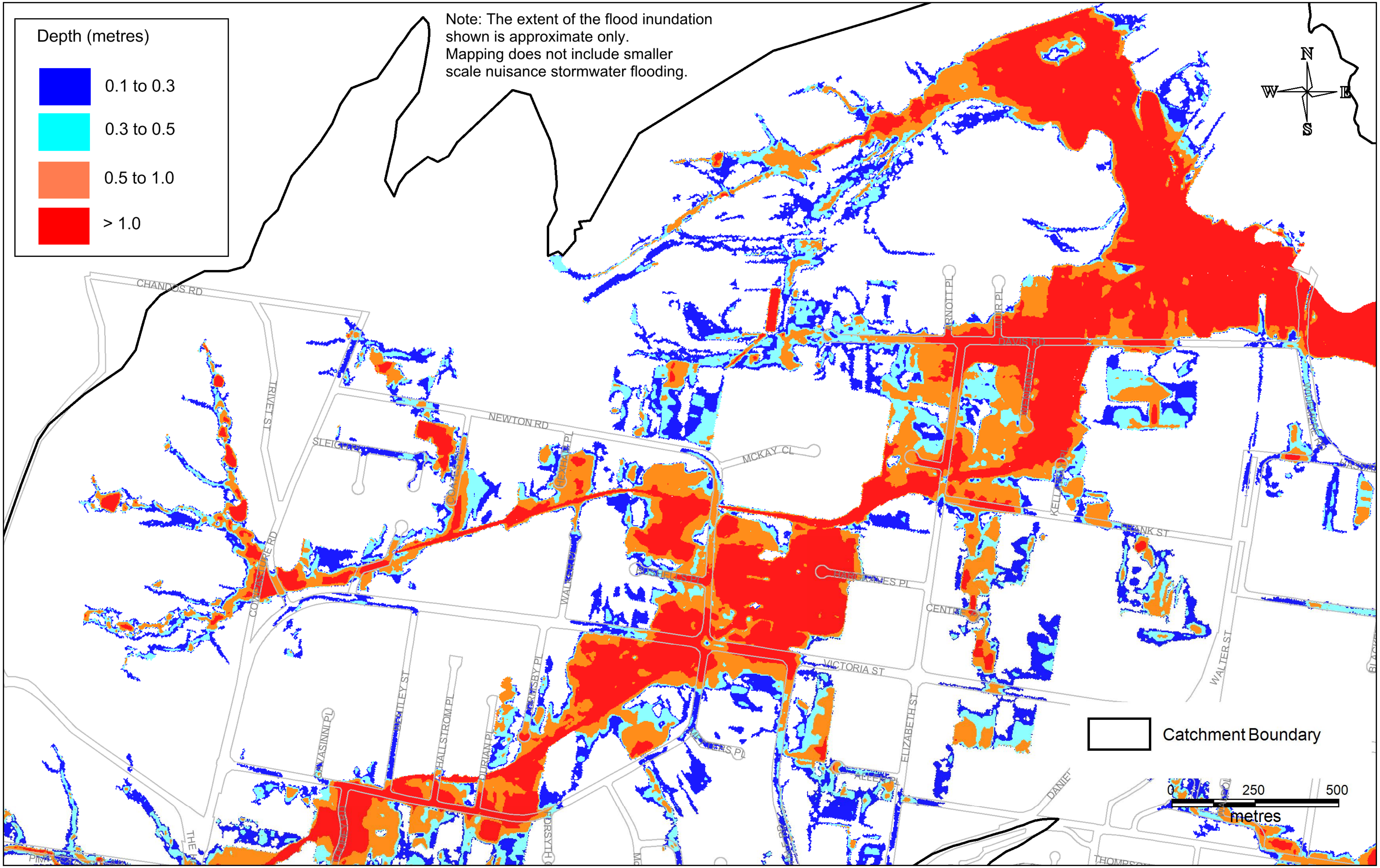
Wetherill Park Catchment Management Plan



PMF Flood Depths
Sheet 1

59914102
October 2014

Wetherill Park Catchment Management Plan



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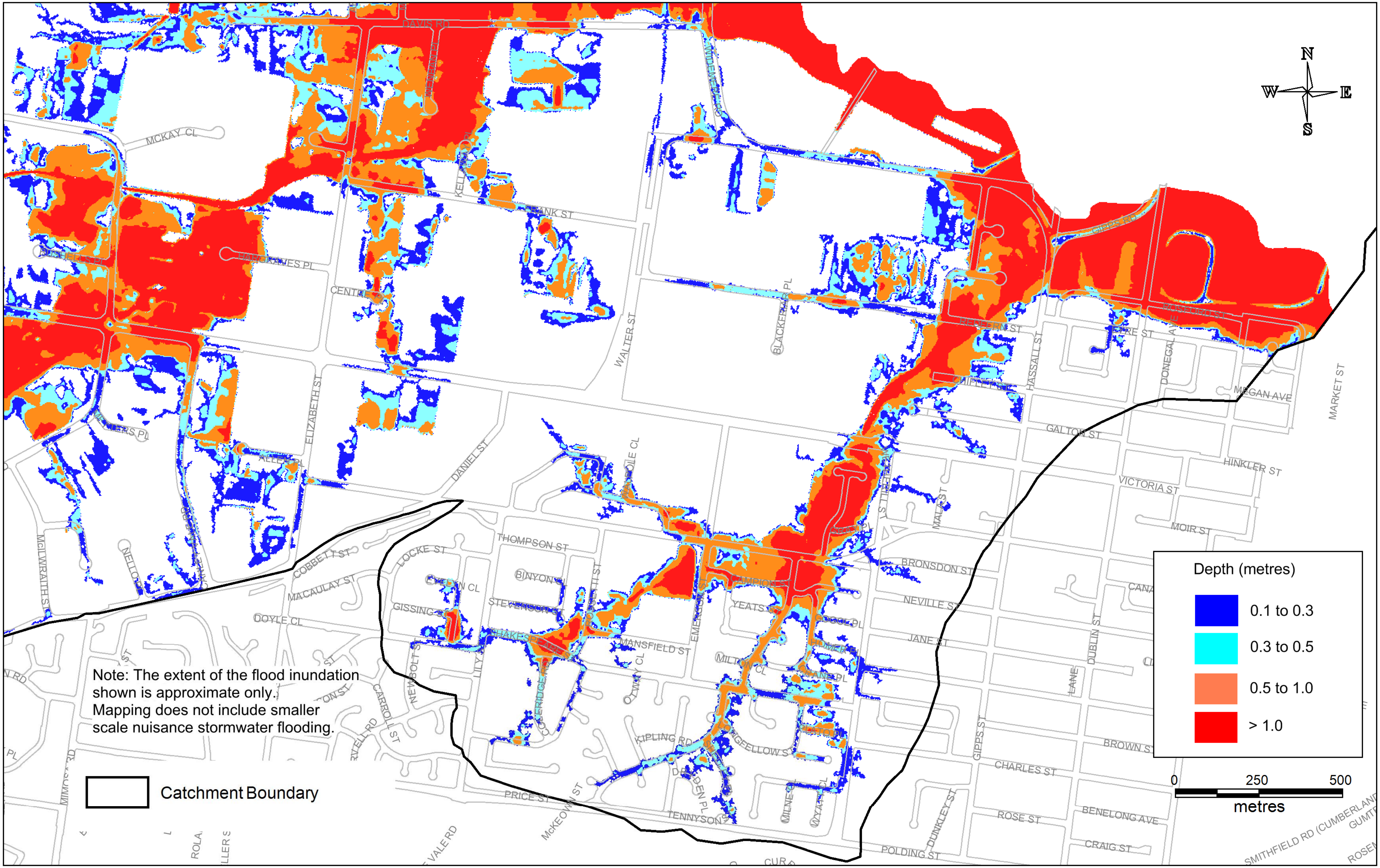
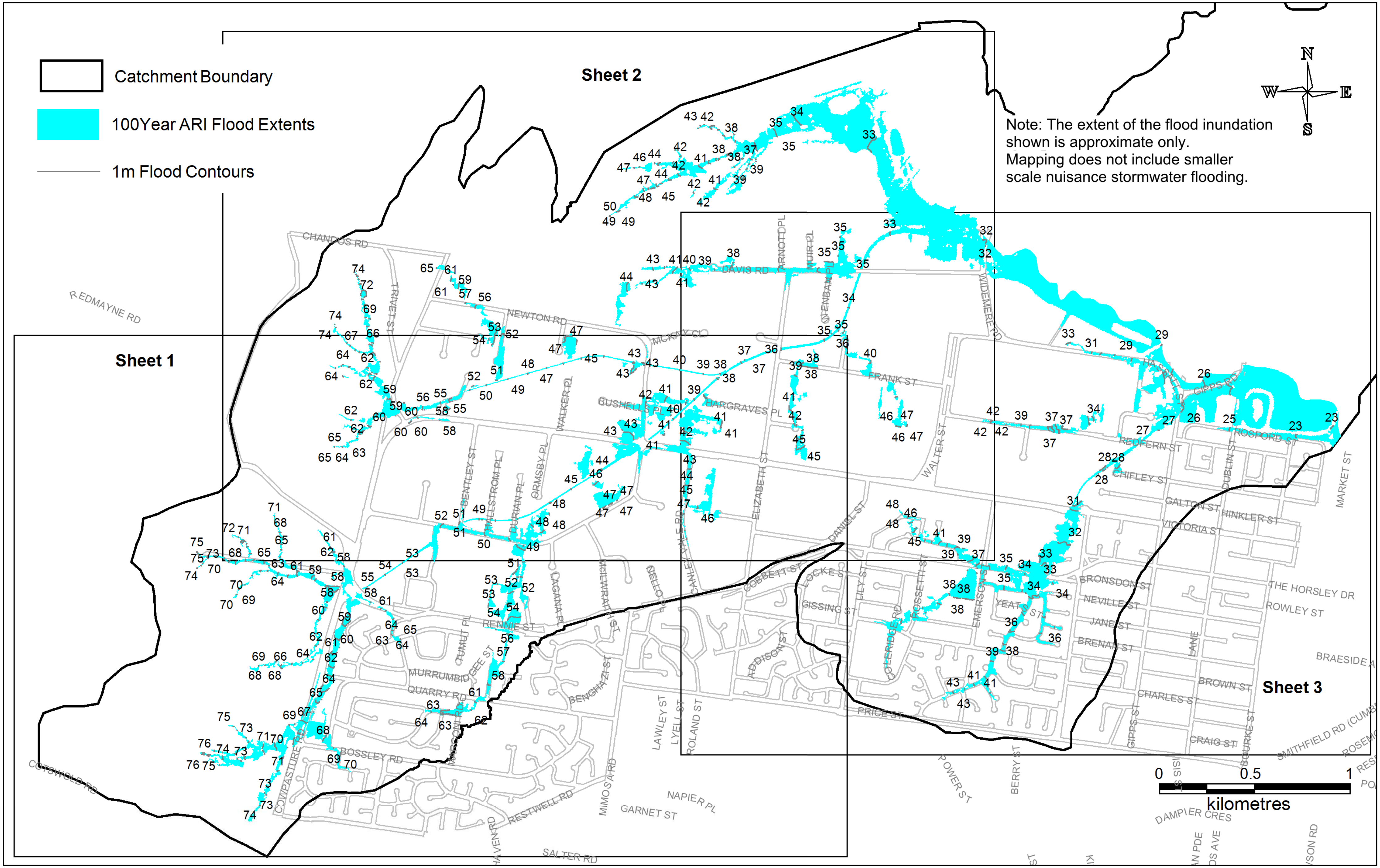
Wetherill Park Catchment Management Plan

PMF Flood Depths
Sheet 2



PMF Flood Depths
Sheet 3



100Year ARI Flood Level Contours

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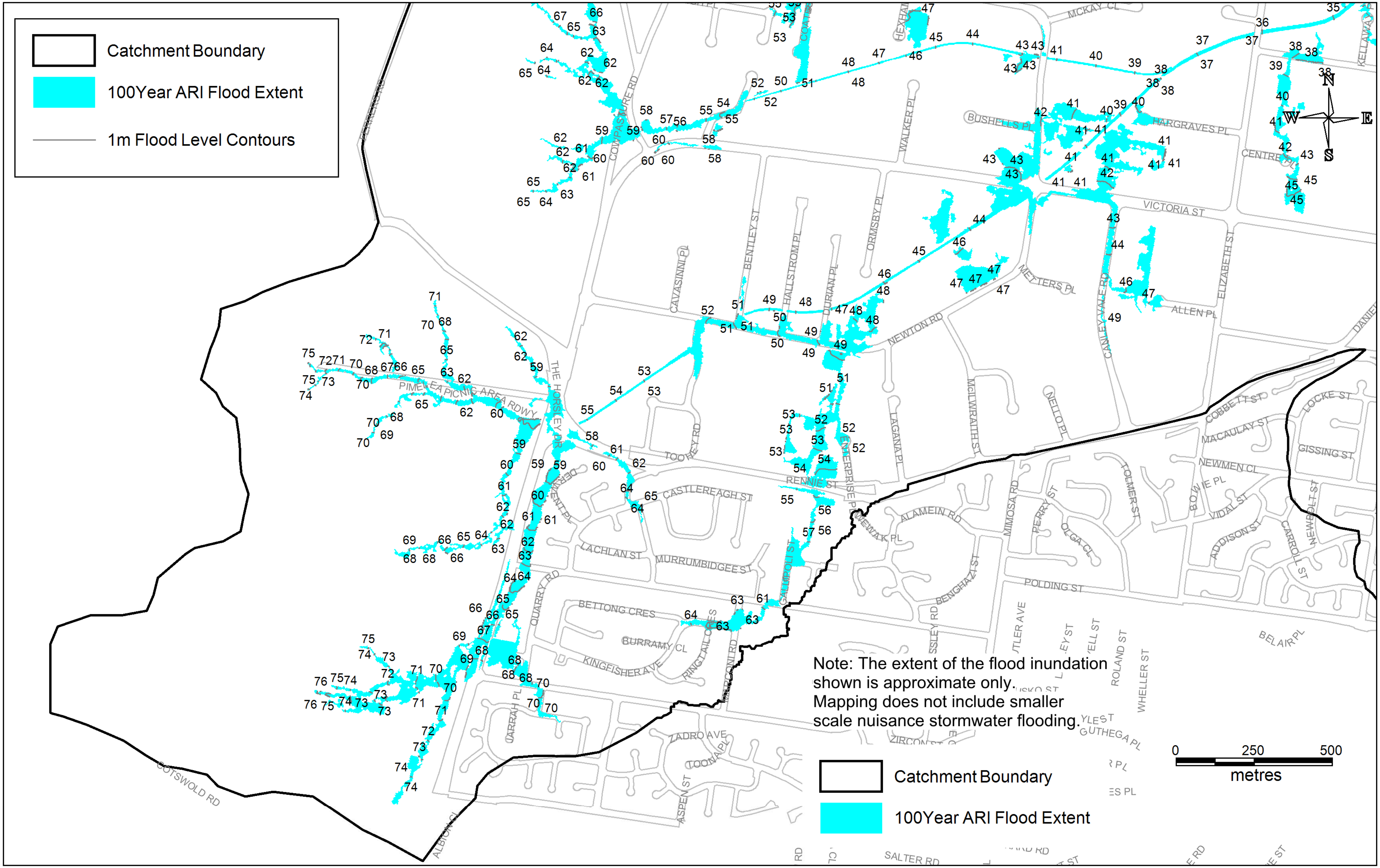

100Year ARI Flood Level Contours
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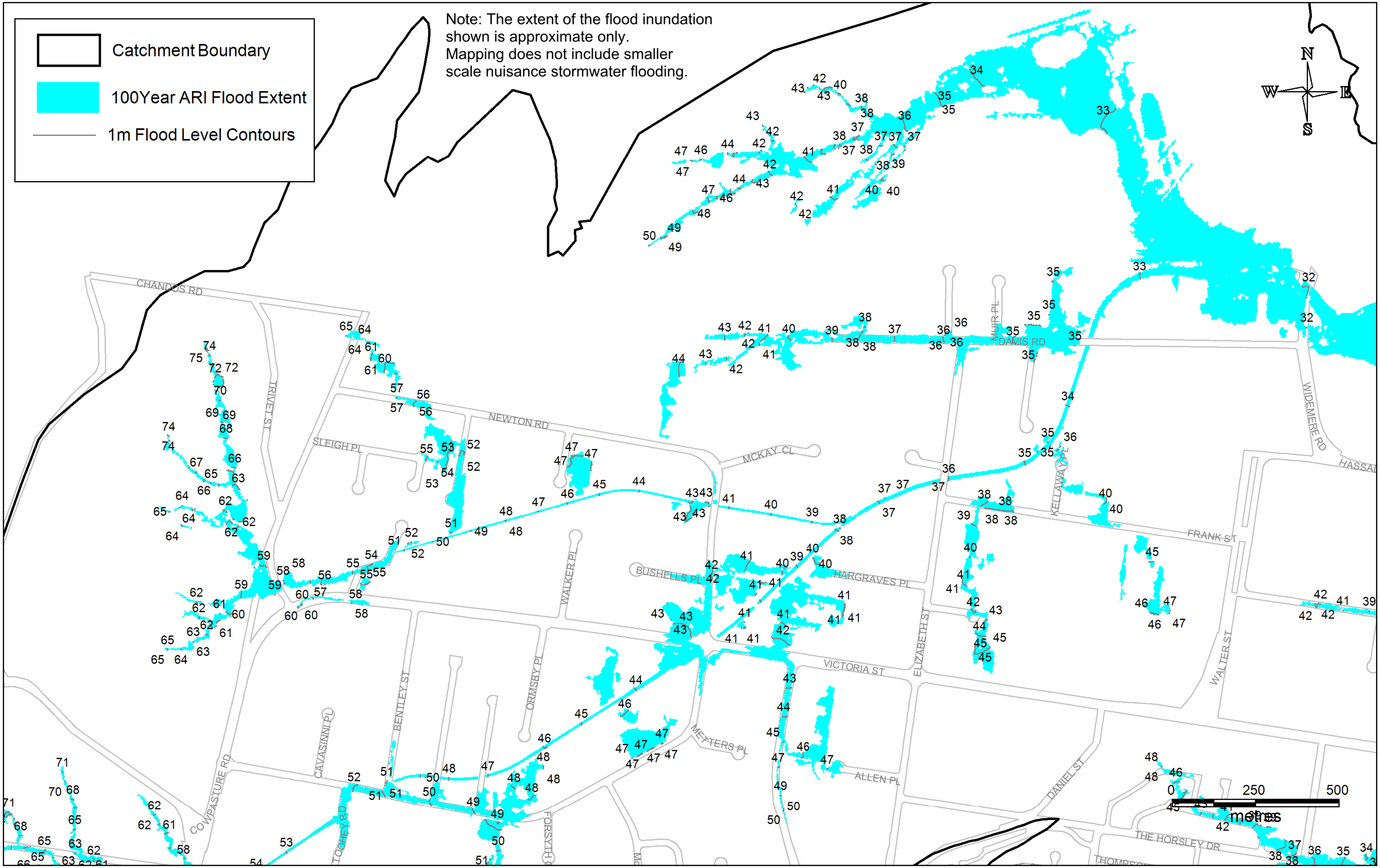

100Year ARI Flood Level Contours
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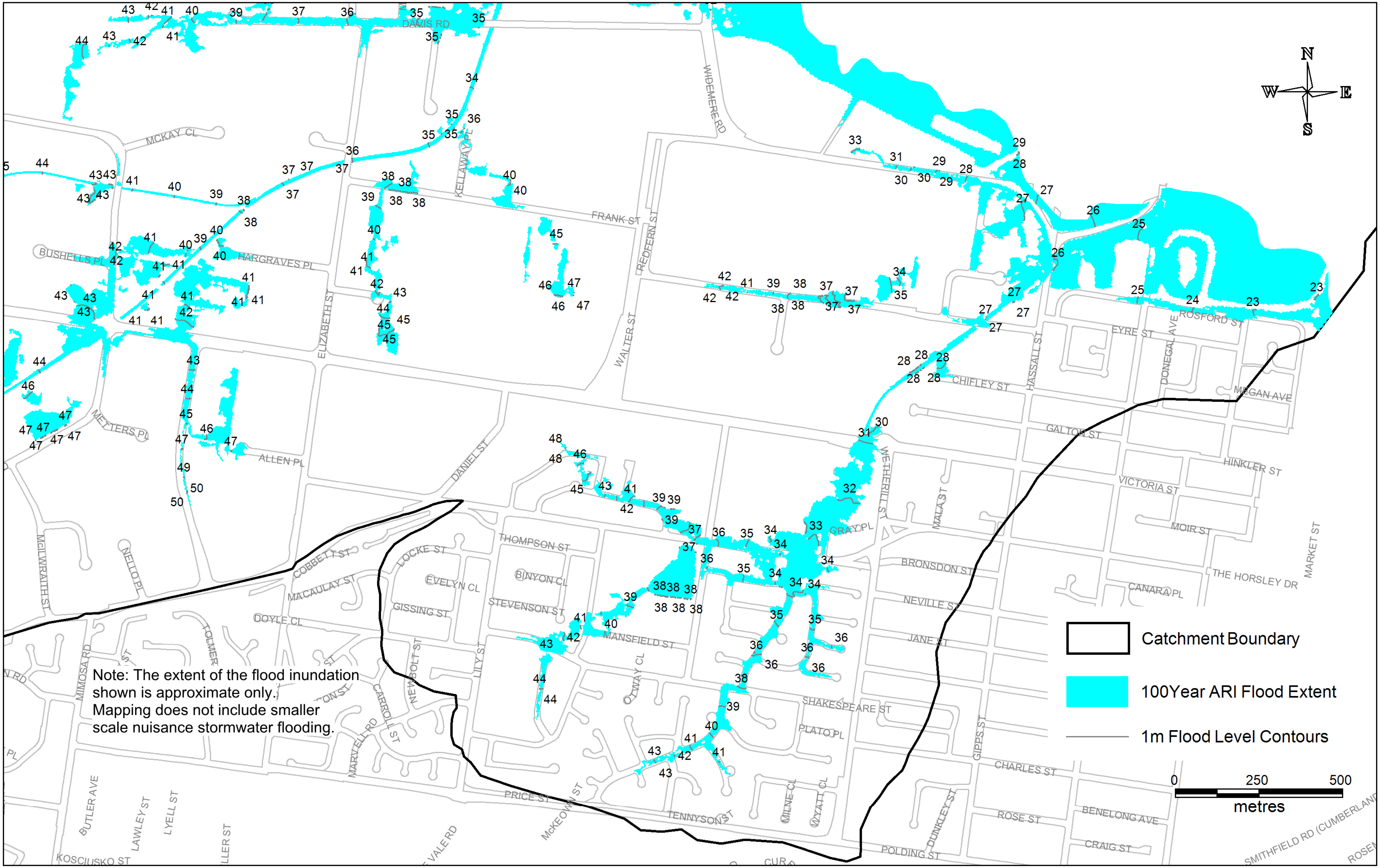

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100Year ARI Flood Level Contours
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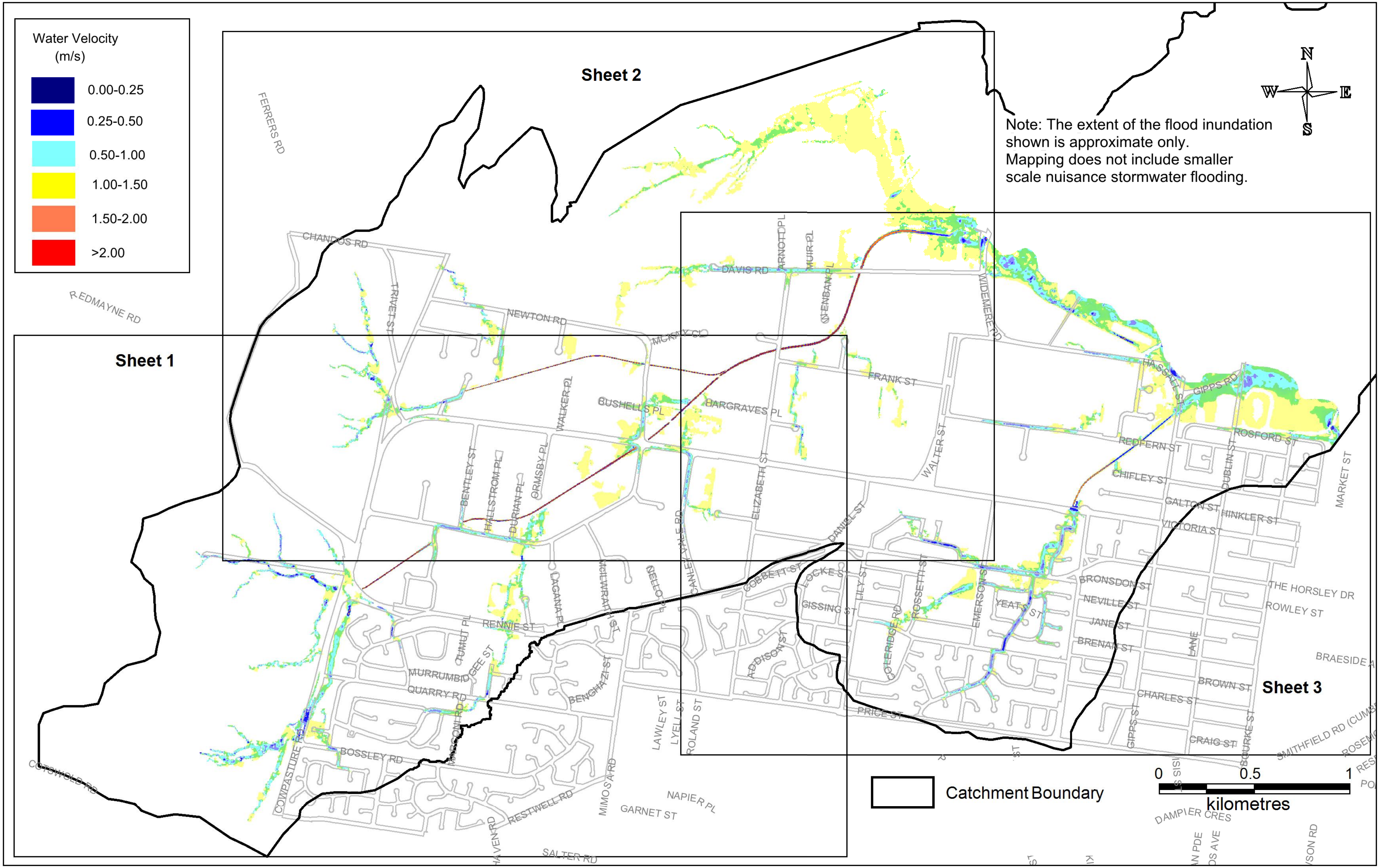
Wetherill Park Catchment Management Plan

Appendix D Flow Velocity Mapping

- Flow velocity grids for 100 year ARI and PMF events presented. This is the revised mapping produced by Cardno.



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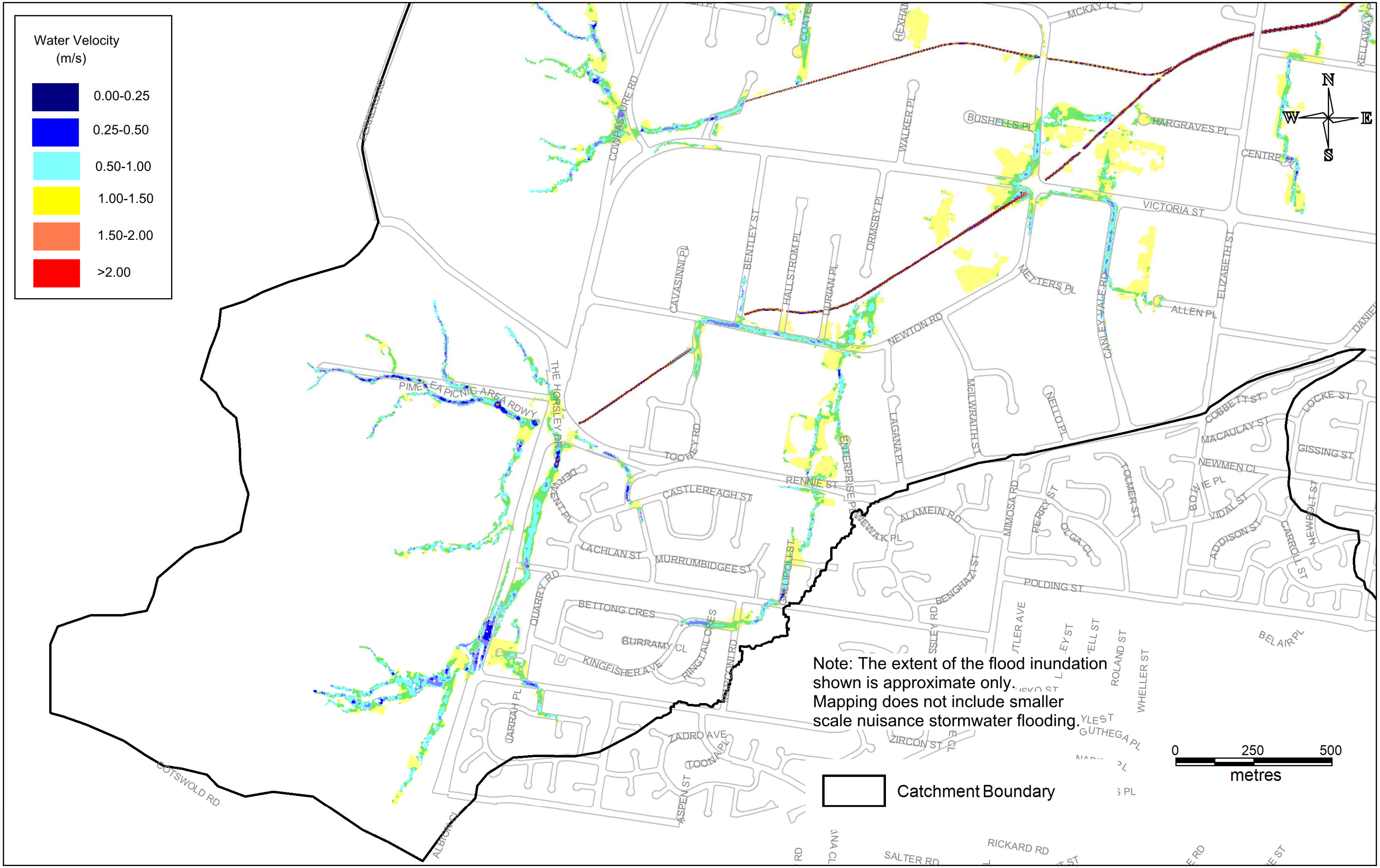

100Year ARI Flood Velocity
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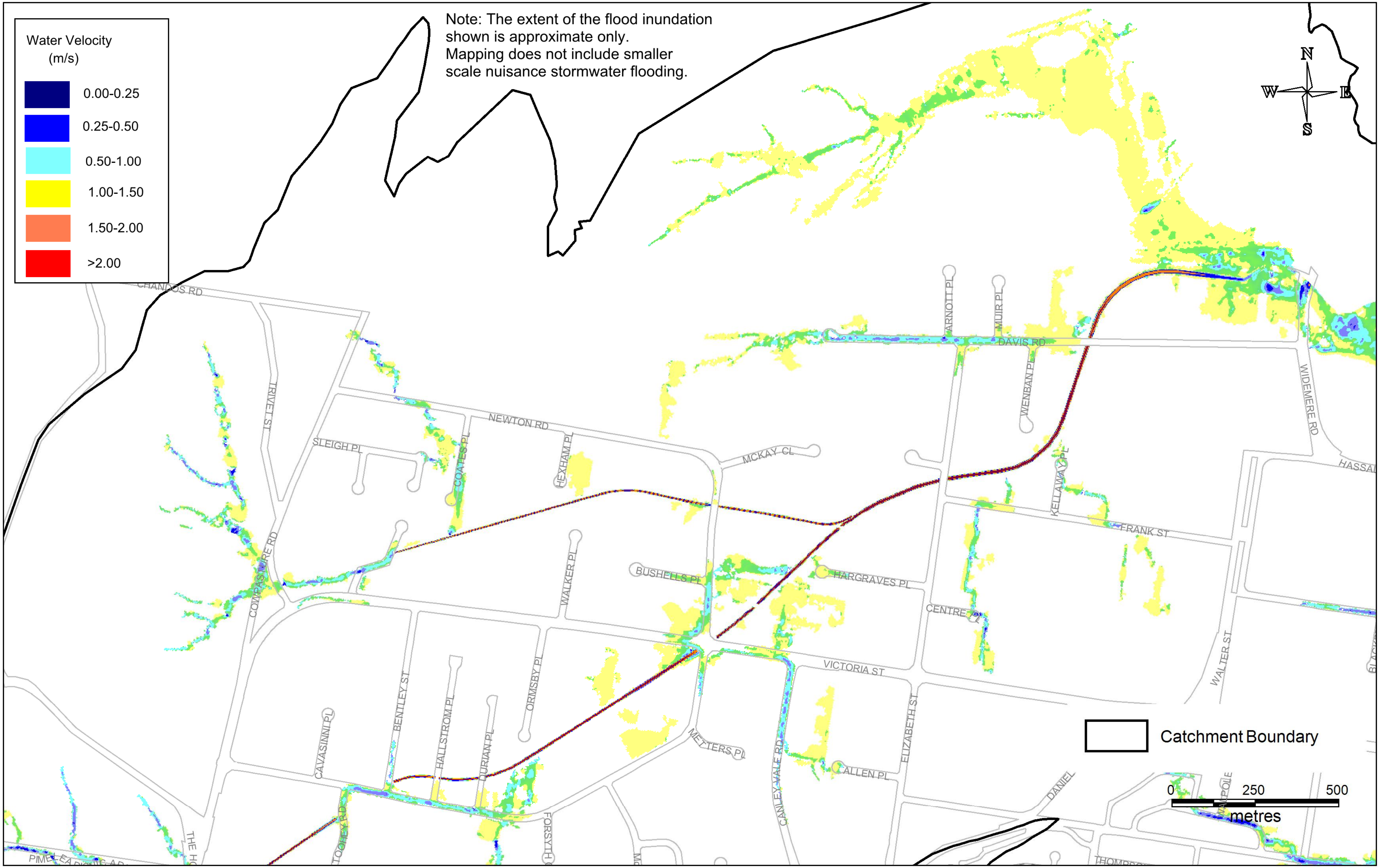

100Year ARI Flood Velocity
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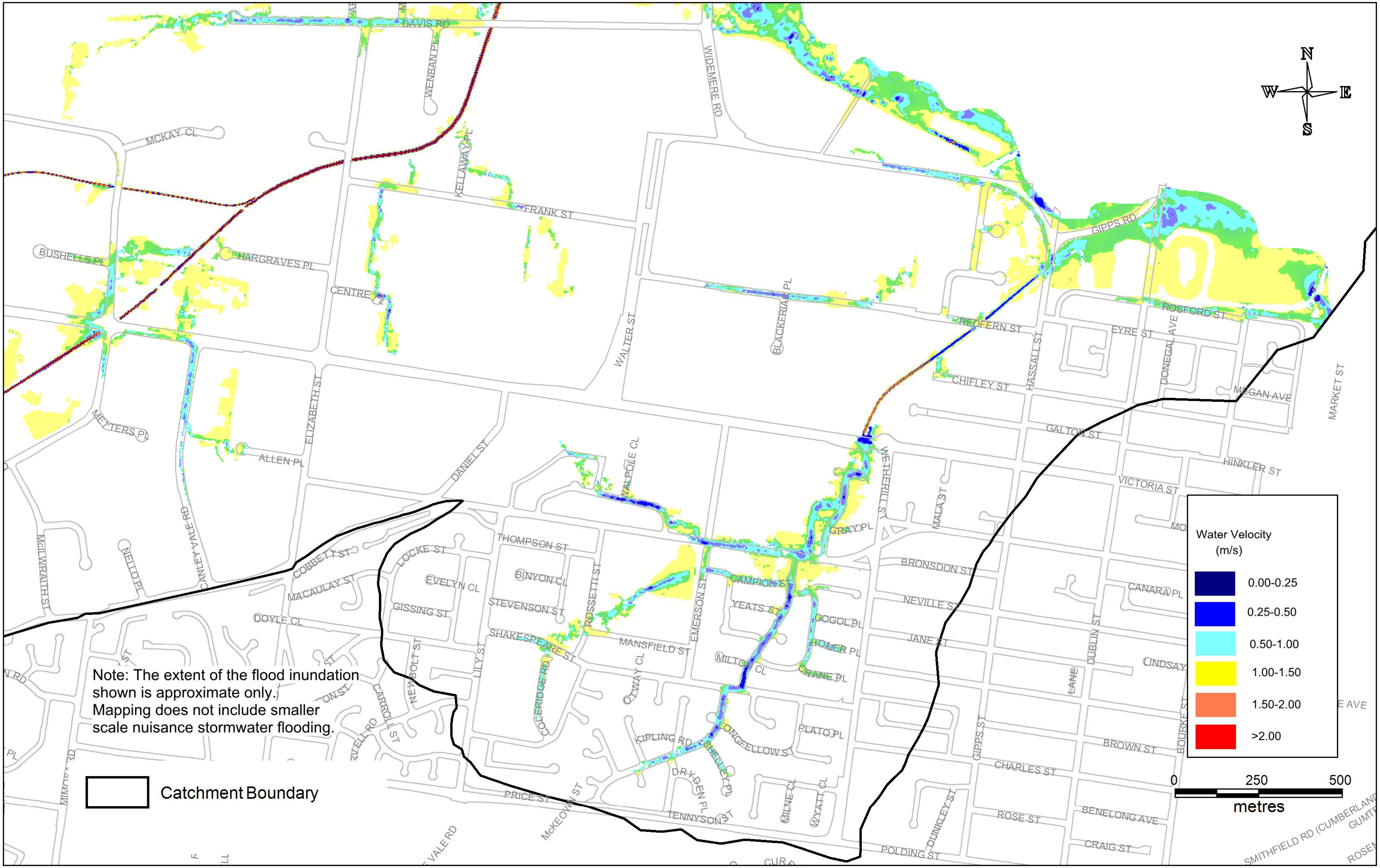

100Year ARI Flood Velocity
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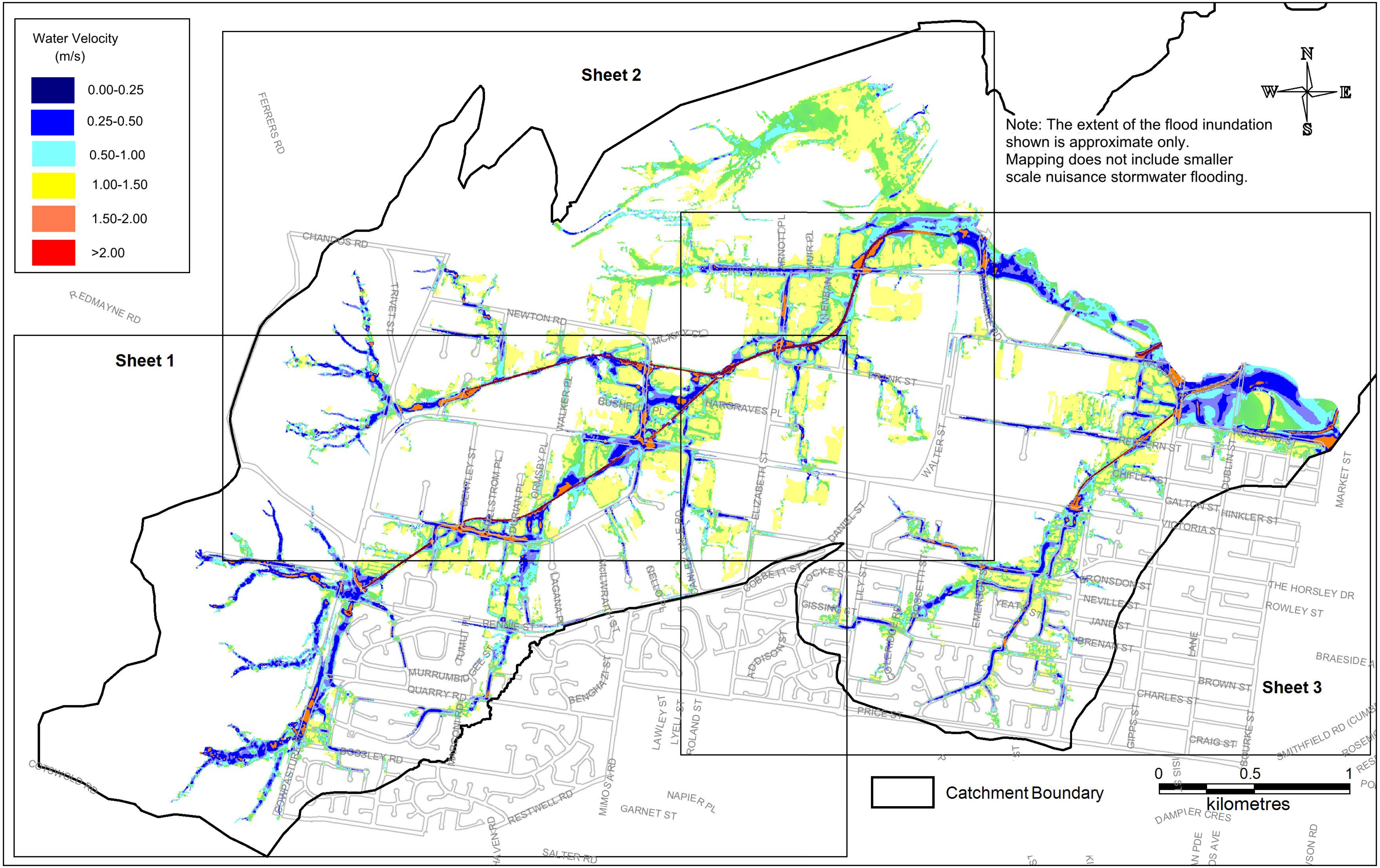

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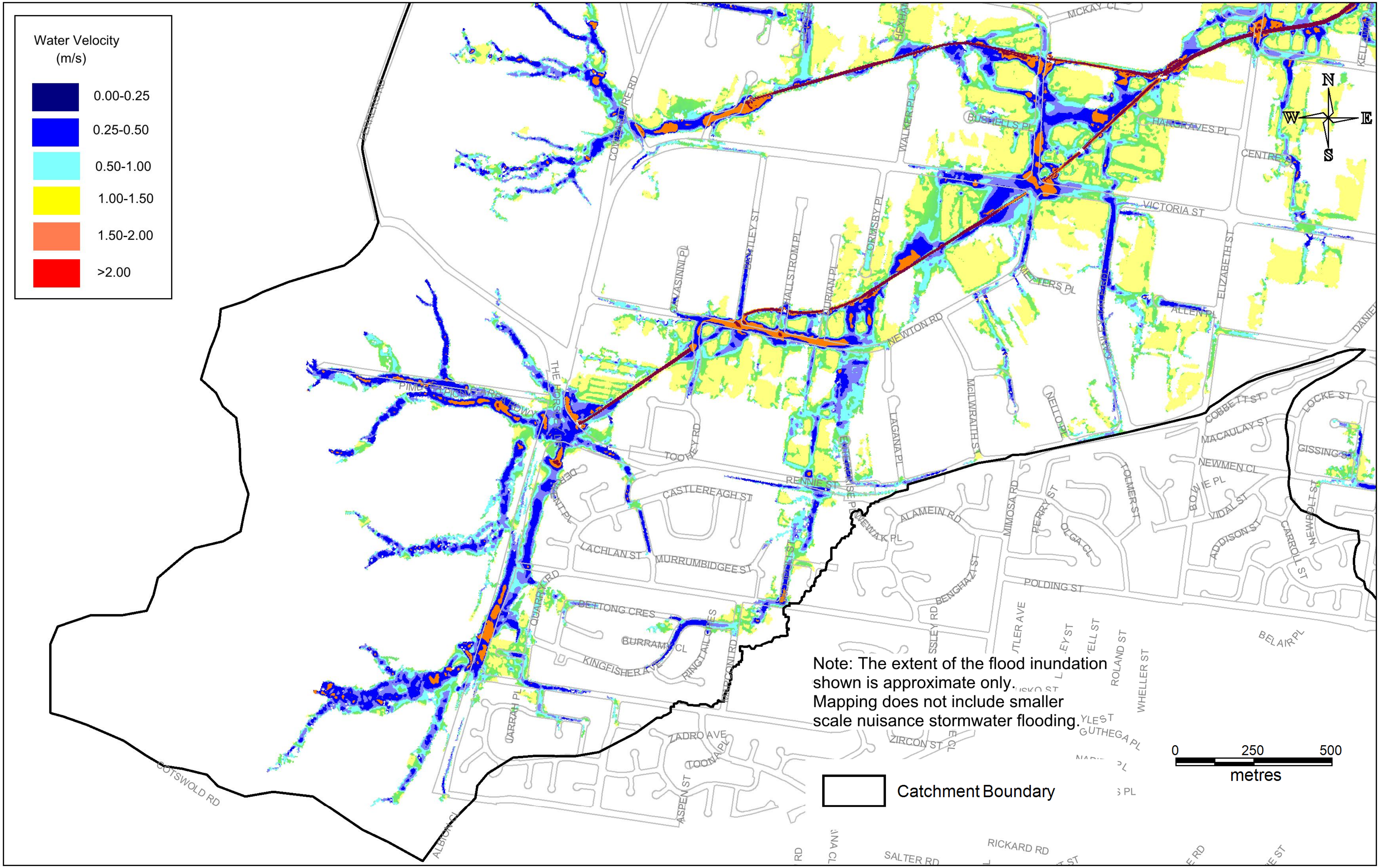

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PMF Flood Velocity
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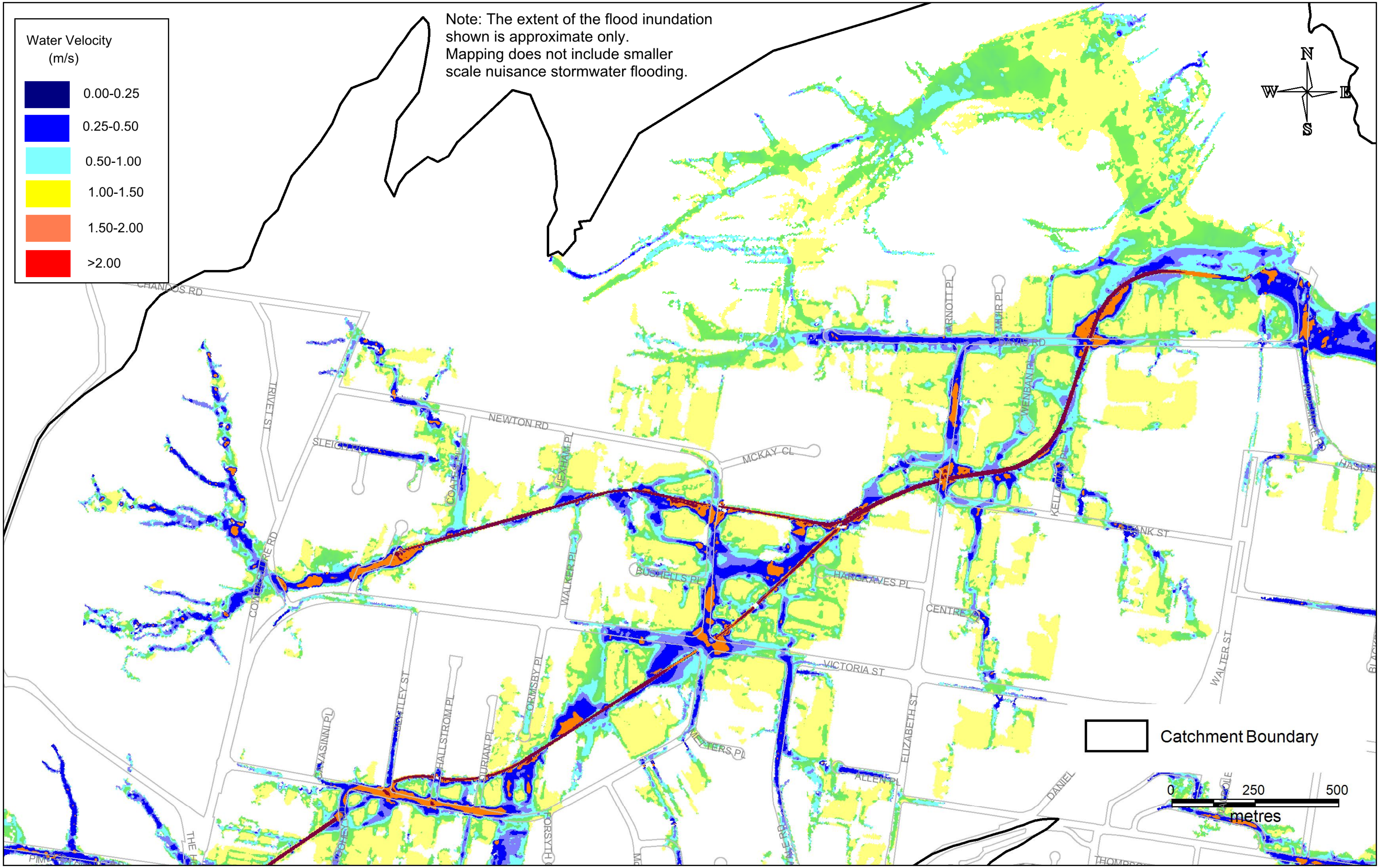

PMF Flood Velocity
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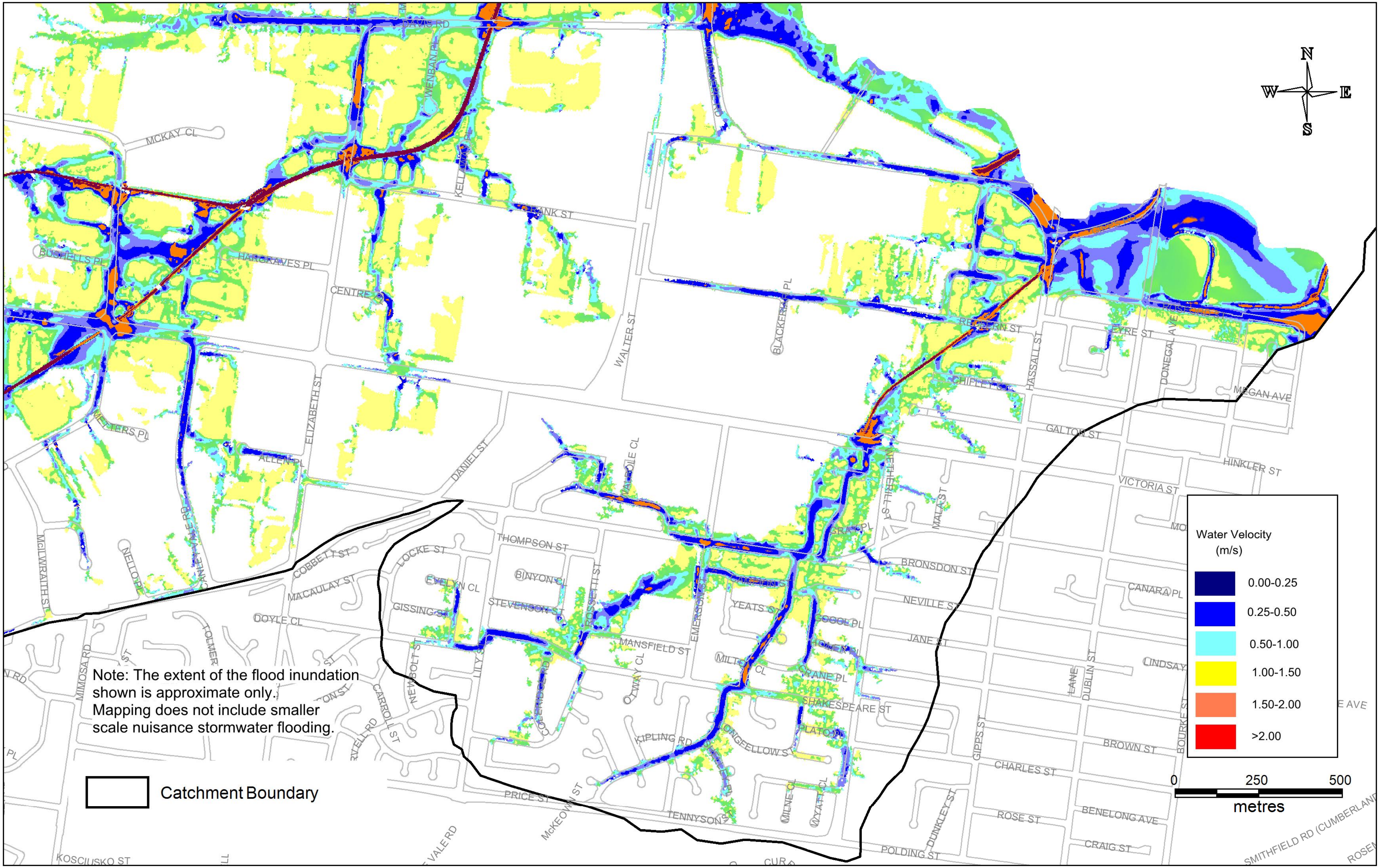
Wetherill Park Catchment Management Plan



PMF Flood Velocity
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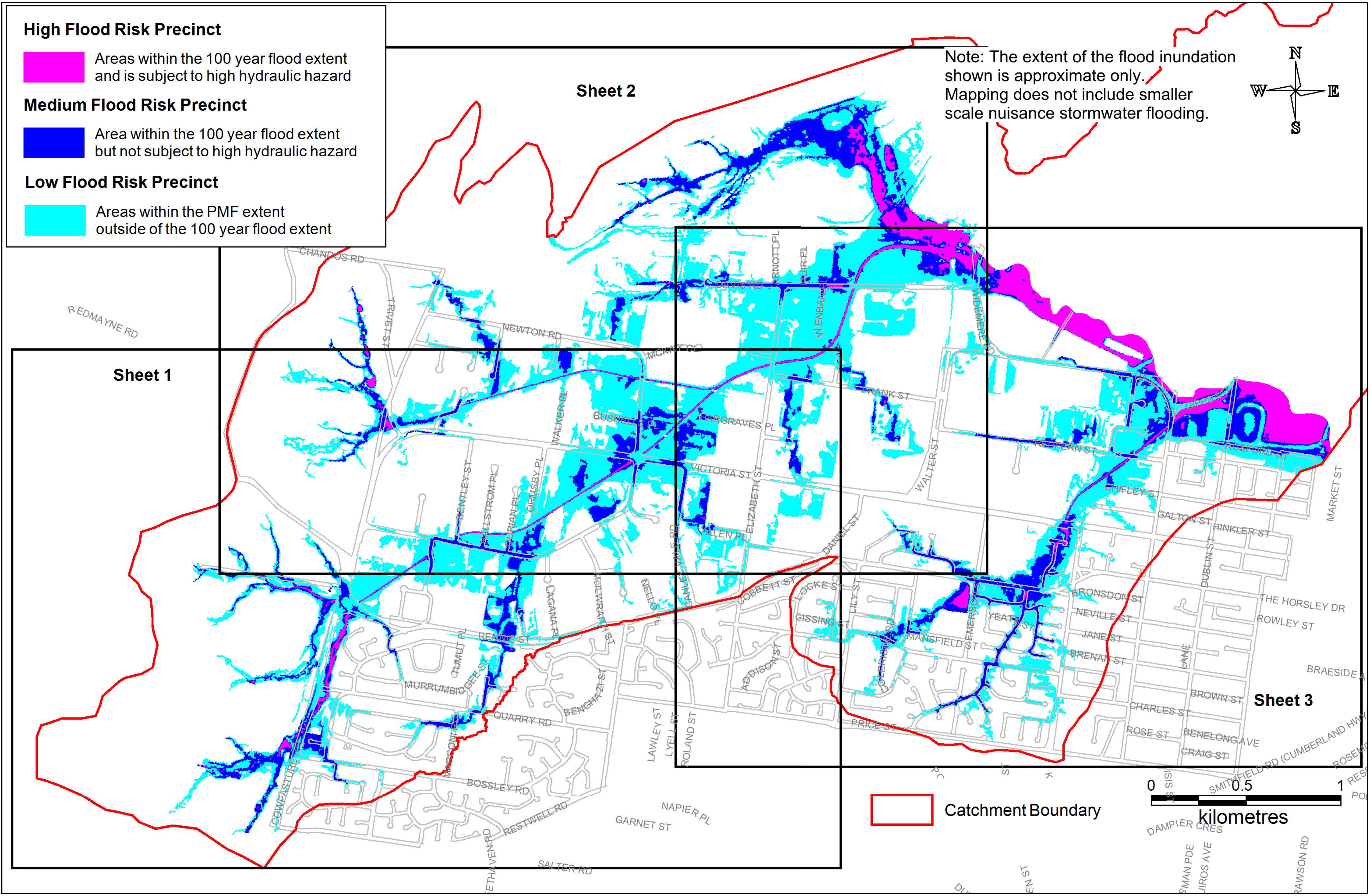
Appendix E Peak Water Levels

Table E-1 Design Peak Water Levels at Calibration Locations

Location	High Water Level (m AHD)					
	Calibration (Measured)	20 year	100 year	500 year	10000 year	PMF
Main Channel						
Cowpasture Rd/The Horsley Drive (culvert inlet)	57.07	57.30	57.91	58.33	58.67	58.95
Cowpasture Rd/The Horsley Drive (on roundabout)	57.73	57.71	57.71	57.82	57.99	58.14
Toohey Rd (culvert inlet)	51.52	51.59	52.40	52.73	53.40	54.27
Hallstrom Place (bridge outlet)	48.25	48.21	48.34	48.49	48.97	50.14
Newton Rd/Victoria Street (culvert outlet)	41.34	40.80	40.88	40.94	41.71	42.86
Elizabeth St (bridge outlet)	35.86	35.79	36.01	36.41	37.25	37.87
Snow Confectionary Pty Ltd (34 Davis Road)	32.68	32.69	32.86	33.06	33.58	34.31
Tributary 1						
Newton Rd (culvert inlet)	42.35	42.54	42.88	43.54	43.88	44.13
Rosford Street						
Hassall St (culvert inlet)	26.23	26.51	26.66	26.78	27.05	27.51
Hassall St (culvert outlet)	25.80	25.92	26.02	26.12	26.37	26.77
Prospect Creek						
Gipps Rd (culvert outlet)	24.67	24.66	24.77	24.99	25.50	25.93
Rosford Reserve (outlet)	22.53	22.88	23.14	23.30	23.53	23.84
Other Locations						
Emerson Street Reserve (in basin)	37.64	37.85	37.91	37.96	38.09	38.22

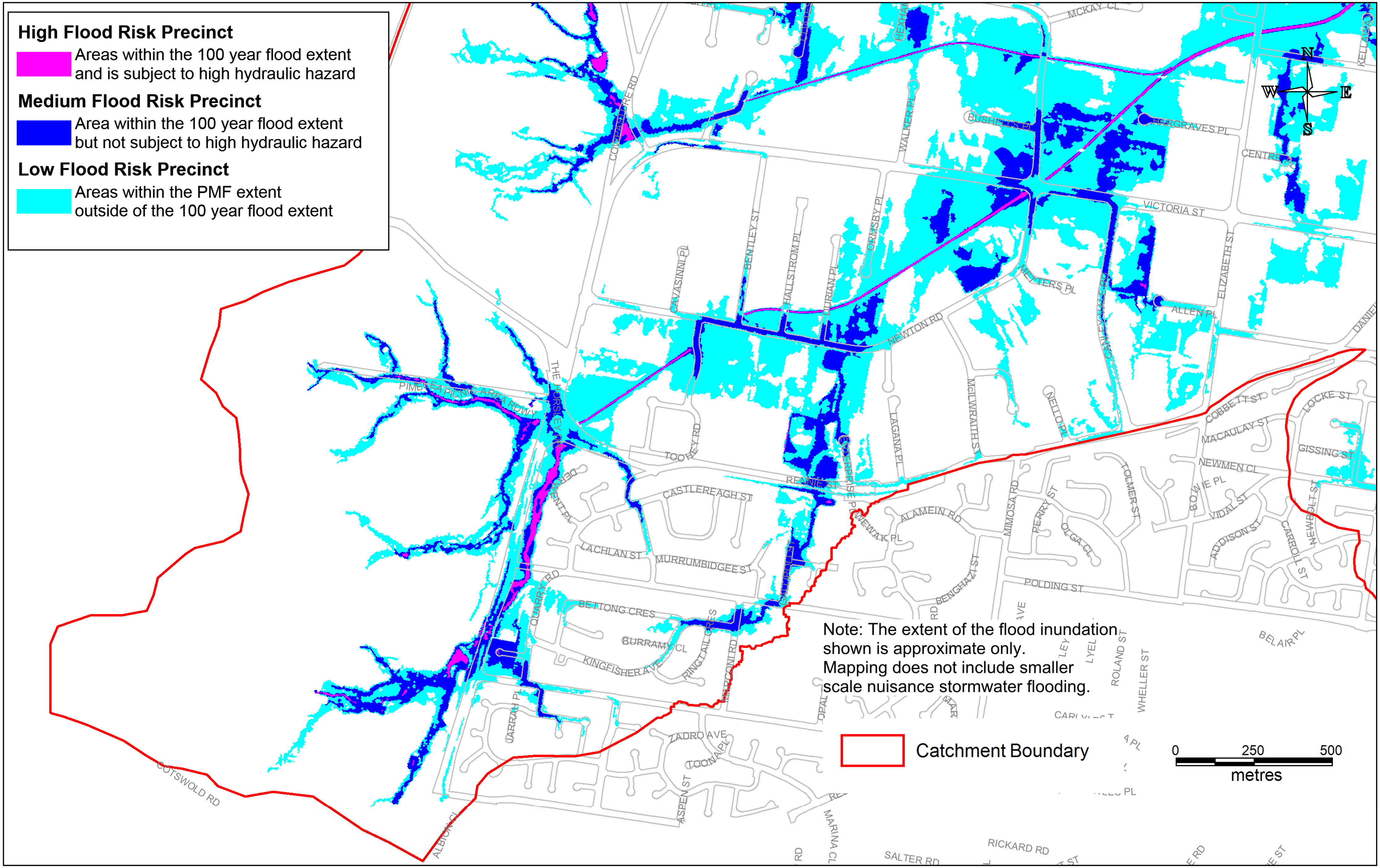
Note:
Peak water levels are provided for the critical storm duration, which is the 2 hour duration storm for all design events except for the PMF, where the 45 minute duration storm is critical.

Appendix F Flood Risk Precinct Mapping



Wetherill Park Catchment Management Plan

Provisional Flood Risk Precincts



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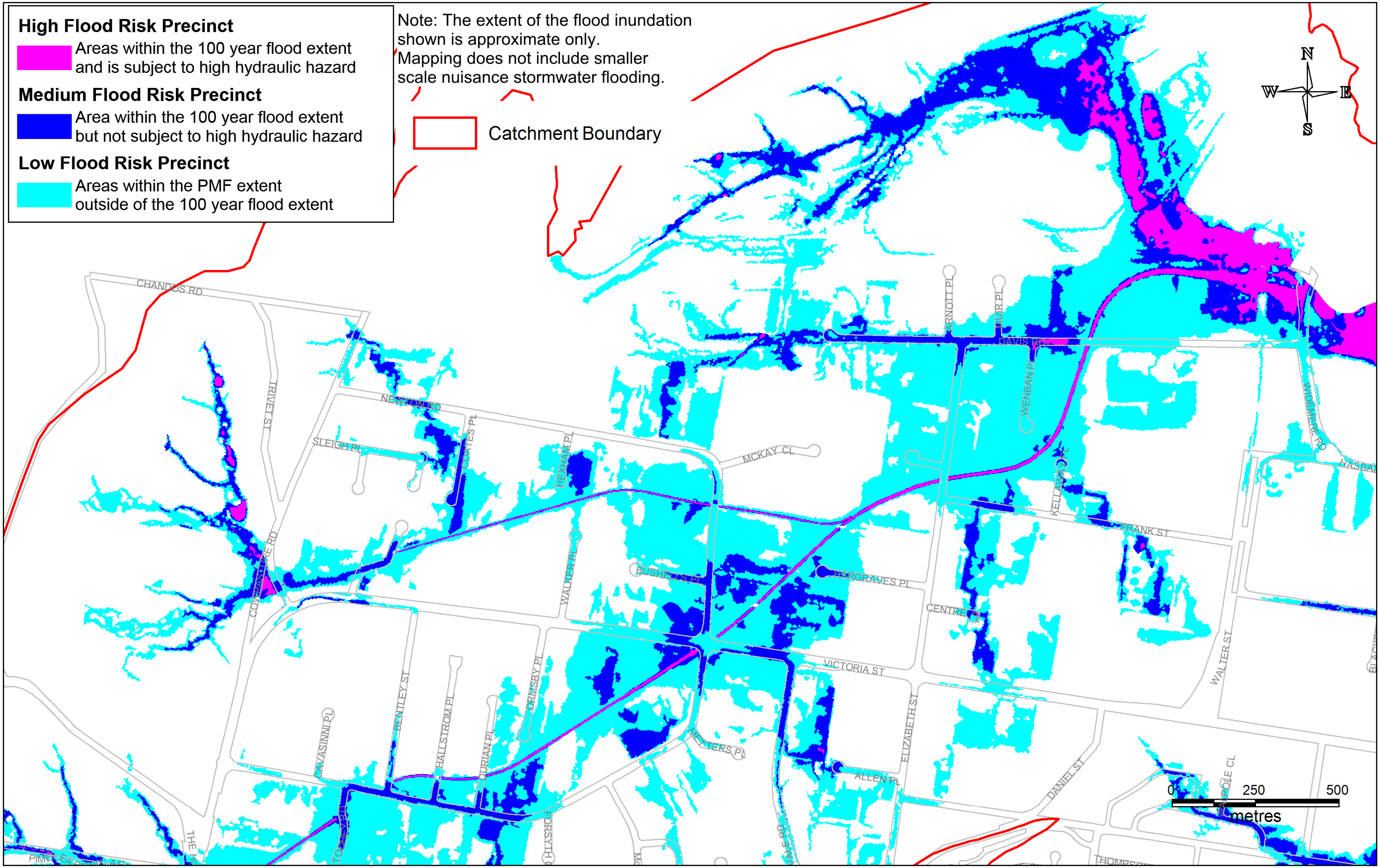
Provisional Flood Risk Precincts
Sheet 1

Wetherill Park Catchment Management Plan



Provisional Flood Risk Precincts
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Wetherill Park Catchment Management Plan

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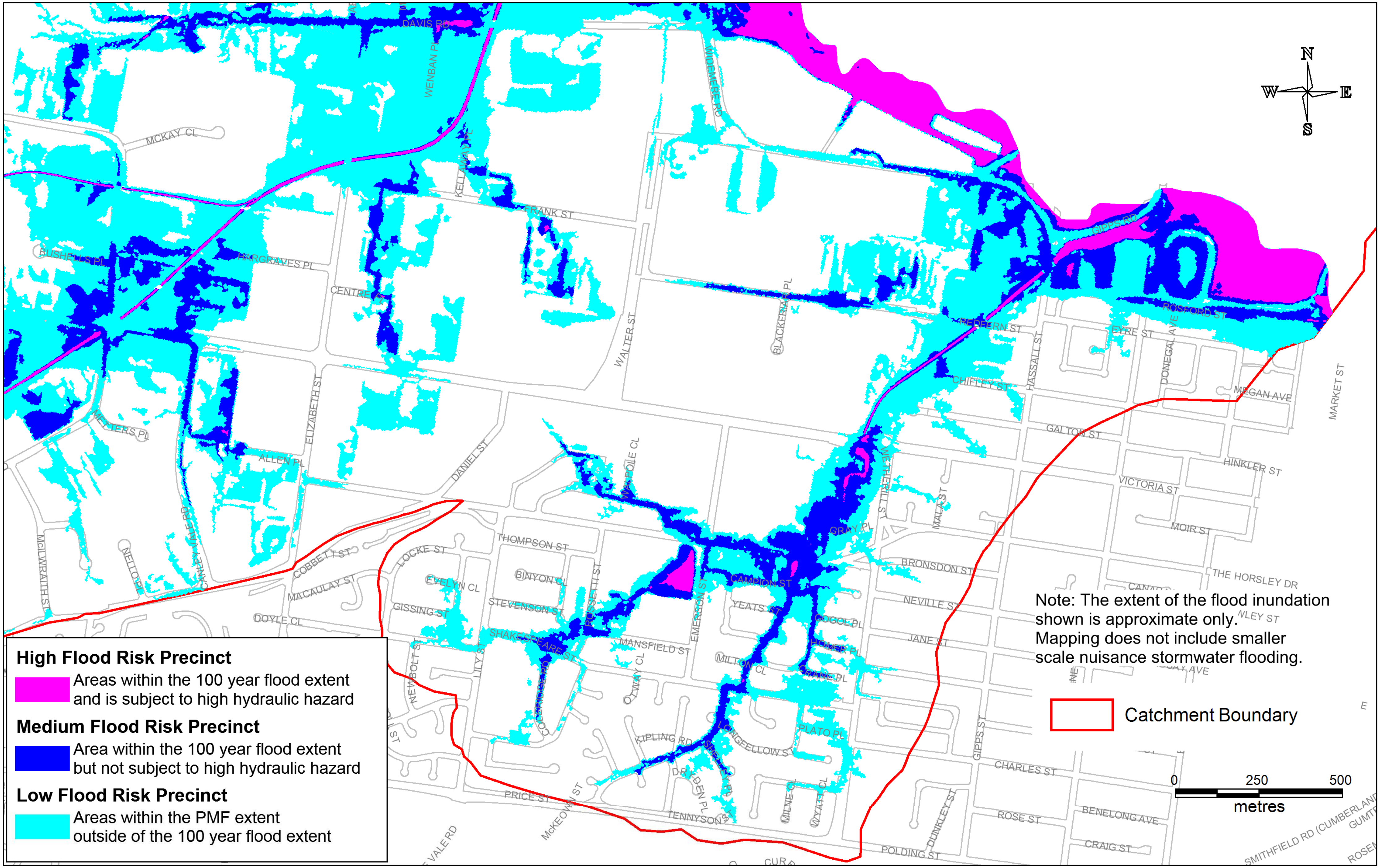

Provisional Flood Risk Precincts
Sheet 3

Appendix G Model Quality Assurance Review Recommendations

A technical review of the Wetherill Park TUFLOW model was undertaken by BMT-WBM, the developers of TUFLOW.

The flood events reviewed for the model were the 18th April 2012 Calibration event, and the 1:100 year ARI design event (2 hour storm duration).

A copy of the review is provided below.

1. Model Configuration

The following table summarises the key model characteristics:

Table 1 Summary Model Configuration

Review	Comments
Simulation filenames	WP_Q100_120min.tcf and WP_18Apr12.tcf
Projection	MGA 94 Zone 56
Domains	1D and 1 x 2D domain
2D grid size	5 metres
TUFLOW build	2012-05-AD-iDP, as part of the review it was recommended to update from the 2011-09 release version to the 2012-05 release, this release included a new wetting / drying algorithm that helps with stability and mass balance for shallow depths such as direct rainfall modelling.
Total number of 2D cells:	763,355
Total number of 1D channels:	1,102

2. 2D Domain

The following section checks the relevant 2D model component.

Table 2 Summary 2D Domain and Topography

Review	Comments
Is the selected 2D grid size appropriate?	The 5 metre grid size is considered appropriate for the intended purpose of identify and mapping the overland flow paths.
Is the selected 2D timestep appropriate?	The 1 second timestep is considered appropriate for a 5m cell size model with direct rainfall application.
Does the DEM accurately represent the surveyed 2D geometry?	Topographic survey was not provided and so this has not been reviewed. If it hasn't already been carried out, recommend comparison of DEM with ground survey (if available), and the original ALS data.
Are the DEM elevations applied correctly in the TUFLOW model?	Yes, the DEM is directly read into the TUFLOW geometry control file. Three breakline layers are inputted to ensure appropriate elevations for key hydraulic controls.
Does the model extent cover the entire catchment?	There is a small section of the upper catchment that is not included in the active model area. However, flow from this area has been accounted for with an inflow boundary in this location. This is considered satisfactory.
Are the Manning's roughness values appropriate?	The hydraulic model uses six land use categories, the adopted roughness values are presented in Table 3 below. For the building land-use category, a depth varying roughness is used. At shallow depths a low roughness is adopted, and at higher depths (>100mm) the roughness is increased. This allows direct rainfall to run-off rapidly, but provides an impediment to flow when a building becomes inundated. This is considered appropriate.
Is the spatial distribution of land-use appropriate?	The spatial distribution of land-use from the check files is presented in Figure 1. The distribution is considered satisfactory.
Any other comments on 2D model?	No.

Table 3 Adopted Manning's n Values

Review	Comments
Pervious (Soil/short grass)	0.03
Impervious (concrete, driveways)	0.02
Roads	0.02
Pervious (Dense vegetation)	0.08
Swimming pools	0.013
Buildings	Depth Varying (0.03 - 1.0)

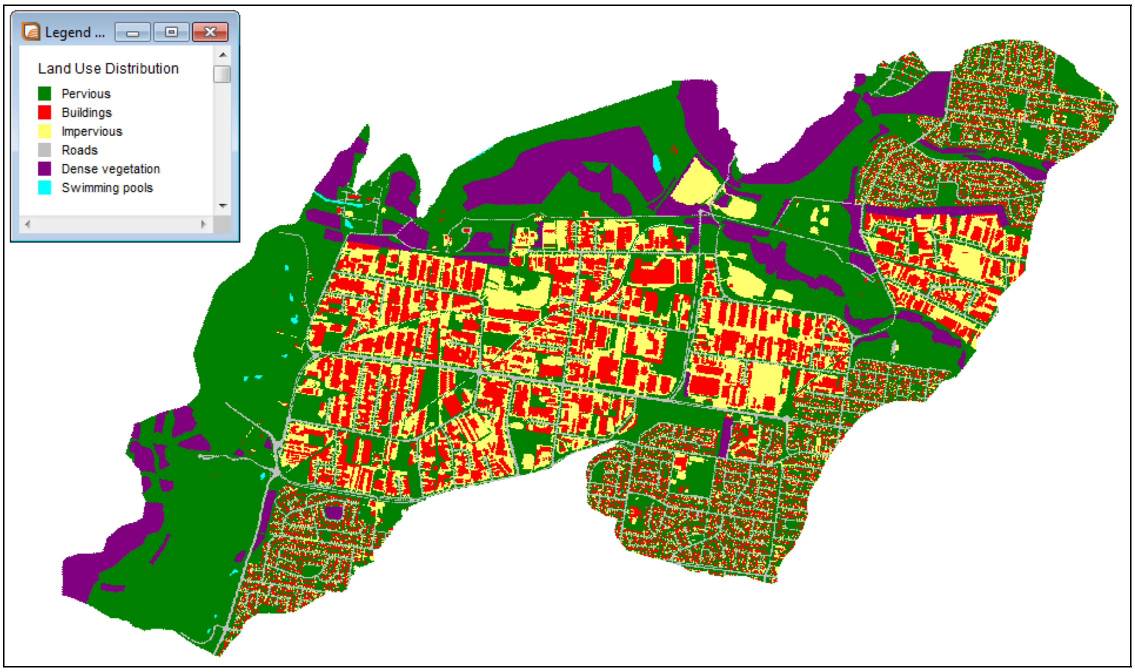

Figure 1 Spatial Distribution of Land-Use Categories

3. 1D Domain

The following section details the key 1D model checks performed during the review.

Table 4 Summary 1D Model Checks

Review	Comments
Is the selected 1D timestep appropriate?	0.10 second (this was recommended to be changed to 0.1 from 0.15 seconds in L.19202.002.01.pdf).
Is the selected 1D resolution appropriate?	The spatial resolution of the channels and cross-sections is considered appropriate.
Selected network connectivity checks	A limited number of connectivity issues were detected and detailed in L.19202.002.01.pdf, these have been rectified and no connectivity issues remain.
Selected pipe gradient checks	No errors were identified.
Selected pipe diameter checks	No errors were identified.
Selected pipe length checks	No errors were identified.
Selected loss coefficient checks	Applied loss coefficients are considered appropriate.
Selected manhole losses checks	Losses are considered appropriate.
Other comments on the 1D	None.

4. 1D/2D Linkages

Following initial review of the 1D/2D linkages, BMT WBM recommended a number of minor changes to improve model stability, these were detailed in L.19202.002.01.pdf and associated GIS files. With these changes incorporated, the 1D/2D linkage is considered satisfactory. Breaklines are incorporated along the top of bank for the lateral open channel boundaries ("HX" type boundaries) which is considered good practice.

Visualisation of the results did not show any circulations anomalies in the 1D/2D linkage behaviour.

5. Model Performance and Mass Balance

5.1. Model Results

Review of the results shows that the model is performing well, with the resultant water levels and flows generally exhibiting a smooth behaviour. There were a limited number of locations where minor oscillations occurred, however these did not occur at the peak water levels and flows and did not impact results at the peak.

The resultant flood behaviour inundates the areas expected flowpaths.

The model simulation time was sufficient to allow the water level to peak throughout the model domain. For future design runs it is recommended that checks are made on the “Time of Peak Water Level” output from TUFLOW, to ensure that the simulation time is sufficient to allow the flood peak to propagate throughout the model area.

The total calculation time with respect to grid size, inflow magnitude and location is considered suitable for use in future analysis. The runtimes were in the order of 12 hours, which is a significant computational cost, however, this is considered suitable for this grid resolution and simulation period and for the stated application.

5.2. Mass Balance

Mass error is an indicator of the solution convergence, and high or continually increasing cumulative mass error usually indicates poor/incorrect input data or poor boundary configuration.

The mass balance for the simulation shows some initial mass error occurring. This is occurring when there is very little volume in the model and therefore a significant percentage mass error is being reported. It is noted that once the flood event starts the mass balance rapidly reduces to within an acceptable range.

A plot of the percentage cumulative mass error (CME) and total volume of water in the model over time is presented below in Figure 2. At time 1.5 hours when the inflow starts, the CME reduces to less than 1%. This is considered within an acceptable range.

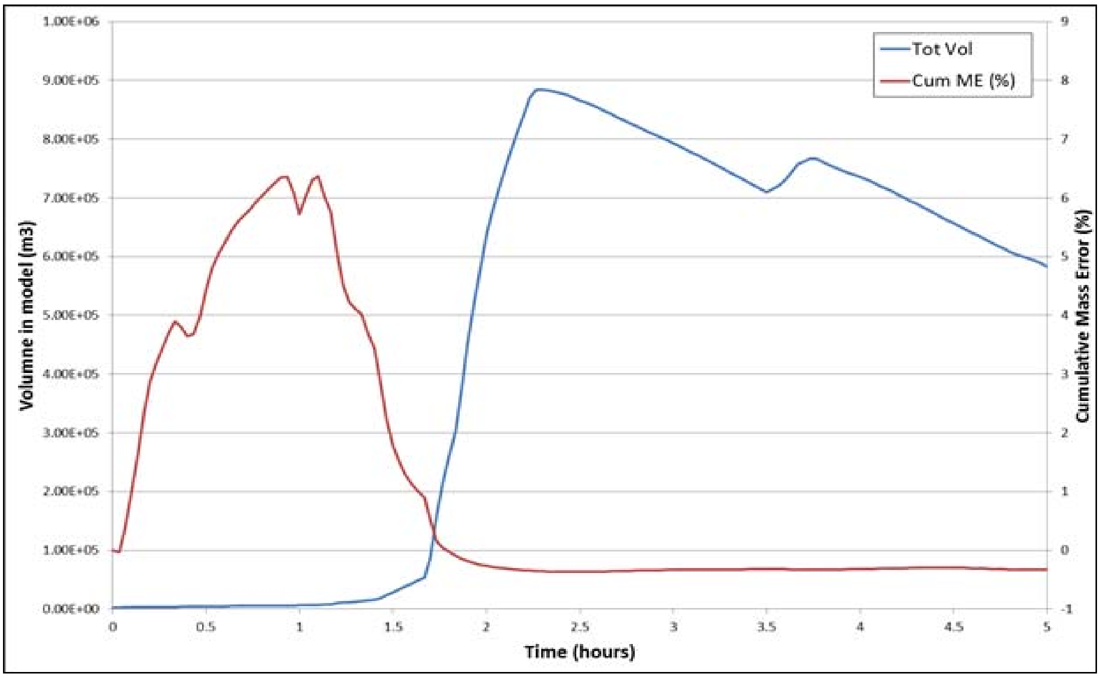

Figure 2 Cumulative Mass Error and Total Volume

6. Structure Representation

The majority of structures were modelled as 1D structures, with a single structure represented as a 2D flow constriction. A summary of the number of structures is outlined in Table 5.

Table 5 Summary of Structures

Structure Method	Number of
2D Flow Constriction	1
1D Bridges ("B" Type)	8
1D Weirs	3
1D Culverts	628

The loss parameters on the 2D structure are considered within the normal range. The losses at the 1D bridge structures are consistent with clear spanning bridges, with additional losses introduced when the bridge deck becomes submerged. This is considered an acceptable approach.

Loss parameters on the culverts are considered suitable.

Review of the results show the hydraulic structures are considered to be performing satisfactorily.

6.1. Blockages

For both the calibration and design event, there are no blockages modelled on the structures. As part of the design modelling and mapping, it is recommended that sensitivity analysis to structure blockage is considered.

7. Calibration

The calibration levels are provided in Table 6 below. In general these show an acceptable calibration to the recorded data. With all modelled levels except the Victoria St/Newton Rd outlet being within 300mm of the observed levels.

At the Victoria St/Newton Road the modelled velocity is very high (greater than 5m/s). This represents a velocity head ($v^2/2g$) in excess of 1m. At the edges of the channel, where the velocity is lower, the water surface could be high by over 1m. Care should be taken when comparing recorded water levels in such high velocity locations.

Table 6 Calibration Levels

Location	High Water Level		
	Observed (m AHD)	**Modelled (m AHD)**	**Difference (m)**
Main Channel			
MC_Cow/THD inlet	57.07	57.02	-0.05
MC_Cow/THD roundabout	57.73	57.71	-0.02
Toohey Road inlet	51.52	51.27	-0.25
Hallstrom Place outlet	48.25	48.07	-0.18
Victoria St/Newton Rd outlet	41.34	40.67	-0.67
Elizabeth Street outlet	35.86	35.62	-0.24
MC_34 Davis Rd	32.68	32.61	-0.08
Tributary 1			
Newton Road inlet	42.35	42.31	-0.04
Rosford Street			
Hassall Street inlet	23.23	26.42	0.19
Hassall Street outlet	25.80	25.93	0.13
RC_Hassall St outlet	25.80	25.87	0.07
Prospect Creek			
RC_Gipps Rd outlet	24.67	24.56	-0.11
PC_Rosford Res outlet	22.53	22.66	0.12
Other Locations			
Emerson St res	37.64	37.74	0.10
RC_3 Shakespeare St	38.27	38.46	0.19

8. Conclusion

Following the adoption of the recommended model changes provided in “L.19202.002.01.pdf”it can be concluded from the technical review that:

- The model is adequately calibrated to the 2012 rainfall event
- The model is healthy and has acceptable model convergence
- The model is suitable for use in identifying and mapping overland flooding within the Wetherill Park catchment.

Yours Faithfully
BMT WBM Pty Ltd

Phillip Ryan
Senior Flood Engineer

Appendix H Choice of Flood Model and Modelling Methodology

1.0 Background

The report "Fairfield City Overland Flood Study" (SKM/FCS, 2004) identified and ranked 18 urban catchments within the Local Government Authority (LGA) for detailed investigation of overland flooding. Flood Studies for six of these catchments have already been undertaken jointly by Sinclair Knight Merz (SKM) and Fairfield Consulting Services (FCS) and are currently at various stages of completion.

Wetherill Park is the next catchment planned for detailed investigation and it is the intention that the Overland Flood Study, Floodplain Risk Management Study and Plan will be undertaken internally by Natural Resources staff.

In preparation for this, the available modelling packages and methodologies were considered and the merits of each assessed. It is intended that this will help establish the methodology for subsequent investigations for the remainder of catchments in the LGA.

2.0 Overview of Modelling Methodologies

In order to assess the merits of each modelling package and modelling approach, a review of available literature was undertaken (see Section 6.0 for details). Informal discussions were also held with the various model developers, as well as with Consultants and other LGAs having previous experience with similar studies.

A summary and brief description of each methodology is provided, along with the catchment to which that methodology has already been applied. For further details of the methodology, the reader is referred to the relevant flood study.

<u>(1) DRAINS/TUFLOW Method</u>

Applied in the Canley Corridor and Fairfield Overland Flood Study

DRAINS model represents:

- catchment hydrology (runoff-routing)
- pipes/pits and open channels (unsteady 1D flow)

TUFLOW model represents:

- pipes/pits[1] and open channels (unsteady 1D flow)
- overland flow (unsteady 2D flow)

1 – Note: the Canley Corridor Study did not include pipes and pits in the TUFLOW model

(2) XP-STORM Direct Rain Method

Applied in the Old Guildford and Smithfield Overland Flood Studies

XP-STORM model represents:

- catchment hydrology (Direct Rain)
- pipes/pits and open channels (unsteady 1D flow)
- overland flow (unsteady 2D flow)

(3) TUFLOW Direct Rain Method

Applied in the Rural Area Flood Study (Ropes, Reedy & Eastern Creek)

TUFLOW model represents:

- catchment hydrology (Direct Rain)
- pipes/pits and open channels (unsteady 1D flow)
- overland flow (unsteady 2D flow)

The DRAINS/TUFLOW Method represents the traditional approach, where the hydrologic and hydraulic components are analysed using separate models. The DRAINS model is used to represent the catchment hydrology, as well as the pipes and pits and the open channels in the system. Runoff generated in the pit sub-catchments are input to the TUFLOW model as inflow boundary conditions. The TUFLOW model represents the pipes and pits and the open channels in its 1D component, the overland flow in its 2D component and allows full dynamic linking between them.

Similarly, the XP-STORM and TUFLOW Direct Rain Methods also allow full dynamic linking between the 1D and 2D flow components. However, instead of using an independent hydrologic model, these methods utilise the Direct Rain Method to calculate inflow hydrographs. The Direct Rain Method is becoming increasingly popular for urban catchments where the hydraulic model covers all or a significant portion of the catchment. Further details are provided in the following section.

3.0 The Direct Rain Method

The Direct Rain Method was investigated in detail to determine whether it can be adopted for use in the flood modelling of the Wetherill Park catchment and for subsequent catchments in the Fairfield City Council LGA.

In the Direct Rain Method, the model effectively considers each cell of the 2D grid as a subcatchment and calculates runoff based on user-specified rainfall inputs (e.g. a rainfall hyetograph, Initial Loss (IL), Continuing Loss (CL) etc.) as per a conventional hydrologic model. This approach means that the catchment hydrology, underground pipes and overland flow can all be represented in a single model, rather than using separate models for the hydrology and the hydraulics.

The obvious benefit of this approach is the reduction in setup time of an independent hydrological model. It also allows for a more orderly file structure and eliminates the chance of human error in transferring output files from the hydrological model to be input to the hydraulic model.

The Direct Rain Method also has a significant technical advantage over traditional hydrologic models in that it removes the need for catchment delineation by the model user. Hence there is no longer the requirement to make assumptions of the locations of catchment outlets (which are not always obvious, particularly when analysing flat areas) and which catchment applies to a particular pit. In the Direct Rain Method, flows are automatically routed to the appropriate pit based on hydraulic principles, not judgement.

Furthermore, additional cross catchment flows may occur in larger events. The Direct Rain Method also overcomes this issue, as the model will automatically divert flood waters along different flowpaths during high flow events.

It was therefore concluded that the Direct Rain Method can be readily applied to model overland flooding in any catchment in the LGA. However, as with any form of numerical modelling, the limitations of the method must be clearly understood and the model properly calibrated and verified to ensure that appropriate parameters are used.

Unfortunately, calibration of overland flow models of urbanised catchments is seldom possible due to unavailability of suitable measured data. However, verification can and should be undertaken.

Verification will involve the set up of independent hydrologic models of a few selected subcatchments within the study area. (To set up an independent hydrologic model of the entire catchment would be very time-consuming and to some extent defeat the purpose of using the Direct Rain Method.)

When setting up the independent hydrologic models for the verification process, it is recommended that, as far as practically possible, the selected subcatchments satisfy the following conditions:

- a range of different land uses are included;
- a range of different terrain slopes are included;
- each can be considered in isolation with limited influence from neighbouring subcatchments. Subcatchments located in the upper reaches of the study area are more likely to meet this requirement;
- few or no hydraulic structures/controls such as culverts and bridges are included. This will allow a realistic comparison between hydrologic and hydraulic models.

The hydrographs generated by the independent hydrologic model set up for each subcatchment should be compared to those calculated by the hydraulic model for the corresponding subcatchment. (Note that when considering flows from the hydraulic model, both the underground pipe flow and the overland flow component should be included.) The key outputs to be compared should be the peak flow, the volume of runoff, timing of the peak flow and the overall shape of the hydrograph.

It is not always expected that the two models will match – in fact, two separate traditional hydrological models with similar parameters can yield significantly different results. However, where there are differences, some interpretation of the results can be made, and the model can be checked as to why this is the case. Engineering judgement should be employed to determine appropriate adjustment of model parameters. Conventional values of hydrologic modelling parameters such as Initial Loss and Continuing Loss may not necessarily be applicable to a corresponding model using the Direct Rain model.

4.0 Choice of 2D Model

The TUFLOW and XP-STORM models were investigated to determine which would be more suitable for use in the flood modelling of the Wetherill Park catchment and for subsequent catchments in the Fairfield City Council LGA.

Since the 2D component of XP-STORM is based on the TUFLOW engine, it is expected that the use of either model will yield only very minor differences in results or model simulation time. It was noted that XP-STORM has its own interface for the TUFLOW engine and subsequently does not require MapInfo, so it could prove easier to set up and manipulate a model.

However during the application of XP-STORM in the Old Guildford Overland Flood Study, a number of issues with the model arose, including:

- The model was unable to read certain data entered as GIS layers (such as Manning's *n* roughness), requiring the data to be input manually;
- The model became unstable in certain pipe sections, particularly at low flows near the end of the simulation;
- The model was unable to perform batch runs;
- The model was unable to perform multiple runs concurrently.

The issues outlined above generally do not apply to TUFLOW. Furthermore due to memory constraints, the flood model for the Old Guildford catchment could not use a grid size smaller than 2.5m. By contrast, the TUFLOW model was capable of using a 2m grid for the comparable Fairfield catchment, thus allowing the user greater flexibility in choice of model grid size.

Some investigation was carried out on the 1D component of TUFLOW, since this will be used to represent the pipes and open drains of the catchment.

The 1D component of TUFLOW dates back to 1971 as the 1D model ESTRY. Bill Syme wrote the 2D component in 1989 as part of his Master's Thesis and linked it to the 1D model, thus creating the model TUFLOW.

Both the free surface and pressurised/pipe flow components of the original ESTRY model have been extensively tested and verified and also perform well in terms of convergence and stability.

TUFLOW's Direct Rain Method is currently being applied in the Rural Area Flood Study (Ropes, Reedy and Eastern Creek). This project is still in progress, but preliminary results to date have been encouraging. In this case however, the method has been applied to a main creek flood study in a largely rural catchment.

Of more direct relevance is the study recently undertaken by Bankstown City Council in which TUFLOW's Direct Rain Method was used to model overland flooding in a number of urbanised catchments. The feedback received was that this approach gave excellent results and also that BMT-WBM, who market and support the TUFLOW model, provided outstanding service and support.

5.0 Conclusions and Recommendations

It has been demonstrated that when properly applied and verified, the Direct Rain Method can be used to provide input flows for 1D/2D models of urbanised catchments. This approach eliminates the need to set up an independent hydrologic model of the catchment, and allows the catchment hydrology, underground pipes and overland flow to be represented in a single model, rather than using separate models for the hydrology and the hydraulics.

The method also offers notable technical advantages, such as the allocation of subcatchment runoff to certain pits or open channels based on hydraulic principles, rather than judgement of catchment delineation. The Direct Rain Method also allows for proper facilitation of cross catchment flows.

The Direct Rain Method should be properly verified, via the modelling of a few selected subcatchments using an independent hydrologic model such as XP-RAFTS or WBNM. The flows generated by the independent hydrologic model should be compared with those of the hydraulic model and where there are differences, some interpretation of the results can be made. The model can be checked as to why this is the case, and, if deemed necessary, adjustments made to the model input parameters.

The XP-STORM model has already been applied in the Old Guildford Overland Flood Study. A number of issues were encountered, particularly regarding the model's capability of reading certain input data, as well as model stability, minimum grid size and the capability of performing batch runs or multiple runs concurrently.

The TUFLOW Direct Rain Method has yet to be applied to any overland flood study of an urbanised catchment in the Fairfield City Council LGA to date. However Bankstown City Council have applied this method to a number of urbanised catchments within their LGA. The feedback received was that Bankstown City Council were very satisfied with the results obtained using this method, as well as the service provided by BMT-WBM, the software supporter.

Furthermore, the technical staff of Fairfield City Council have considerably more familiarity and experience with the TUFLOW package, and to date have had little or no opportunity to use XP-STORM for project work.

Therefore, it is the recommendation of this study that the *TUFLOW Direct Rain Method* be used for the Wetherill Park Overland Flood Study.

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Appendix I Hydraulic Model Update Summary Report (Cardno 2015)

Hydraulic Model Update Summary Report

Wetherill Park Catchment Management Plan, Stage 1

59914102

Prepared for
Fairfield City Council

8 January 2015

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Document Information

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Document History

Version	Effective Date	Description of Revision	Prepared by:	Reviewed by:
V2	8 Jan 15	Final Report	Kieran Geraghty	David Whyte
V1	28 Oct 14	Draft Report	Kieran Geraghty	David Whyte

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Appendices

Tables

Figures

1 Introduction

1.1 Background

Cardno have been engaged by Fairfield City Council to undertake the first stage of the Catchment Management Plan for the Wetherill Park catchment.

A Draft Overland Flood Study was completed by Fairfield City Council in 2013 to identify flood behaviour within the study area under existing conditions. As part of this, a combined 1D/2D TUFLOW hydraulic model was developed using the direct rainfall methodology. This model will be used to assess the effectiveness of flood mitigation options as part of the Catchment Management Plan.

This report outlines a review of the existing hydraulic model, identifies recommended updates and documents the results and differences of these updates. Flood mapping undertaken as part of the Overland Flood Study (2013) has been reviewed due to the model amendments and updated flood maps are attached to this report.

1.2 Completed Assessments

A number of assessments have been completed prior to this report, including the following:

- Hydraulic Model Review (March 2014);
- Probable Maximum Precipitation Calculation Review (June 2014); and
- Model Updates and Preliminary Results (July 2014).

1.2.1 Hydraulic Model Review

A detailed review of the existing hydraulic model was undertaken at the commencement of the project and following a site inspection. A number of issues were identified for further investigation including pipe network connections, 1D channel schematisation, culvert and bridge modelling methodology and 1D/2D connections. During the initial site visit it was evident that a large noise wall exists along Cowpasture Road which was not included in the existing model but had the potential to alter overland flow.

As a result of this review, it was recommended that a number of refinements be made to the existing hydraulic model. The updated model was assessed for the 100yr Average Interval Recurrence (ARI) event to identify any differences in flood behaviour. Further details on the hydraulic model review and recommendations are included in Appendix A.

1.2.2 Probable Maximum Precipitation Calculation Review

A review of the methodology used to calculate the Probable Maximum Precipitation (PMP) for the study area was undertaken. The PMP is used with spatial and temporal distributions to estimate the Probable Maximum Flood (PMF).

A comparison of rainfall intensities and distribution adopted in the Overland Flood Study (2013) to those identified as part of the review indicated general consistency between both. The parameters derived as part of the review have been used in the hydraulic model update for assessment of the PMF event. Further details on the PMP calculation review are included in Appendix B.

1.2.3 Model Updates and Preliminary Results

The recommendations developed as part of the hydraulic model review were undertaken for the 100yr ARI event and the critical storm duration of 2hrs. The resulting differences in flood behaviour were identified, mapped and discussed. The main differences related to the impacts of the noise wall on overland flow, combined with updated channel schematisation, leading to a reduction in peak flood levels within the main channel.

Further details on the model updates and preliminary results are included in Appendix C.

2 Model Results

2.1 Model Validation to April 2012 Storm Event

The hydraulic model developed as part of the Overland Flood Study (2013) was calibrated to the storm event of 18 April 2012. Debris marks were recorded and surveyed by Council staff following the event and the updated model was run to validate the results to those of the Overland Flood Study.

No rainfall gauges exist within the study area with three gauges located in close proximity including;

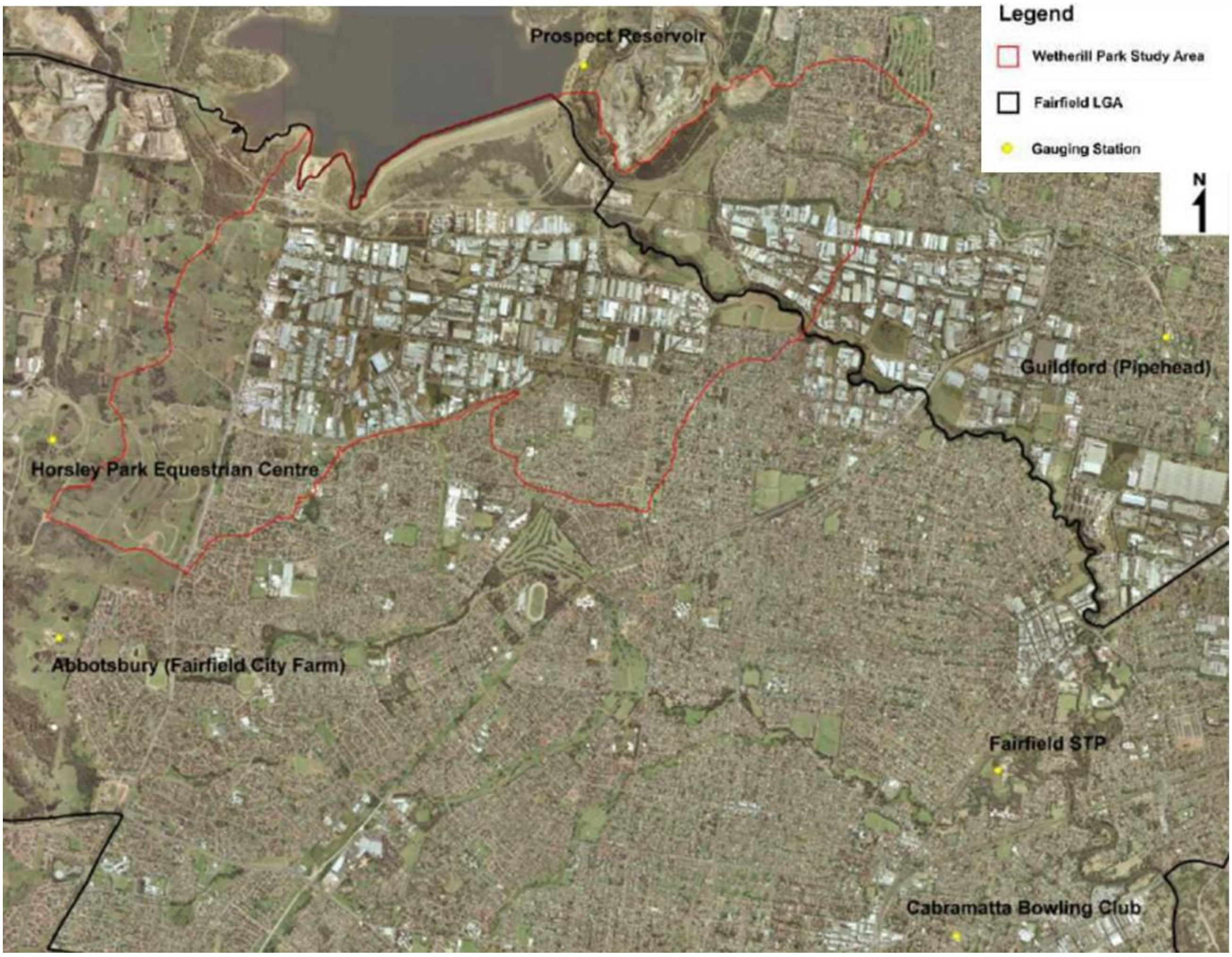
> Abbotsbury, Fairfield City Farm (GS567169);

> Horsley Park Equestrian Centre (GS067119); and

> Prospect Reservoir (GS567083).

The location of the rainfall gauges in relation to the study area are shown in Figure 2-1.

Figure 2-1 Location of Rainfall Gauges



Source: Wetherill Park Overland Flood Study, Fairfield City Council (2013)

The updated hydraulic model was run using the recorded rainfall information from each of the three individual gauges and also using the average of the recorded rainfall from these three gauges.

Results based on average rainfall from all three gauges are outlined in Table 2-1.

Table 2-1 Model Validation Results – Average Recorded Rainfall

	Observed (m AHD)	Flood Study Model Results (2013) (m AHD)	Difference (m)	Updated Flood Study Model Results (2014) (m AHD)	Difference (m)
Main Channel					
Cowpasture Rd/The Horsley Drive (culvert inlet)	57.07	57.18	0.11	57.22	0.15
Cowpasture Rd/The Horsley Drive (on roundabout)	57.73	57.71	-0.02	57.76	0.03
Tooheys Rd (culvert inlet)	51.52	51.43	-0.09	51.26	-0.26
Hallstrom Place (bridge outlet)	48.25	48.15	-0.10	48.00	-0.25
Newton Rd/Victoria Street (culvert outlet)	41.34	40.76	-0.58	40.62	-0.72
Elizabeth St (bridge outlet)	35.86	35.70	-0.16	35.56	-0.30
Snow Confectionary Pty Ltd (34 Davis Road)	32.68	32.66	-0.02	32.60	-0.08
Tributary 1					
Newton Rd (culvert inlet)	42.35	42.37	0.02	42.30	-0.05
Rosford Street					
Hassall St (culvert inlet)	26.23	26.46	0.23	26.42	0.19
Hassall St (culvert outlet)	25.80	25.89	0.09	25.87	0.07
Prospect Creek					
Gipps Rd (culvert outlet)	24.67	24.60	-0.07	24.66	-0.01
Rosford Reserve (outlet)	22.53	22.71	0.18	22.64	0.11
Other Locations					
Emerson Street Reserve (in basin)	37.64	37.79	0.15	37.71	0.07

Using average rainfall values results in similar correlation to that used in the Overland Flood Study (2013) with the majority of calibration locations being within ±0.2m of recorded water levels. Results show considerable differences between modelled and recorded water levels at the Newton Road and Victoria Street culvert where very high velocities are experienced. The Overland Flood Study (2013), and peer review comments, state peak water levels may be under estimated at high velocity locations due to variability in velocity calculation in the 1D culvert and channel network.

Updated model results based on data from individual rainfall gauges is outlined in Table 2-2 and compares results with the observed water levels.

Table 2-2 Model Validation Results - Individual Rainfall Gauges

	Observed (m AHD)	Updated Flood Study Model Results (2014) (m AHD)			Difference (m)		
		Abbotsbury Gauge	Horsley Park Gauge	Prospect Reservoir Gauge	Abbotsbury Gauge	Horsley Park Gauge	Prospect Reservoir Gauge
Main Channel							
Cowpasture Rd/The Horsley Drive (culvert inlet)	57.07	57.11	57.42	57.11	0.04	**0.35**	0.04
Cowpasture Rd/The Horsley Drive (on roundabout)	57.73	57.75	57.78	57.75	0.02	**0.05**	0.02
Tooheys Rd (culvert inlet)	51.52	51.06	51.50	51.06	-0.46	**-0.02**	-0.46
Hallstrom Place (bridge outlet)	48.25	47.90	48.13	47.90	-0.35	**-0.12**	-0.35
Newton Rd/Victoria Street (culvert outlet)	41.34	40.54	40.89	40.53	-0.80	**-0.45**	-0.81
Elizabeth St (bridge outlet)	35.86	35.48	35.79	35.47	-0.38	**-0.07**	-0.39
Snow Confectionary Pty Ltd (34 Davis Road)	32.68	32.58	32.72	32.56	-0.10	**0.04**	-0.12
Tributary 1							
Newton Rd (culvert inlet)	42.35	42.18	42.53	42.22	-0.17	0.18	**-0.13**
Rosford Street							
Hassall St (culvert inlet)	26.23	26.37	26.54	26.32	0.14	0.31	**0.09**
Hassall St (culvert outlet)	25.80	25.84	25.95	25.81	0.04	0.15	**0.01**
Prospect Creek							
Gipps Rd (culvert outlet)	24.67	24.65	24.71	24.60	-0.02	0.04	**-0.07**
Rosford Reserve (outlet)	22.53	22.63	22.68	22.54	0.10	0.15	**0.01**
Other Locations							
Emerson Street Reserve (in basin)	37.64	37.66	37.84	37.59	0.02	0.20	**-0.05**

Results show variability between modelled results using the individual rainfall gauges data and observed flood marks with significant differences evident along the main channel at Tooheys Road culvert inlet (up to 0.46m), Newton Road and Victoria Street culvert outlet (up to 0.81m) and Elizabeth Street bridge outlet (up to 0.39m) while a much better correlation (±0.15m) is seen at other locations.

The individual gauge results shows that a generally improved correlation is seen along the main channel using the Horsley Park gauge rainfall while for Tributary 1, Rosford Street, Prospect Creek and Emerson Street basin, the Prospect Reservoir gauge rainfall provides a better correlation. These results show the majority of calibration locations being within ±0.15m of recorded water levels.

2.2 Critical Storm Duration

Flood behaviour was modelled for the 20yr, 100yr, 500yr, 10,000yr ARI and PMF events for the rainfall durations outlined in Table 2-3.

Table 2-3 Hydraulic Model Scenarios

ARI / Duration	15 min	25 mins	30 mins	45 mins	1hr	90 mins	2hr	3hr	4hr	4.5hr	5hr	6hr	9hr
20yr		*	*	*	*	*	*	*		*		*	*
100yr		*	*	*	*	*	*	*		*		*	*
500yr		*	*	*	*	*	*	*		*		*	
10,000yr		*	*	*	*	*	*	*				*	
PMF	*		*	*	*	*	*	*	*		*	*	

The critical storm duration for peak flood levels within the study area vary depending on the location and flood characteristics for specific locations. Results indicated the 2hr storm duration was critical in the majority of the catchment. Peak water levels were identified throughout the study area based on the maximum of each storm duration for each ARI event.

The critical storm duration in the study area for the 20yr, 100yr ARI and PMF events are shown in Figure D1, Figure D2 and Figure D3 in Appendix D.

2.3 Flood Extents, Depths and Velocities

The expected flood behaviour within the study area is outlined in Appendix D including the following:

> 20yr, 100yr ARI and PMF peak flood depths are shown in Figure D4 to Figure D15;

> 100yr ARI flood extents and flood level contours are shown in Figure D16 to Figure D19;

> 100yr ARI and PMF peak flood velocities are shown in Figure D20 to Figure D27; and

> Flood profiles along Main Channel, Tributary 1 and Rosford Channel are shown in Figure D28, Figure D29 and Figure D30 respectively.

2.4 Flood Risk Precinct Mapping

Flood risk precinct mapping was prepared based on the interpretation of the 100yr ARI and PMF peak depth and velocity grids and as per outlined in the Fairfield City Councils Development Control Plan (DCP) which are presented in Table 2-4.

Table 2-4 Flood Rick Precinct Characteristics (DCP, 2013)

Risk Precinct	Description
High	The area of the floodplain below the 100yr ARI flood extent that is either subject to high hydraulic hazard or where significant evacuation difficulties exist
Medium	The area of the floodplain below the 100yr ARI flood extent that is not in a High flood risk precinct i.e. land that is not subject to high hydraulic hazard or where no significant evacuation difficulties exist
Low	All other land within PMF extents that is not defined as High or Medium flood risk precincts

Provisional Flood Risk Precincts have been identified within the study area and are outlined in Figure D31 to Figure D34 in Appendix D.

3 Conclusion

The existing TUFLOW hydraulic model developed as part of the Wetherill Park Overland Flood Study (2013) has been updated following detailed review of the modelling methodology and parameters.

Results of the updated model were validated to those from the existing model, based on the April 2012 calibration event results. In general, results were within ±0.2m at calibration locations indicating reasonable correlation.

The updated hydraulic model was used to reassess and remap existing flood behaviour within the study area for the 20yr, 100yr, 500yr, 10,000yr ARIs and PMF events. Peak water levels, depths and velocities have been determined for each event and provisional flood risk precinct mapping completed for the study area.

The outcomes of this hydraulic model review will form the basis for assessment of flood mitigation options for the Wetherill Park catchment.

Wetherill Park Catchment Management Plan, Stage 1

APPENDIX

A

HYDRAULIC MODEL REVIEW

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Memorandum

Attention	Sean Howie
Organisation	Fairfield City Council

Sent by	Kieran Geraghty	**Date:** 7 March 2014
Subject	Wetherill Park Catchment Management Plan, Stage 1 - Review of Overland Flood Study Hydraulic Model	
File No	59914102_L02	

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Cardno have been commissioned by Fairfield City Council to undertake the first stage of the Catchment Management Plan for the Wetherill Park catchment. This memorandum outlines a review of the Tuflow hydraulic model developed as part of the Overland Flood Study. This model will form the basis for assessment of flood mitigation options as part of this Study

Comments on various aspects of the model are outlined in Table 1-1

Table 1-1 Model Review Comments

Parameter	Comment
Pipe network	The system appears to be well defined, however in some locations there is inconsistency as to pipe connections (for example, there are 4 x 1.8m dia. pipes going into 2 x 1.8m dia. pipes). This may be the reality, but will need to be confirmed with available information.
Channel network	In some locations cross sections do not appear to have been picked up correctly. This causes issues as the inverts are being set by the cross sections. There is a large channel represented in 2d upstream of The Horsley Drive. It is likely that in the current model the volume of water entering the site is larger than if this channel was defined in 1D. This would impact downstream flood levels. There are some very long connections within the model – this will cause some variation to where flow should be entering the system compared to where it is getting put into the system. While it is unlikely to make significant differences, it may overestimate flows in some areas and underestimate it in others. At the confluence of the main channel and Tributary 1, west of Elizabeth Street there is double counting of the channel flow area and volume, this is likely pushing potential flood levels higher than would otherwise occur. Bridges have only been defined up to the soffit level. This is not an issue unless the bridge overtops, at this point instabilities will occur as flow goes over the 2D and drops back into the 1D channel, as there is no calculation point within this range (2d > 1d definition) the model will become unstable.
Connections	In general the connections are satisfactory, however in some locations the nulling of the 1D is incomplete. This may cause instabilities as there are connections adjacent to each other, this results in flow leaving one connection and occasionally getting drawn straight back in, which can initiate instabilities. There are several pits which do not have connections to the 2D, this may in fact be accurate however there are several pipes at the upstream of reaches where this is the case.
2D definition	In some locations, at the downstream end of the 1D channels the 2D is higher than the 1D. The reason is most likely due to a noisy 2D surface, however the 1D will back up to reach this level prior to being able to freely discharge. This artificially raises the water levels and decreases channel capacity. As this occurs at the downstream areas of the model it may not be critical to the areas of interest for this Study. At locations where bridges are defined, the upstream and downstream levels do not always match. This results in the bridge not acting as effectively as it could which may

Parameter	Comment
	impact surrounding flood levels
Roughness	The roughness layer covers the full model extent, however the grid is on a rotation. As a result the building roughness extents join together in places. The building roughness is depth varying with a very low roughness below 0.1m and then full blockage above this. This approach is in line with other flood modelling projects undertaken for Council.

It is noted that the above commentary may not be critical in all parts of the study area and where proposed flood mitigation options will be focused. In most cases, model stability would improve which would result in a smoother running model but may not overly affect existing model results.

Site Visit Observations

During the site visit in February 2014, a number of issues were identified which may impact the current model results, namely:

> The presence of a noise wall along the eastern side of Cowpasture Road (south of Bossley Road) as shown below has the potential to alter the flood behaviour by blocking the overland flow path. This noise wall was not included in the model; and

> The modelling methodology used to assess the culverts and the 1D channel near the intersection of Victoria Street and Newton Road. It is unclear how well defined this is within the current model and it was suggested that a HEC-RAS model be developed to confirm the hydraulics in this area, given it is a known flooding trouble spot.

Noise wall on Cowpasture Road and culverts at intersection of Victoria St and Newton Rd

Recommendations

The following recommendations are made following detailed review of the hydraulic model:

> Confirmation of the pipe network where inconsistencies are evident, or complex pipe networks exist, to confirm the model is reflective;

> Review open channels to ensure flows connect at the right location and that in confluence areas capacity is not double counted;

> 1D/2D connections should be reviewed including null areas;

> Confirm the 2D surface is representative including the upper catchment in Western Sydney Parklands and interaction with the Sydney Water Supply Channel which would influence flows to developed areas downstream;

> Where all flow is being transferred between 1d and 2d realistic levels should be used to ensure accurate flood levels;

> Bridge definitions should be extended (if higher flow events are expected) to ensure that the profile is not exceeded as this leads to instabilities; and

> Update model to include the noise wall along Cowpasture Road, undertake model checks and comparison with HEC-RAS for the channel and culverts at intersection of Newton Road and Victoria Street.

Way Forward

The majority of the above recommendations would likely lead to improved model stability but may not overly change the existing results. The following suggestions are made:

> In the first instance, proceed with these recommendations and see what differences occur in the model results for say the 100yr Average Recurrence Interval (ARI) event. If no significant differences are apparent, then the model would be used to assess the flood mitigation options as part of this study;

> Alternatively, should these recommendations result in significant differences then the full range of ARI events could be rerun and mapping revised for the Overland Flood Study.

APPENDIX

B

PMP CALCULATION REVIEW

Memorandum

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Attention	Sean Howie
Organisation	Fairfield City Council
Sent by	Kieran Geraghty **Date:** 27 June 2014
Subject	Wetherill Park Catchment Management Plan, Stage 1 Review of Probable Maximum Precipitation Calculation
File No	59914102_L03

This memorandum outlines the calculation of the Probable Maximum Precipitation (PMP) for the study area and comparison to the values adopted in the Overland Flood Study (2013).

The PMP is used with spatial and temporal distributions to calculate the Probable Maximum Flood (PMF). The PMP has been estimated using the publication "*The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method*" (Commonwealth Bureau of Meteorology, 2003).

Table 1-1 outlines the data for the PMP calculation with calculated rainfall intensities outlined in Table 1-2. For the Wetherill Park catchment a weighted average of three ellipses based on the enclosed areas of A, B and C were applied. A plot which shows the catchment area along with the PMP ellipses is shown in Figure 1-1.

Table 1-1 PMP Calculation Values

Parameter	Value
Total area (km^2)	19.15
Moisture Adjustment Factor	0.70
Elevation Adjustment Factor	1.00
Percentage Smooth	100%
Area Within Spatial Distribution Ellipse A (km^2)	2.6
Area Within Spatial Distribution Ellipse B (km^2)	9.85
Area Within Spatial Distribution Ellipse C (km^2)	6.70

Table 1-2 Rainfall Intensities (mm/hr)

	15 mins	30 mins	45 mins	1 hr	90 mins	2 hrs	3 hrs
Depth (mm)	140	210	260	310	360	400	440
Intensity (mm/hr)	560	420	347	310	240	200	147

A comparison of rainfall Intensities adopted in the existing Overland Flood Study to those determined as part of this assessment is shown in Table 1-3.

Figure 1-1 PMP Ellipses

Table 1-3 Rainfall Intensity Comparison

Duration	Updated		Existing Overland Flood Study	
	Depth (mm)	Intensity (mm/hr)	Depth (mm)	Intensity (mm/hr)
15 Mins	140	560	140	560
30 Mins	210	420	200	400
45 Mins	260	347	260	347
60 Mins	310	310	300	300
90 Mins	360	240	340	227
2 Hours	400	200	380	190
3 Hours	440	147	430	143

In general, the comparison indicates the intensities adopted in the Overland Flood Study are consistent with the current assessment. The Overland Flood Study report indicates that the critical duration in a PMF event is 45 minutes and Table 1-3 indicates identical rainfall intensities were determined as part of this assessment.

The revised rainfall distribution is calculated based on the temporal distribution for short duration PMP (Table1, BoM 2003). The temporal distribution pattern for short term PMP is shown in Table 1-4.

Table 1-4 Temporal Distribution for Short Duration PMP

% of time	% of PMP
5	0.04
10	0.06
15	0.08
20	0.07
25	0.07
30	0.07
35	0.07
40	0.06
45	0.07
50	0.05
55	0.06
60	0.05
65	0.05
70	0.05
75	0.04
80	0.03
85	0.03
90	0.02
95	0.02
100	0.01

A comparison of the revised rainfall distribution to that adopted in the existing Overland Flood Study Model is outlined in Table 1-5.

Table 1-5 Rainfall Distribution Comparison

Depth	Time (hrs)	140	140	210	200	260	260	310	300	360	340
Intensity		560	560	420	400	347	347	310	300	240	227
Duration		15min		30 min		45 min		60 min		90 min	
0	0	0	0	0	0	0	0	0	0	0	0
1	0.083	61.6	61.13	42	40.67	31.2	30.62	24.8	24	18	15.87
2	0.166	53.2	53.2	50.4	46.67	44.2	42.47	37.2	37	25.2	24.18
3	0.25	25.2	25.67	42	40.67	39	40.44	37.2	35	28.8	29.09
4	0.333			37.8	35.33	36.4	37.84	37.2	35	32.4	26.44
5	0.416			25.2	24.67	33.8	32.07	34.1	32	28.8	26.44
6	0.5			12.6	12	28.6	28.89	31	29	25.2	26.44
7	0.583					23.4	23.4	27.9	28	25.2	23.8
8	0.666					15.6	15.31	24.8	25	25.2	25.69

Depth	Time (hrs)	140	140	210	200	260	260	310	300	360	340
Intensity		560	560	420	400	347	347	310	300	240	227
Duration		15min		30 min		45 min		60 min		90 min	
9	0.75					7.8	8.96	21.7	22	21.6	19.64
10	0.833							15.5	15	21.6	22.29
11	0.916							12.4	11	21.6	18.89
12	1							6.2	7	21.6	18.89
13	1.083									18	17.38
14	1.166									14.4	13.22
15	1.25									10.8	11.33
16	1.333									10.8	8.69
17	1.416									7.2	7.56
18	1.5									3.6	4.16

Highlighted values determined as part of this assessment.

The parameters calculated as part of this assessment are generally consistent with those adopted as part of the Overland Flood Study (2013) with the values of rainfall depth and intensity for the critical storm duration of 45mins matching exactly.

Relatively minor differences are highlighted between the parameters for other storm durations and are not expected to significantly impact PMF flows.

Wetherill Park Catchment
Management Plan, Stage 1

APPENDIX

C

PRELIMINARY RESULTS

Memorandum

Attention	Nona Ruddell Sean Howie		
Organisation	Fairfield City Council		
Sent by	Kieran Geraghty	**Date:**	14 July 2014
Subject	Wetherill Park Catchment Management Plan, Stage 1 - Hydraulic Model Update Preliminary Results		
File No	59914102_L04		

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Cardno have been commissioned by Fairfield City Council to undertake the first stage of the Catchment Management Plan for the Wetherill Park catchment. This memorandum outlines updates made to the Tuflow hydraulic model developed as part of the Overland Flood Study (2013). This model will form the basis for assessment of flood mitigation options as part of the Catchment Management Plan.

The model review memo (dated 7 March 2014) outlines comments on the methodology adopted as part of the Overland Flood Study following detailed review of the existing Tuflow model and observations made during the initial site visit. The existing model has been updated and the results are discussed below.

This assessment focuses on the 100yr Average Recurrence Interval (ARI) event only and for the critical storm duration of 2 hours. Preliminary mapping has been undertaken to identify resulting difference and are attached. It is noted that the 100yr ARI extents and depths (Figure 1A to Figure 1C) do not have a depth filter applied.

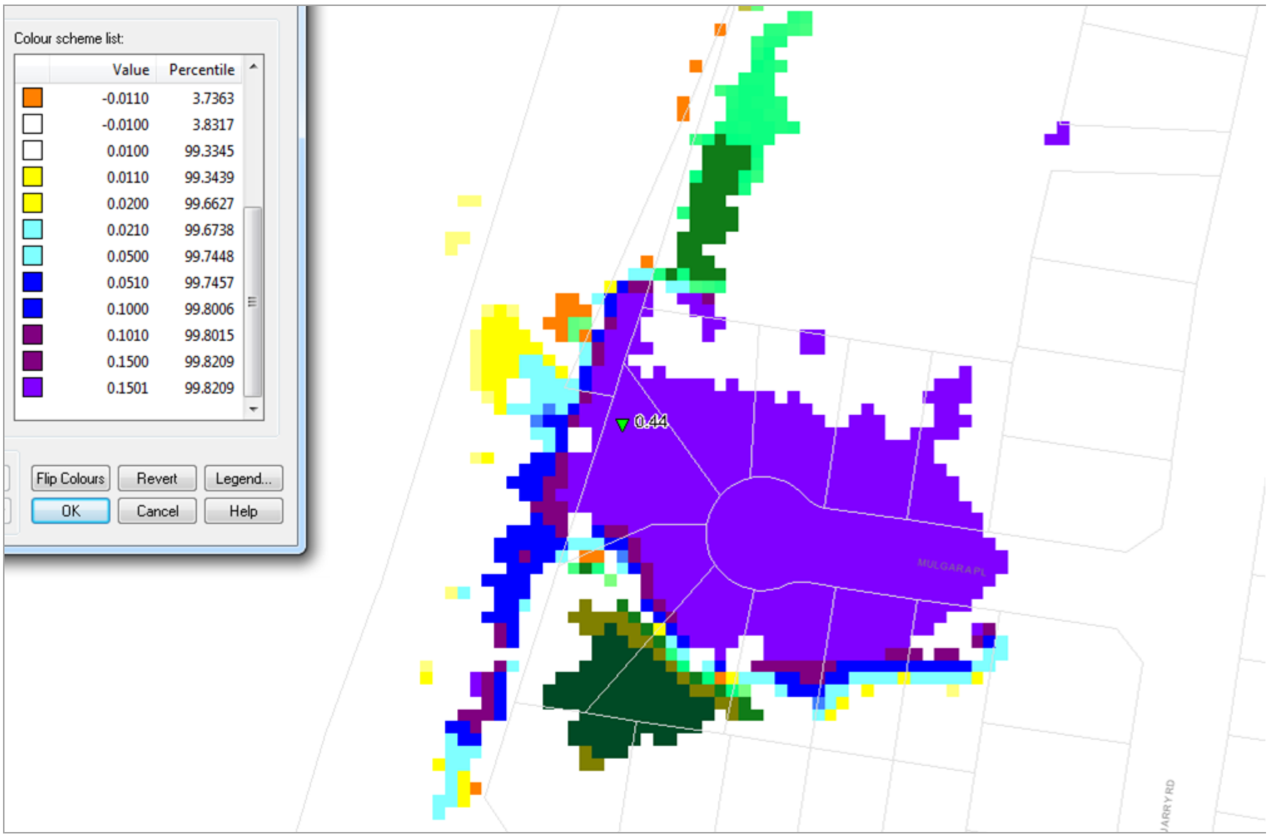
Inclusion of Noise Wall adjacent to Cowpasture Road

The inclusion of the noise wall at the rear of residential properties off Cowpasture Road has a significant affect on overland flow. The wall effectively blocks overland flow from the branch originating east of Jarrah Place resulting in increased water ponding at the residential properties on Mulgara Place adjacent to the wall (up to 0.44m, see Figure 2A). This water subsequently drains away via the existing pipe network. As the noise wall temporarily retains a portion of the overland flow, decreases in 100yr water levels downstream are evident as shown on Figure 2A.

Updates to Channel Schematisation

In the existing hydraulic model, there are very long connections within the 1D network. Between The Horsley Drive and Toohey Road the 1D channel is represented by a single reach length of approximately 500m. In order to refine the 1D network, the reach was updated to represent cross sections at intervals of 100m which results in greater definition of channel flow. The channel schematisation used in both the existing and updated models is shown in Figure 3.

Figure 2A Extract – Differences in 100yr ARI Water Levels due to Inclusion of Noise Wall



The figures below indicate the differences in 100yr ARI event peak flow and velocities between the existing and updated models as a result of the channel amendments.

In the existing case, a peak flow of 44.3m^3/s is shown for the single reach between The Horsley Drive and Toohey Road while in the updated model the peak flow ranges from 39.8m^3/s to 42.1m^3/s. The reductions in 1D peak channel flows are attributed to the channel refinements and the inclusion of the noise wall upstream which temporarily retains some of the overland flow. Note the figures below outline 1D peak flows (i.e. flow conveyed within the channel only). Updates to channel schematisation were also applied between Newton Road and Victoria Street with similar results.

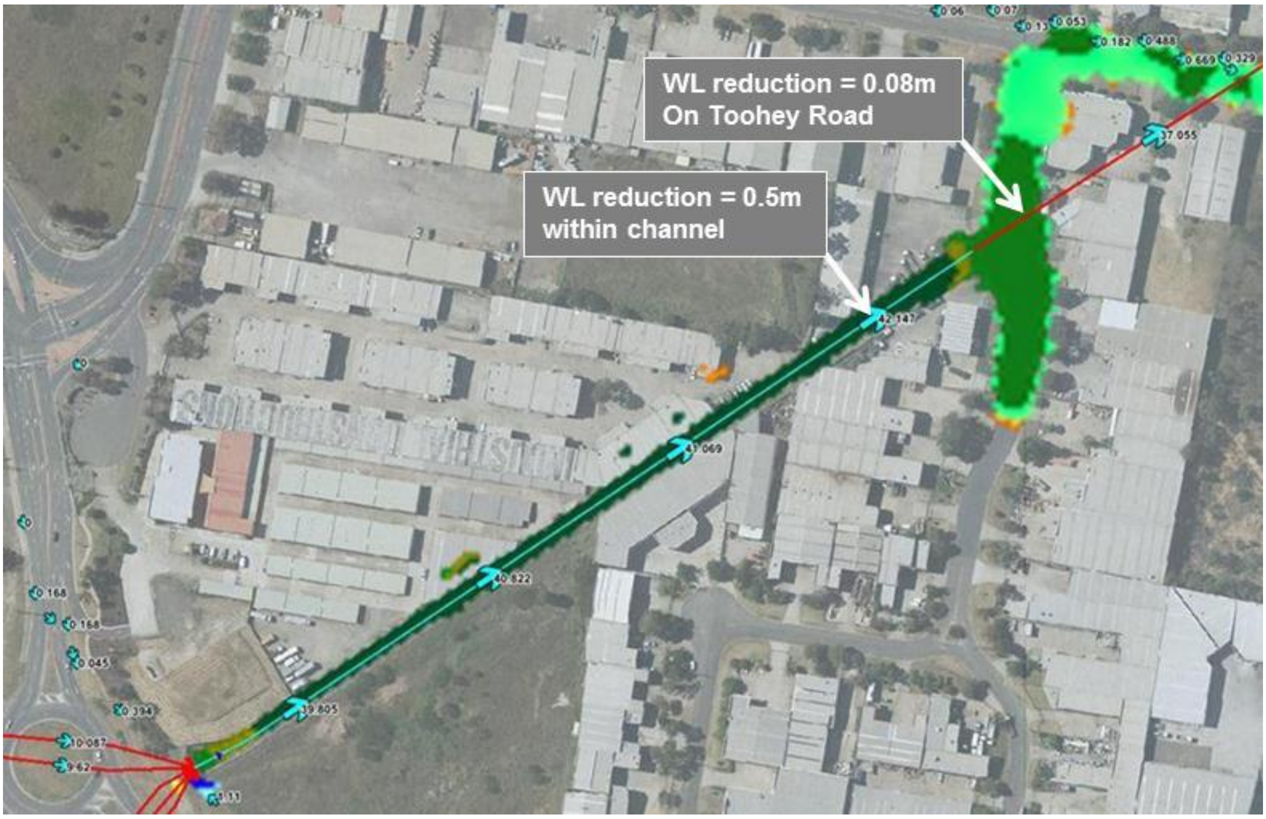
The reductions in peak flow and amendments to the channel schematisation have resulted in in-channel water level reductions of up to 0.5m just upstream of Toohey Road and 0.35m upstream of Victoria Street.

At the intersection of Toohey Road and Newton Road, the direction of overland flow and associated velocities for both the existing and updated model is similar. Flow overtopping the channel is conveyed northwards along Toohey Road and Newton Road.

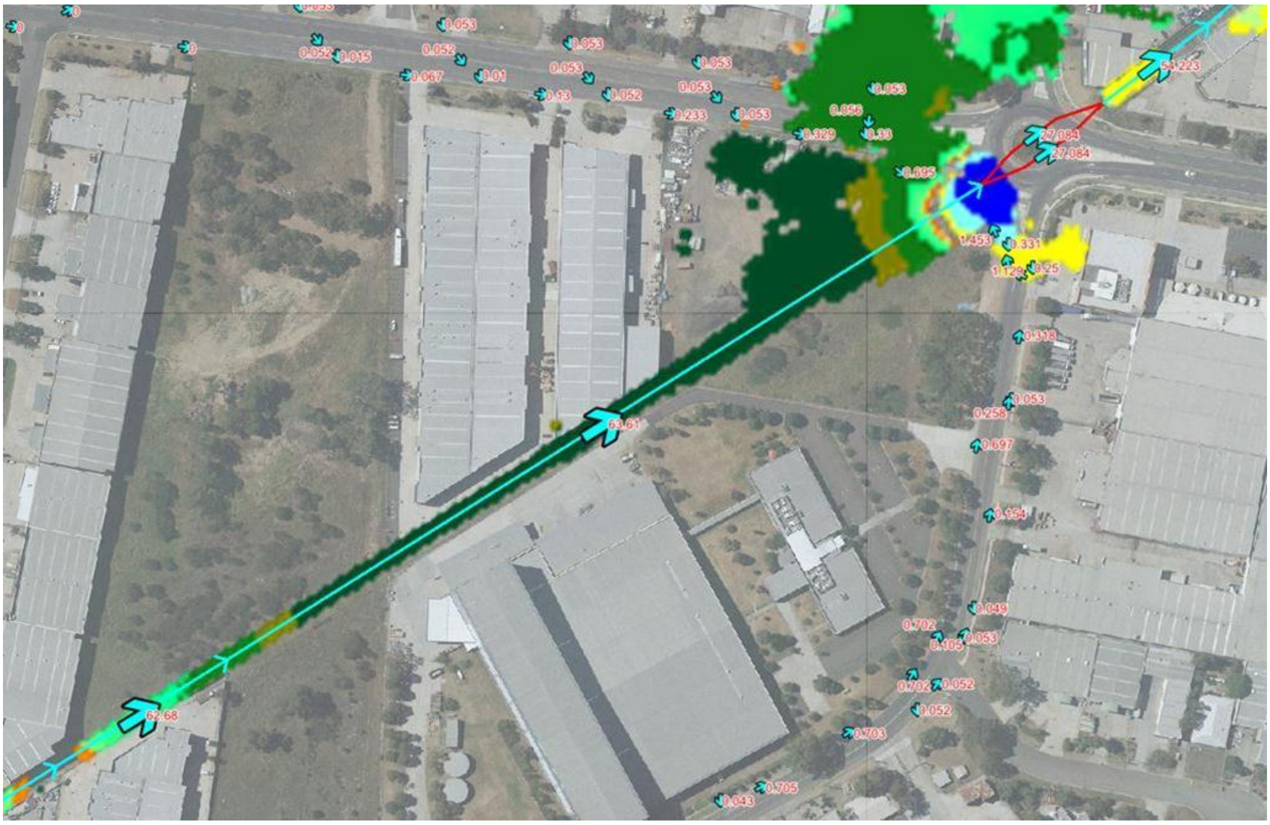
At the intersection of Newton Road and Victoria Street, the additional flow conveyed through the culverts, and improvements due to the channel updates, have resulted in less flow overtopping the channel and being conveyed northwards along Newton Road. Comparison of velocity figures show the general direction of velocity vectors is similar with slight reductions in 1D velocity (i.e. within the channel) and also overland.

1D Channel Flows Upstream of Toohey Road – Existing Model (m^3/s)

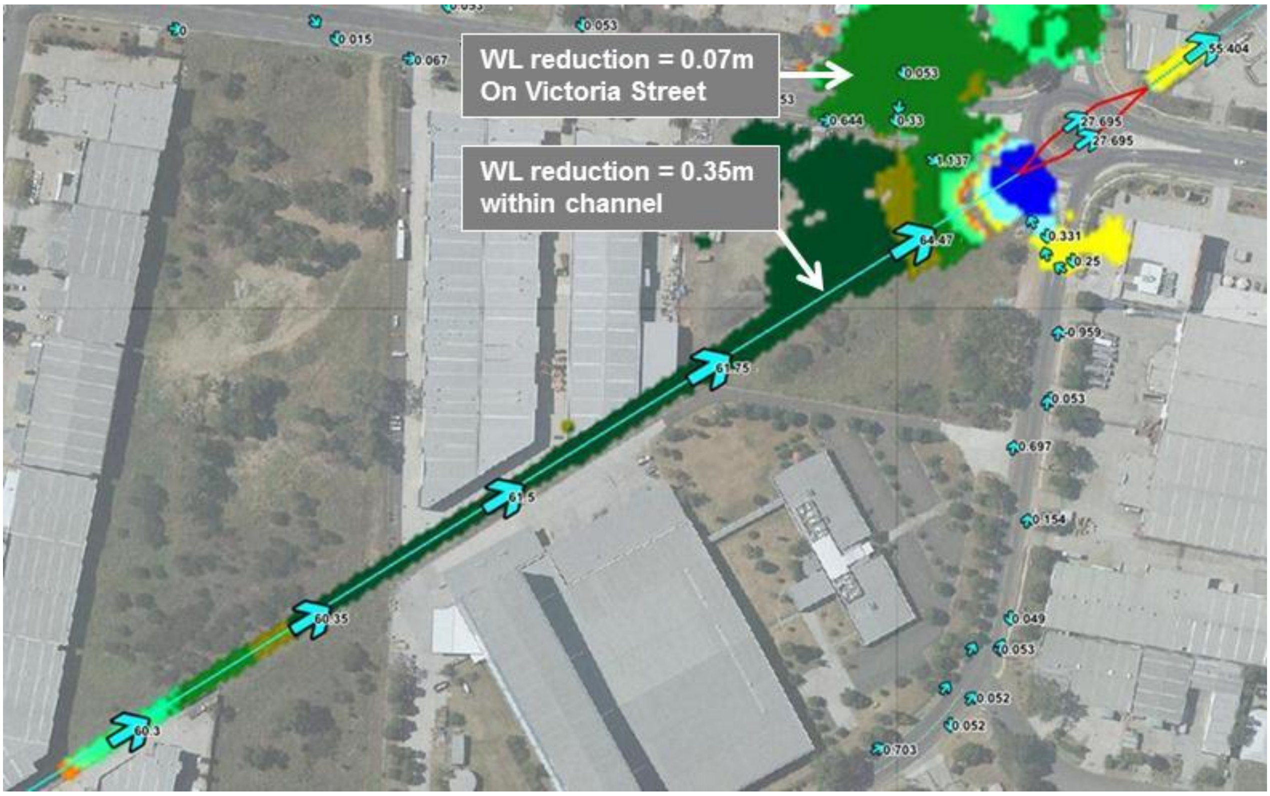
1D Channel Flows Upstream of Toohey Road – Updated Model (m^3/s)



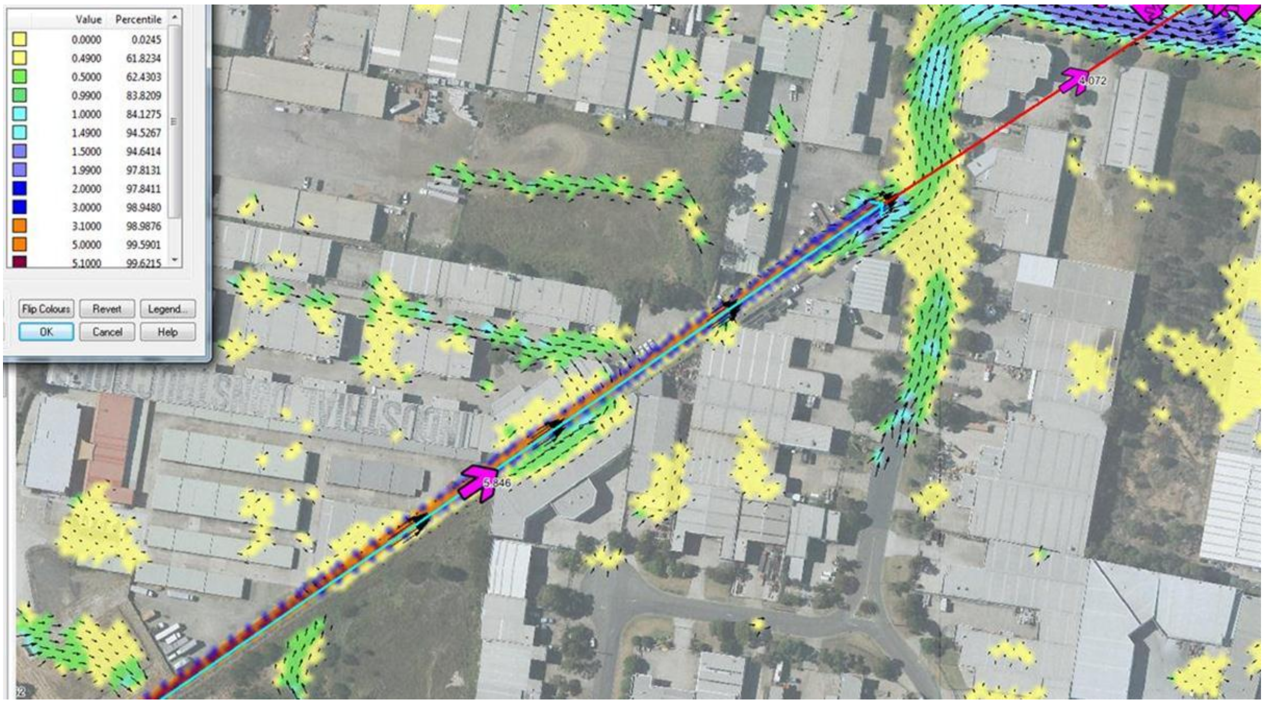
1D Channel Flows Upstream of Victoria Street – Existing Model (m^3/s)



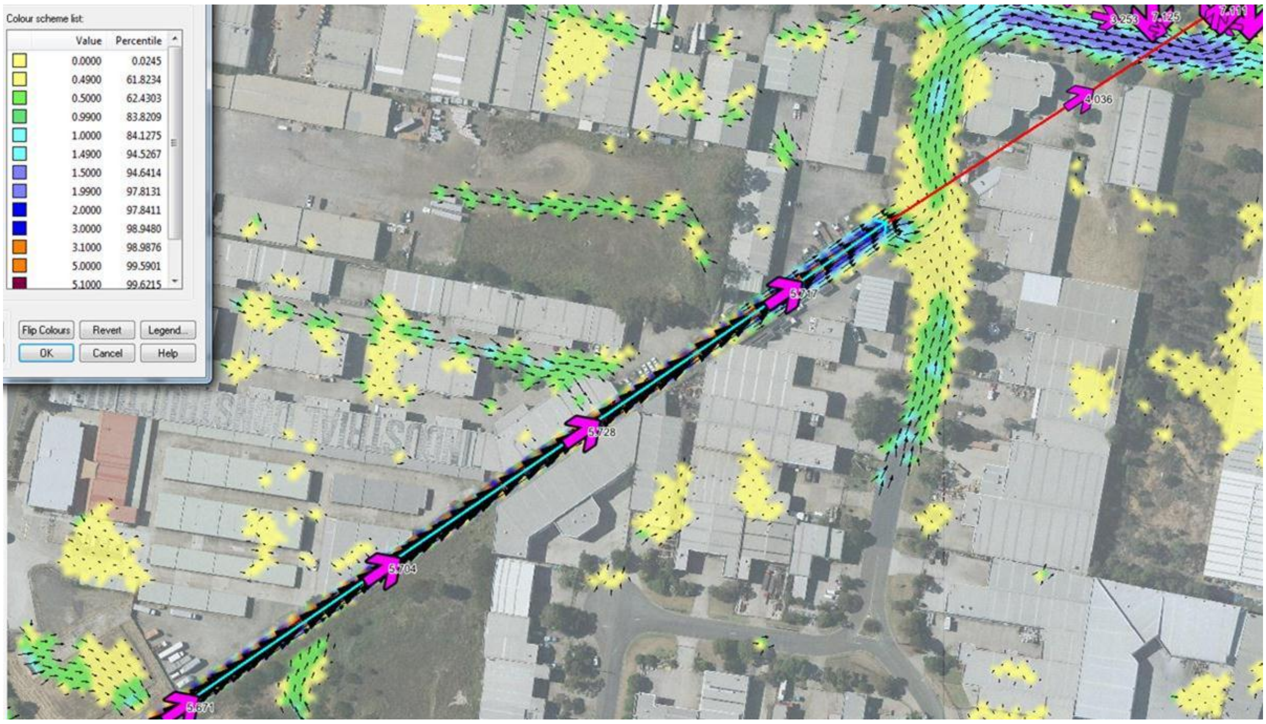
1D Channel Flows Upstream of Victoria Street – Updated Model (m^3/s)



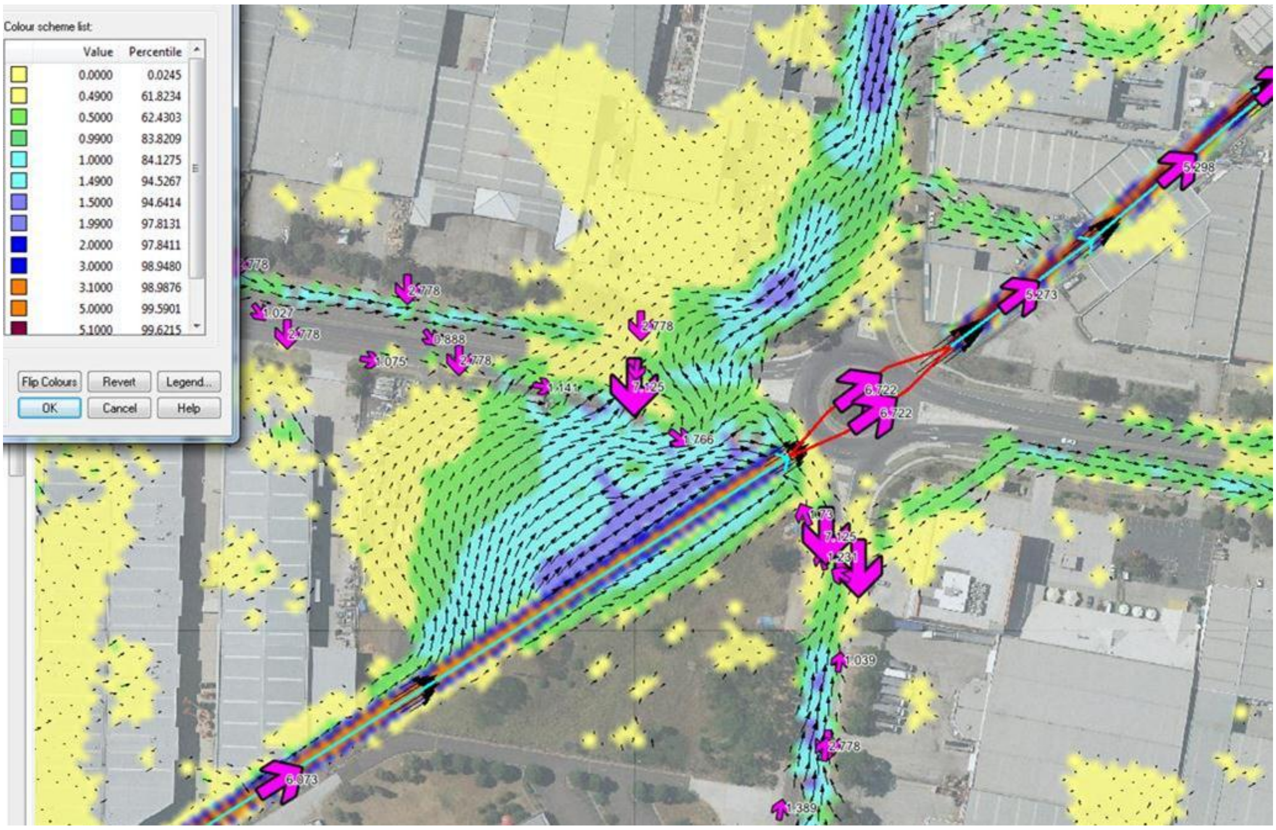
Intersection of Toohey Road and Newton Road – Existing Model Velocities (m/s)



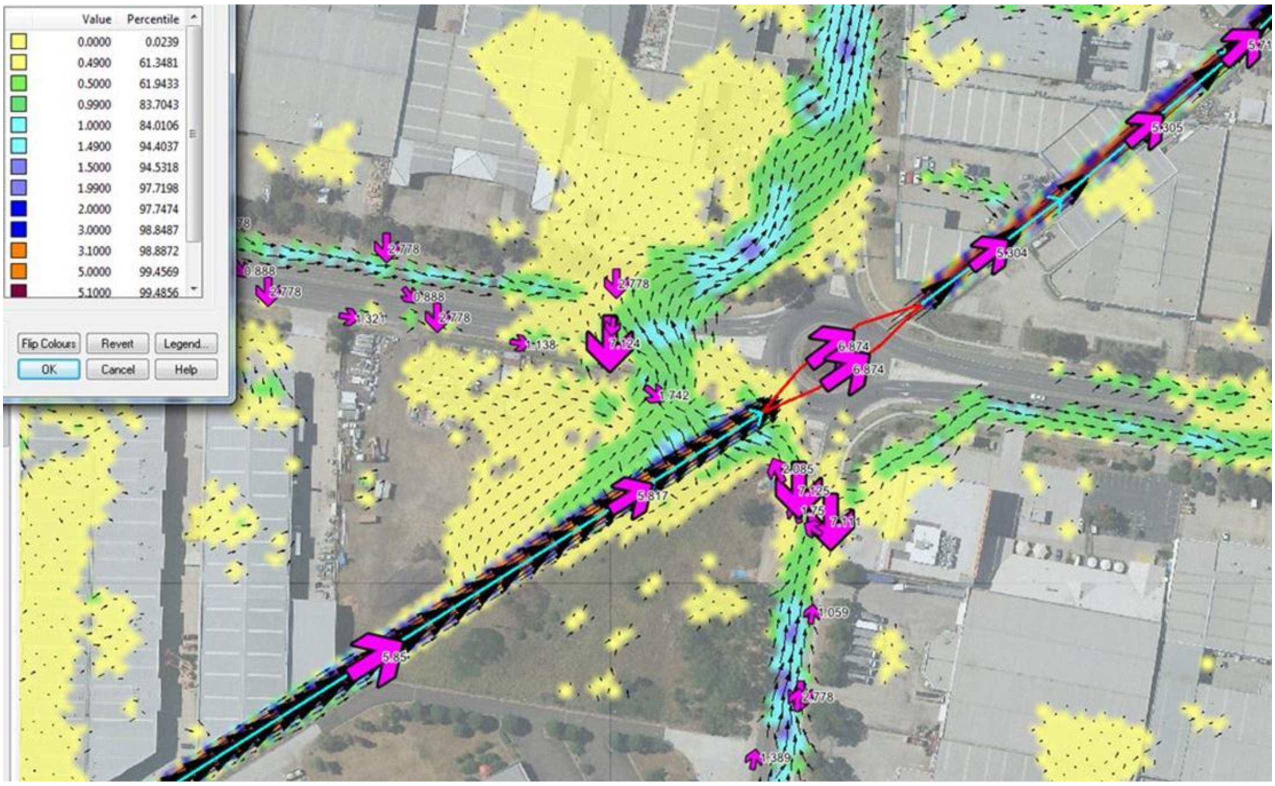
Intersection of Toohey Road and Newton Road – Updated Model Velocities (m/s)



Intersection of Newton Road and Victoria Street – Existing Model Velocities (m/s)



Intersection of Newton Road and Victoria Street – Updated Model Velocities (m/s)



Culvert Flows

The peak culvert flows at the main crossings along the main channel are shown in Table 1-1.

Table 1-1 Culvert Flows

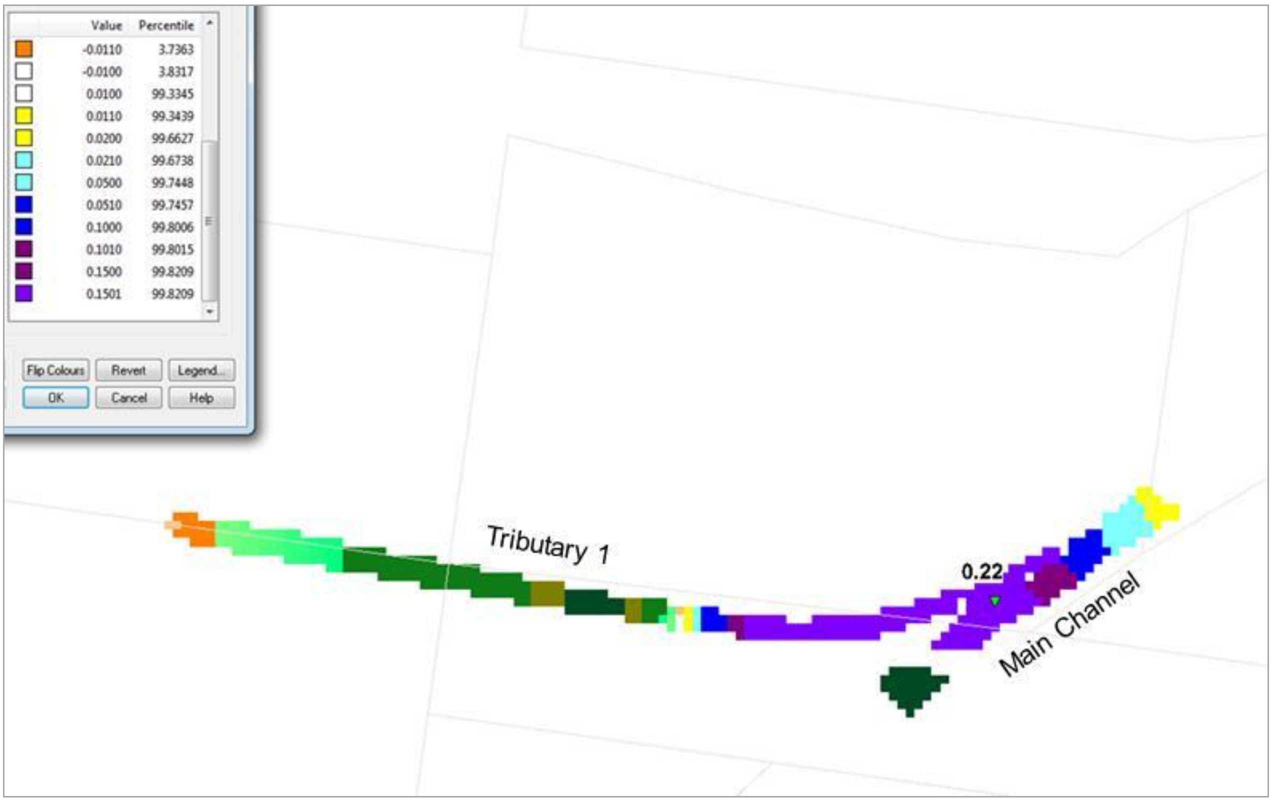
Location	Existing Model	Updated Model
Intersection of Toohey Road and Newton Road	37.4m^3/s	37.1m^3/s
Intersection of Newton Road and Victoria Street	27.1m^3/s, 27.1m^3/s	27.7m^3/s, 27.7m^3/s

No changes to the culverts were made as part of the model update. There is a slight reduction in culvert flow at the Toohey Road and Newton Road intersection with a 1.2m^3/s increase in culvert flow expected at the intersection of Newton Road and Victoria Street as the culverts operate more efficiently.

Confluence of Main Channel and Tributary 1

At the confluence of the main channel and Tributary 1 there was double counting of channel flow area and volume in the existing model. In the current model this has been rectified and as a result there are increases in flood levels of up to 0.22m, as shown below. However, the 100yr extent is contained within the channel so is not considered to be a major issue.

Figure 2C Extract – Differences in 100yr ARI Water Levels at Main Channel and Trib. 1 Confluence



Summary

The amendments to the existing hydraulic model have altered the channel and overland flow behaviour within parts of the study area as follows:

- Inclusion of the existing noise wall adjacent to Cowpasture Road impacts the overland flow path originating east of Jarrah Place. This results in increased water ponding at residential properties on Mulgara Place, adjacent to the noise wall, with increases of up to 0.44m expected. The wall acts as an obstruction and reduces peak flows downstream as it temporarily retains overland flow;

- Updates to channel schematisation, combined with reductions in peak flow as a result of the noise wall, have refined peak flows and resulted in reductions in 100yr ARI flood levels and velocities both within the channels and where overland flow occurs. Reductions in water levels of up to 0.5m and 0.35m were identified upstream of Toohey Road and Victoria Road, respectively;

- The culverts beneath the intersection of Victoria Road and Newton Road are operating more effectively and conveying an additional $1.2m^3/s$ in flow compared to the existing hydraulic model. This results in reductions in above ground ponding in the vicinity of the intersection; and

- At the confluence of the main channel and tributary 1, double counting of channel flow area and volume occurred in the existing model. This was updated as part of this assessment with increases in 100yr ARI water levels of up to 0.22m resulting. The main channel conveys the 100yr ARI peak flows at this location and this increase is not considered to be an issue.

Wetherill Park Catchment Management Plan, Stage 1

APPENDIX

D

FLOOD MAPPING

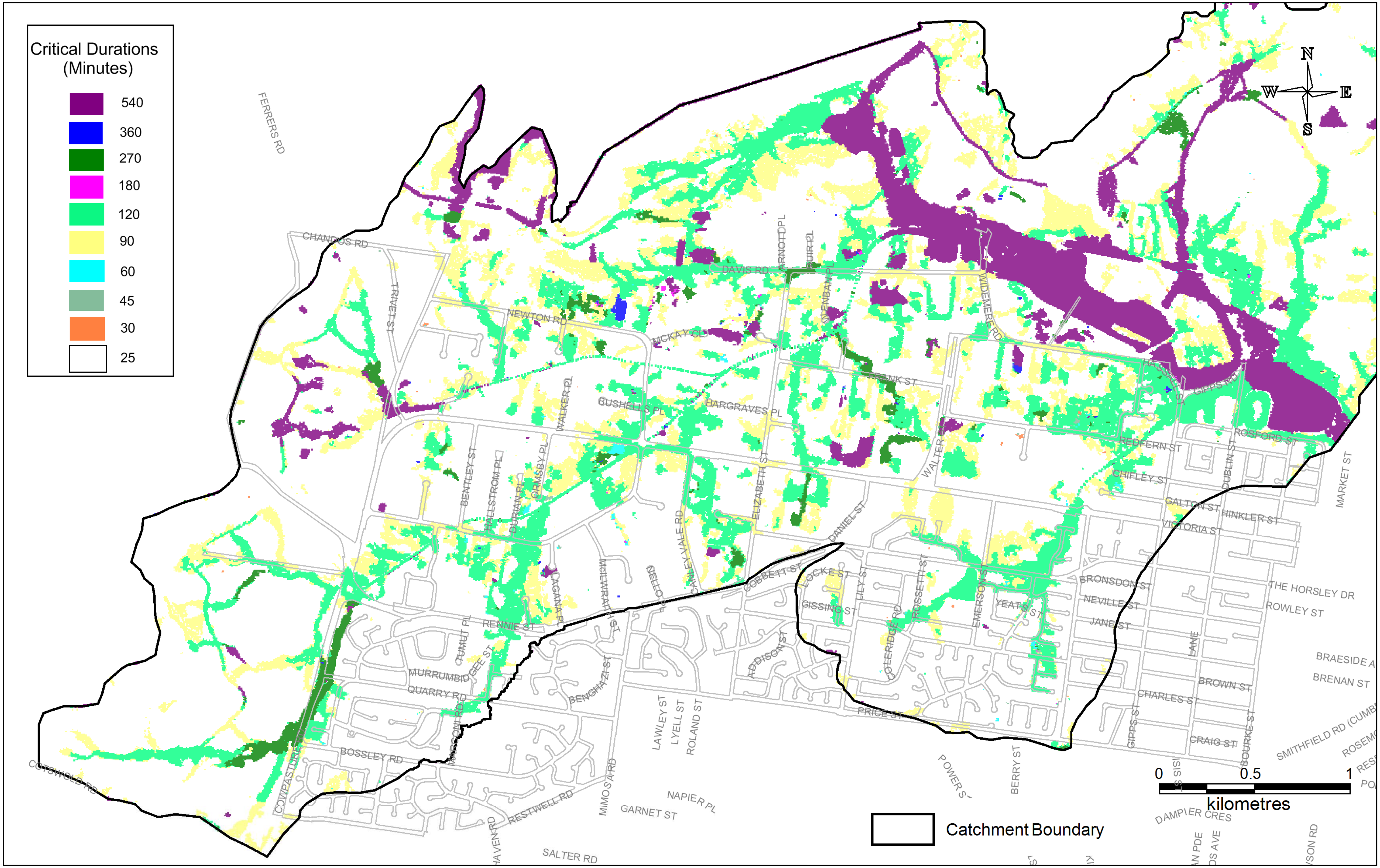


Figure D1 Critical Duration 20Year ARI

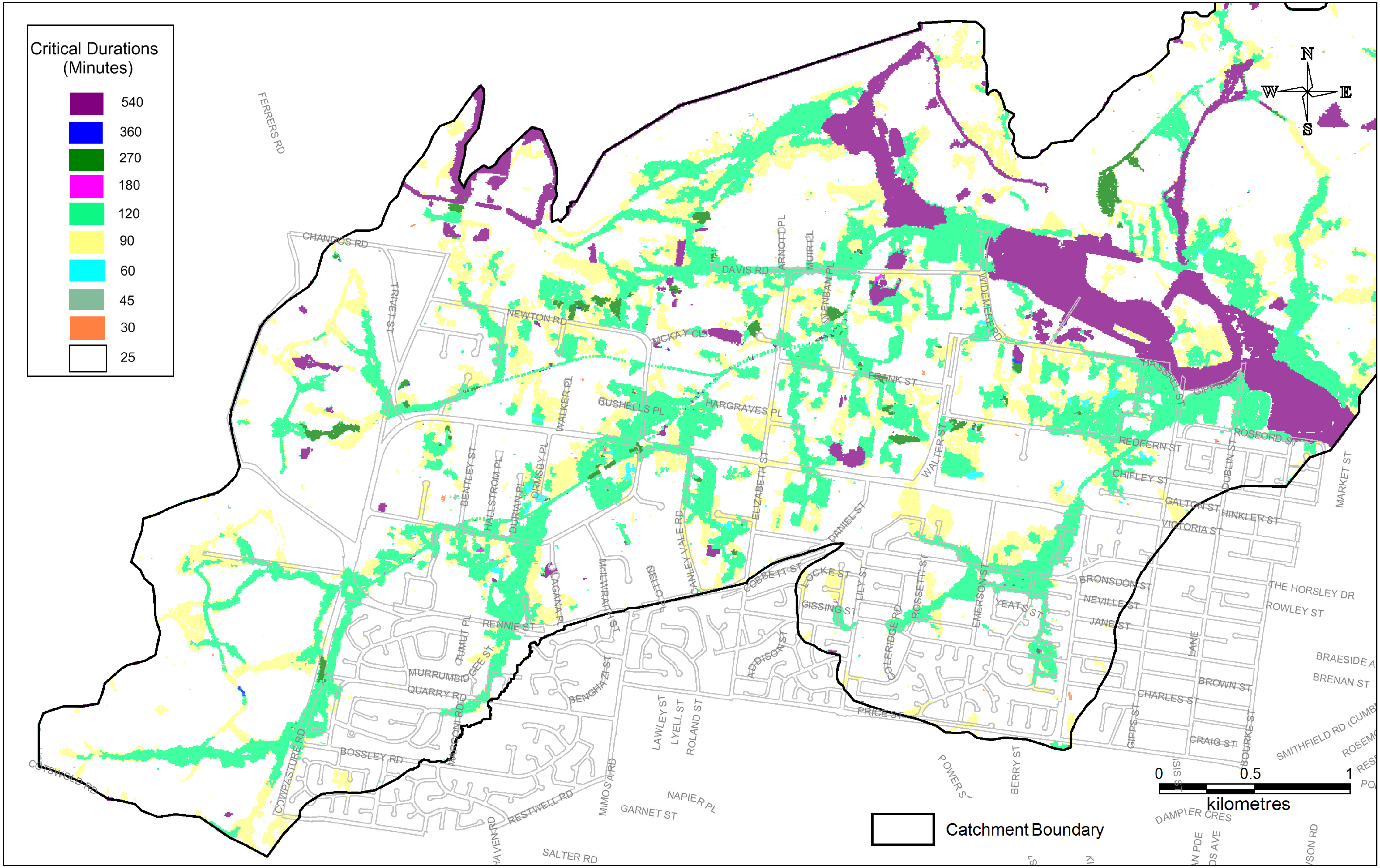


Figure D2 Critical Duration 100Year ARI

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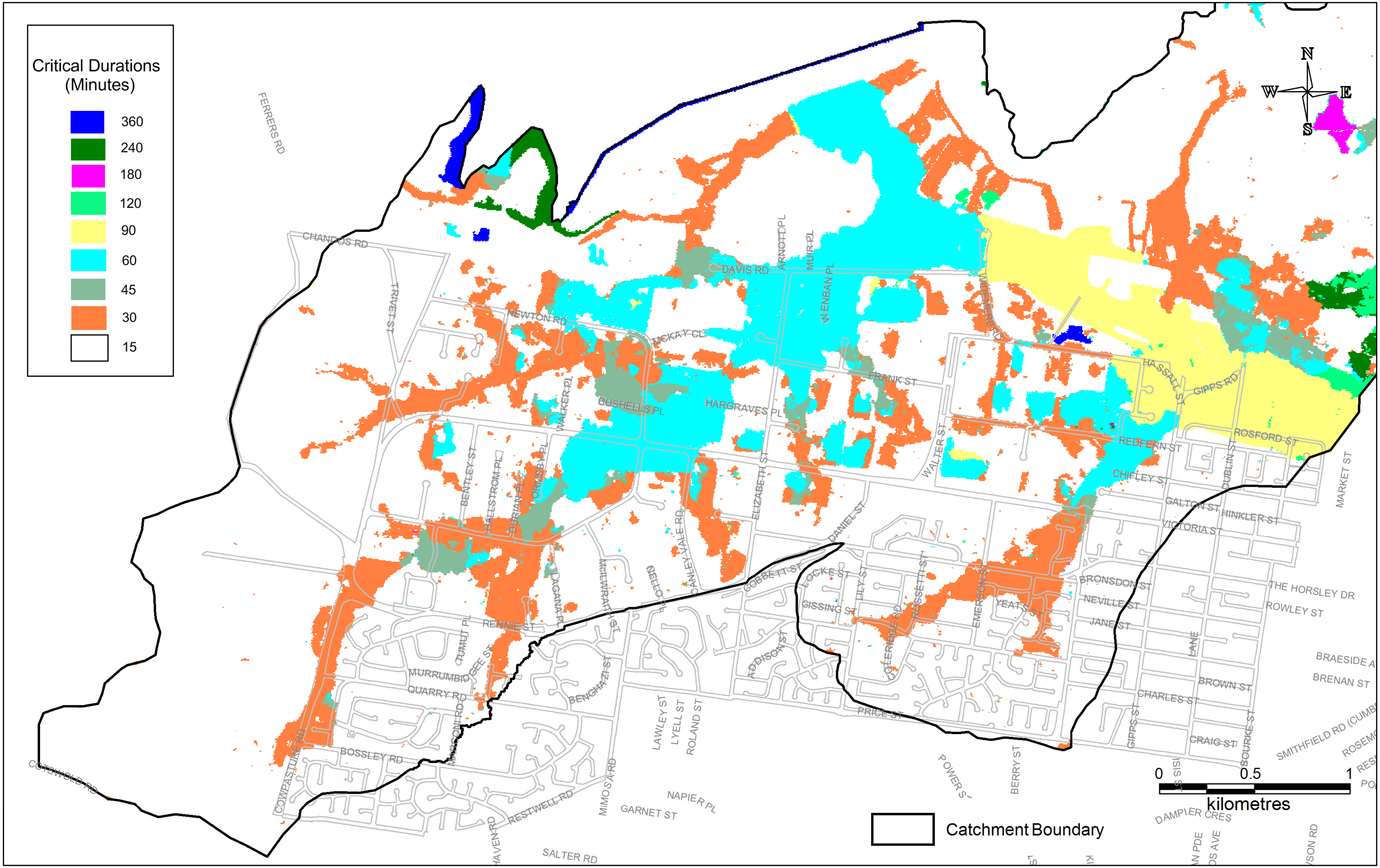

Figure D3 Critical Duration PMF

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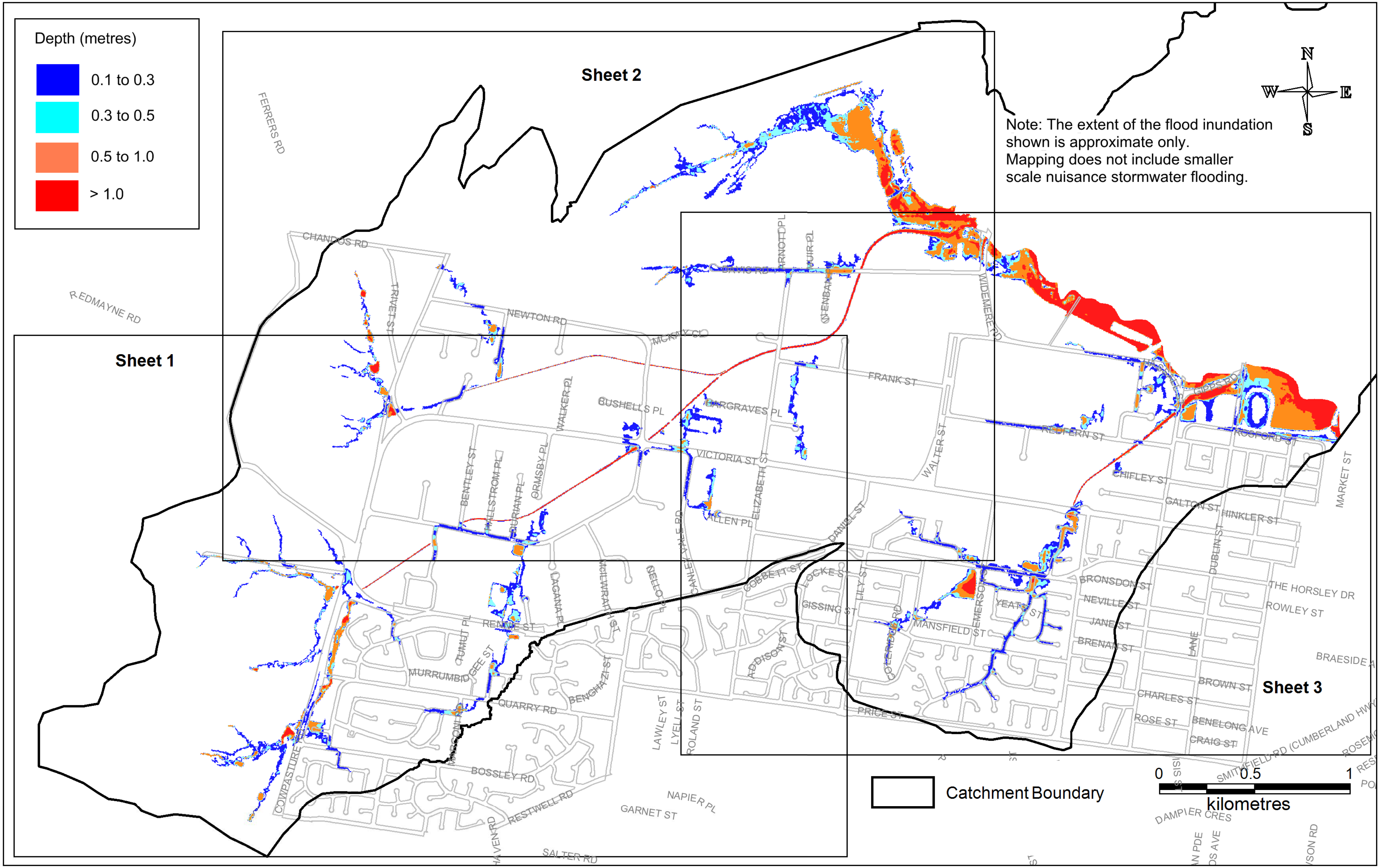

Figure D4 20Year ARI Flood Depths

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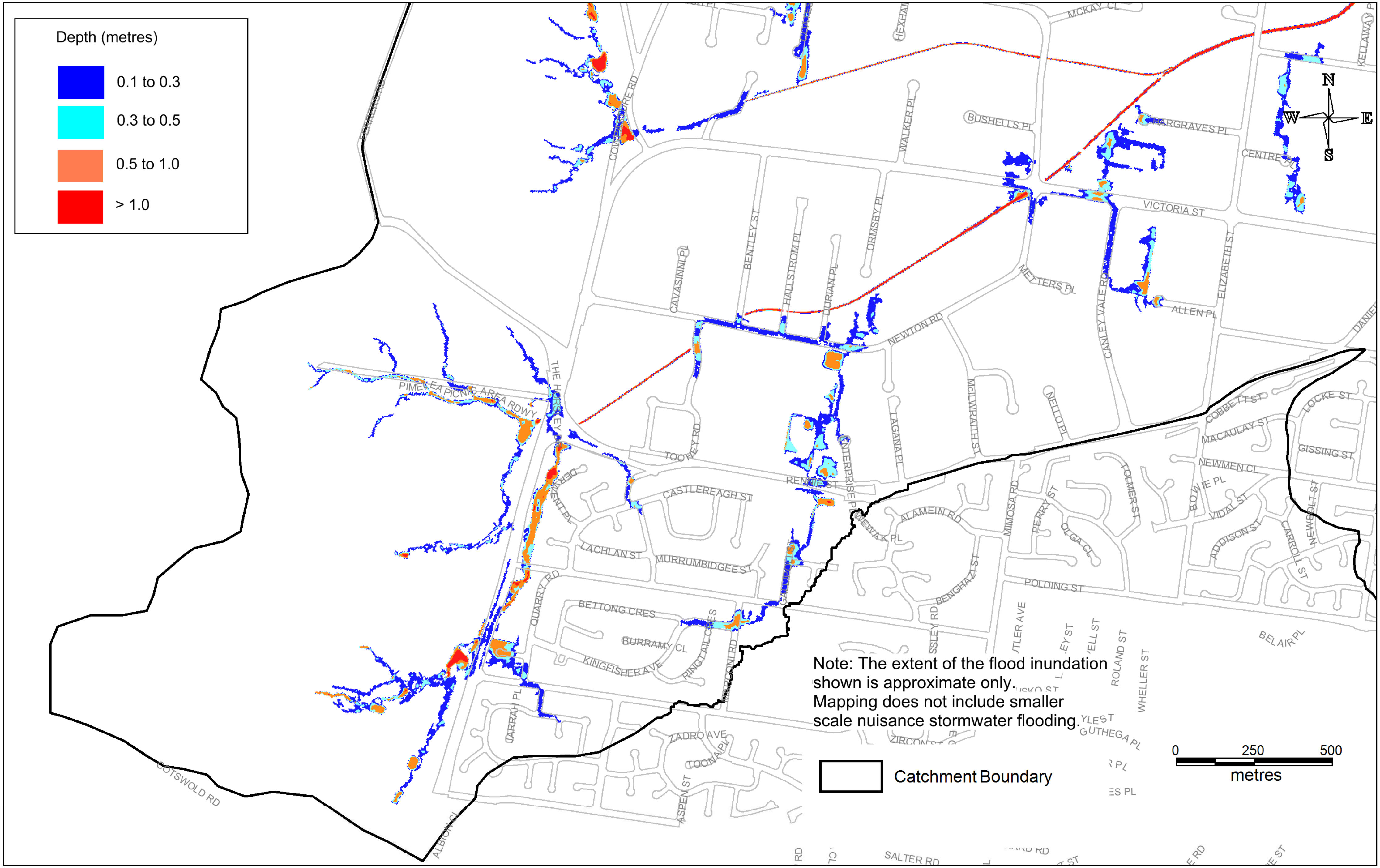

Figure D5 20Year ARI Flood Depths
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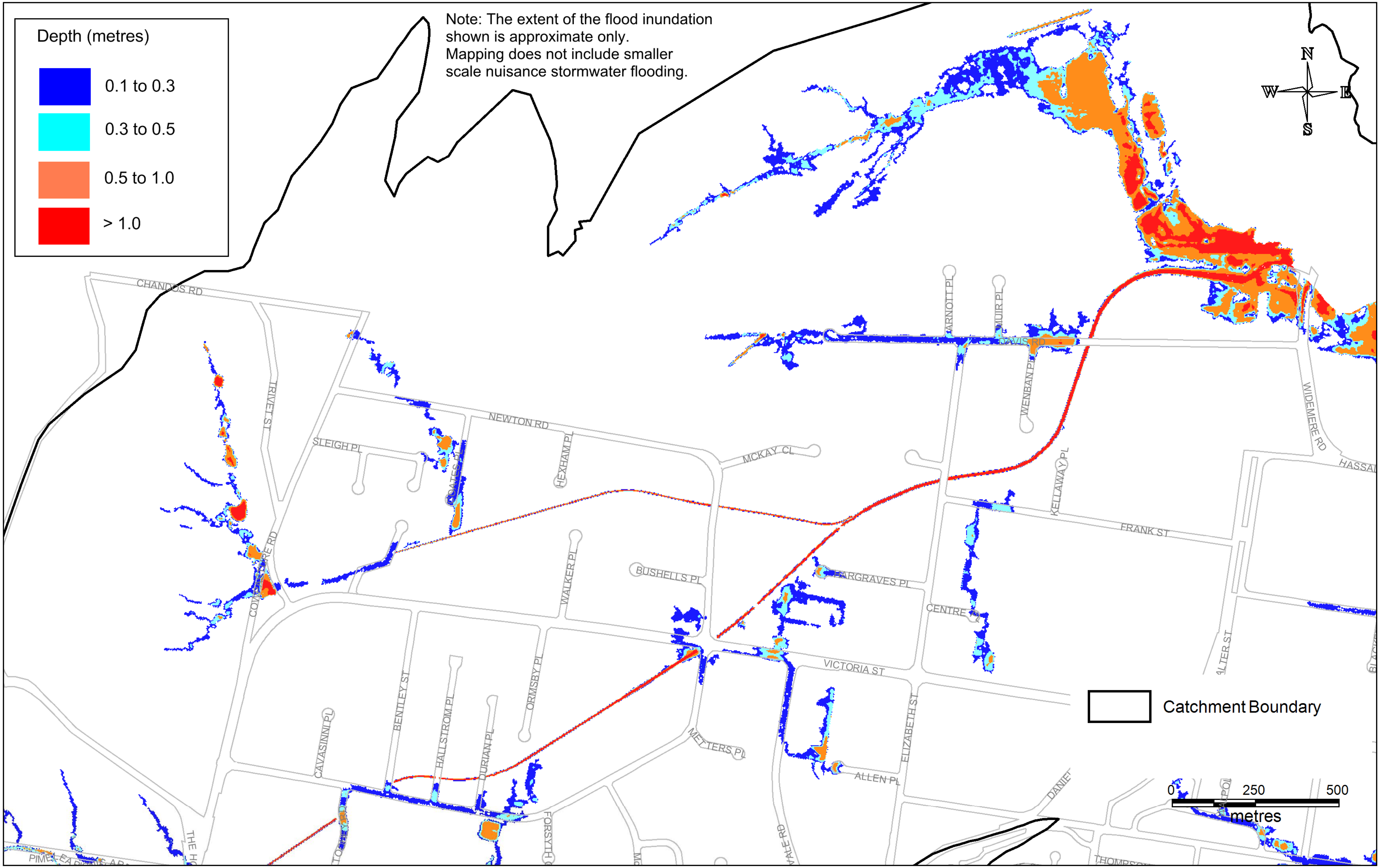

Figure D6 20Year ARI Flood Depths
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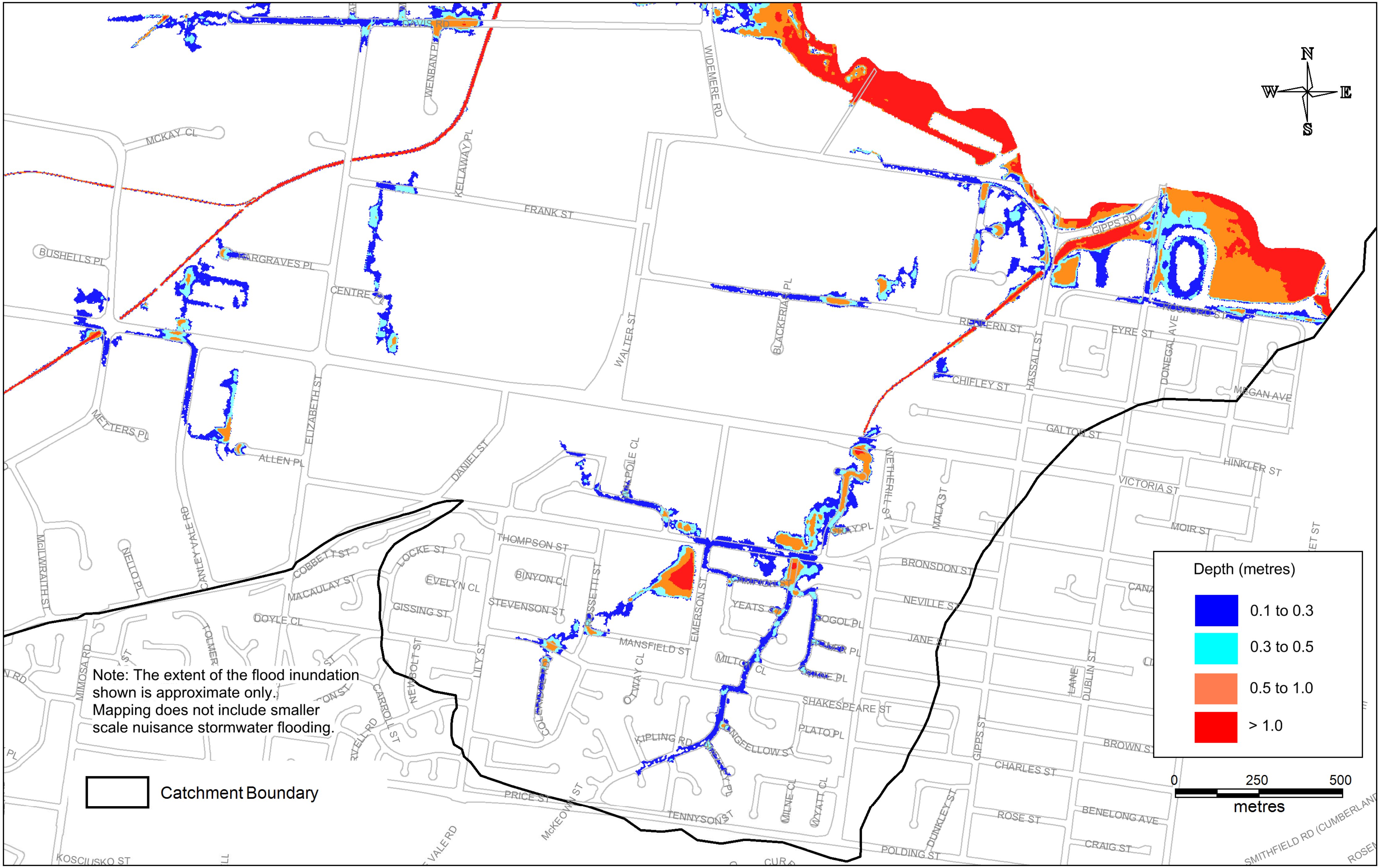

Figure D7 20Year ARI Flood Depths
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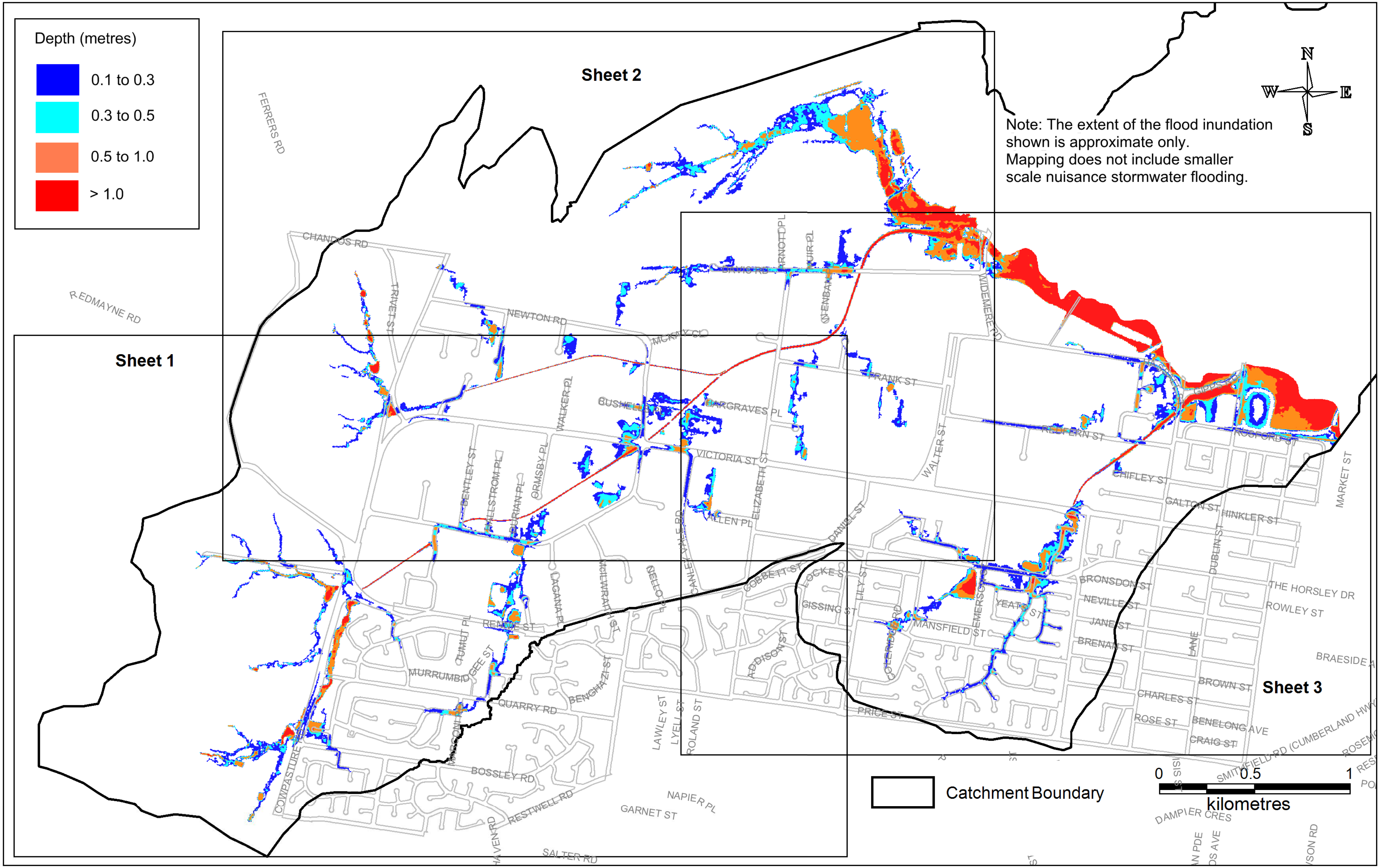

Figure D8 100Year ARI Flood Depths

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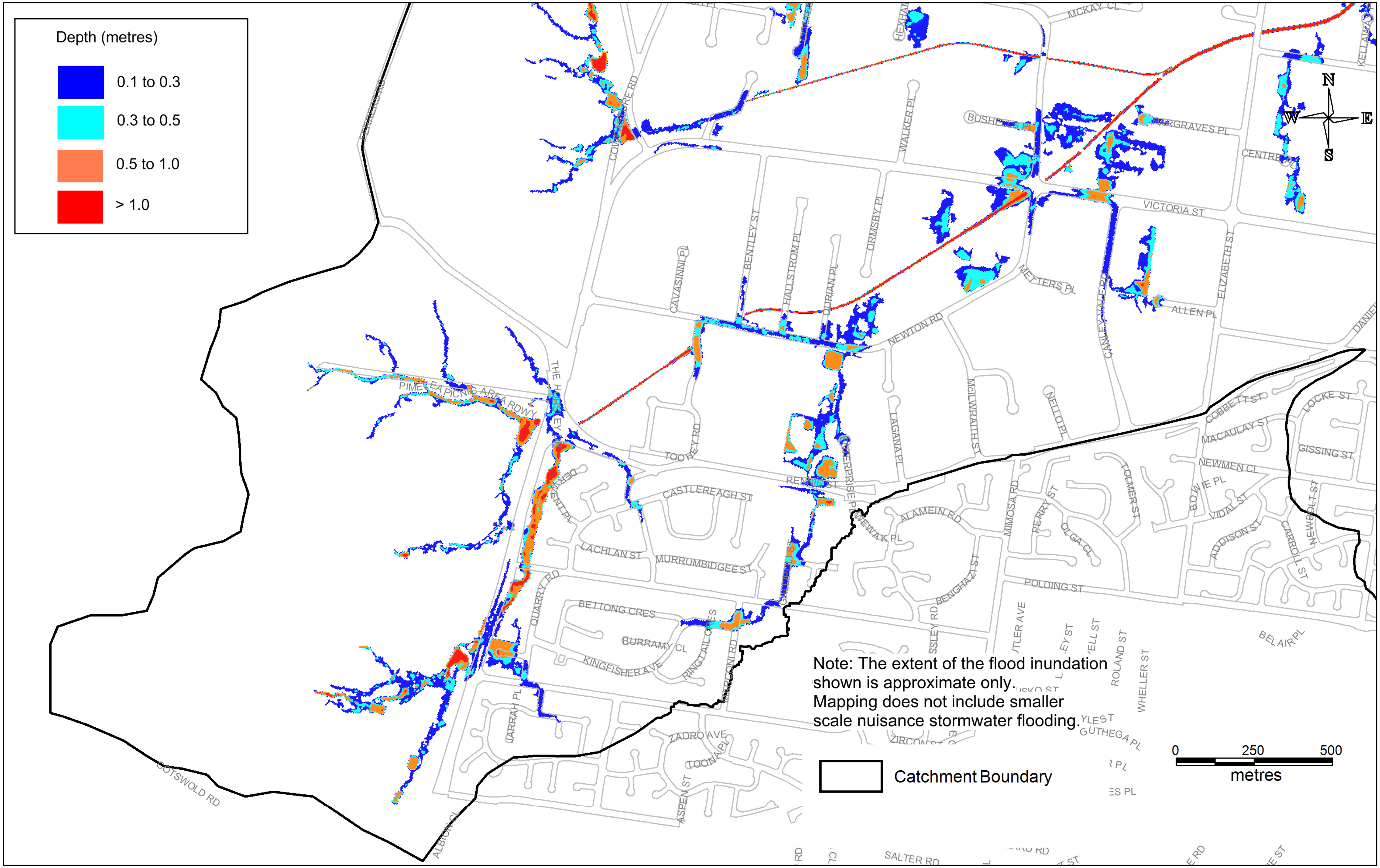

Figure D9 100Year ARI Flood Depths
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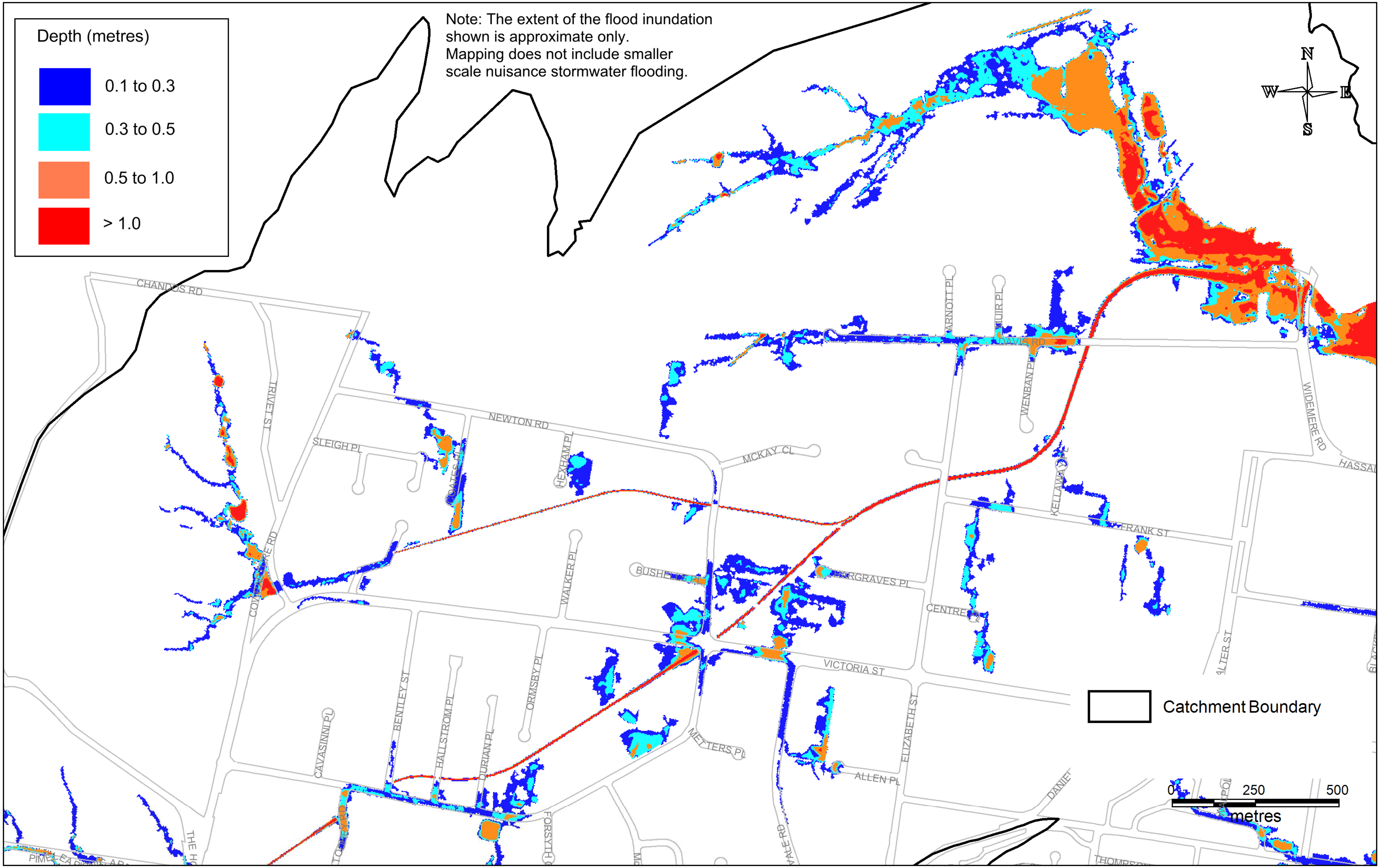

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Figure D10 100Year ARI Flood Depths
Sheet 2

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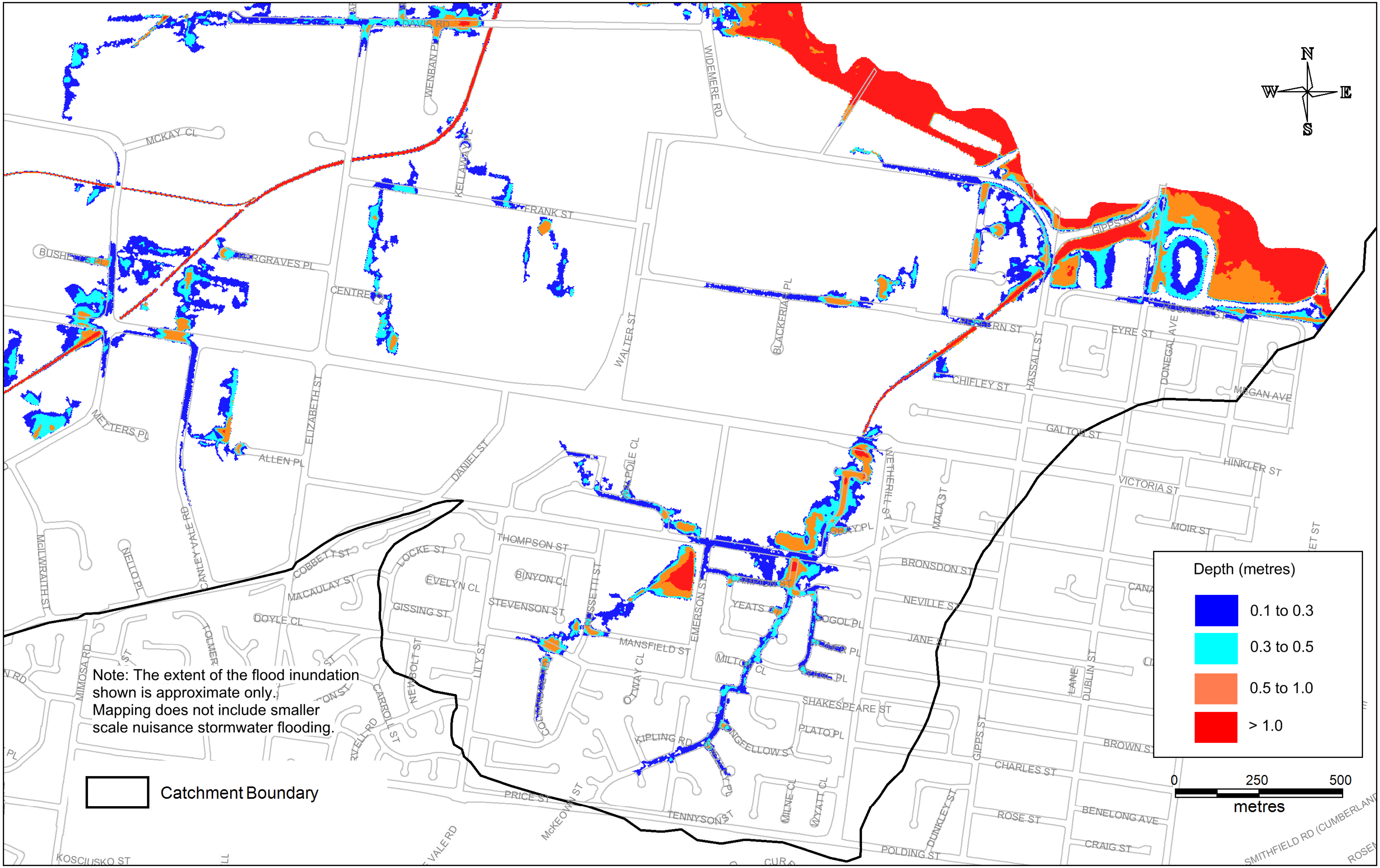

Figure D11 100Year ARI Flood Depths
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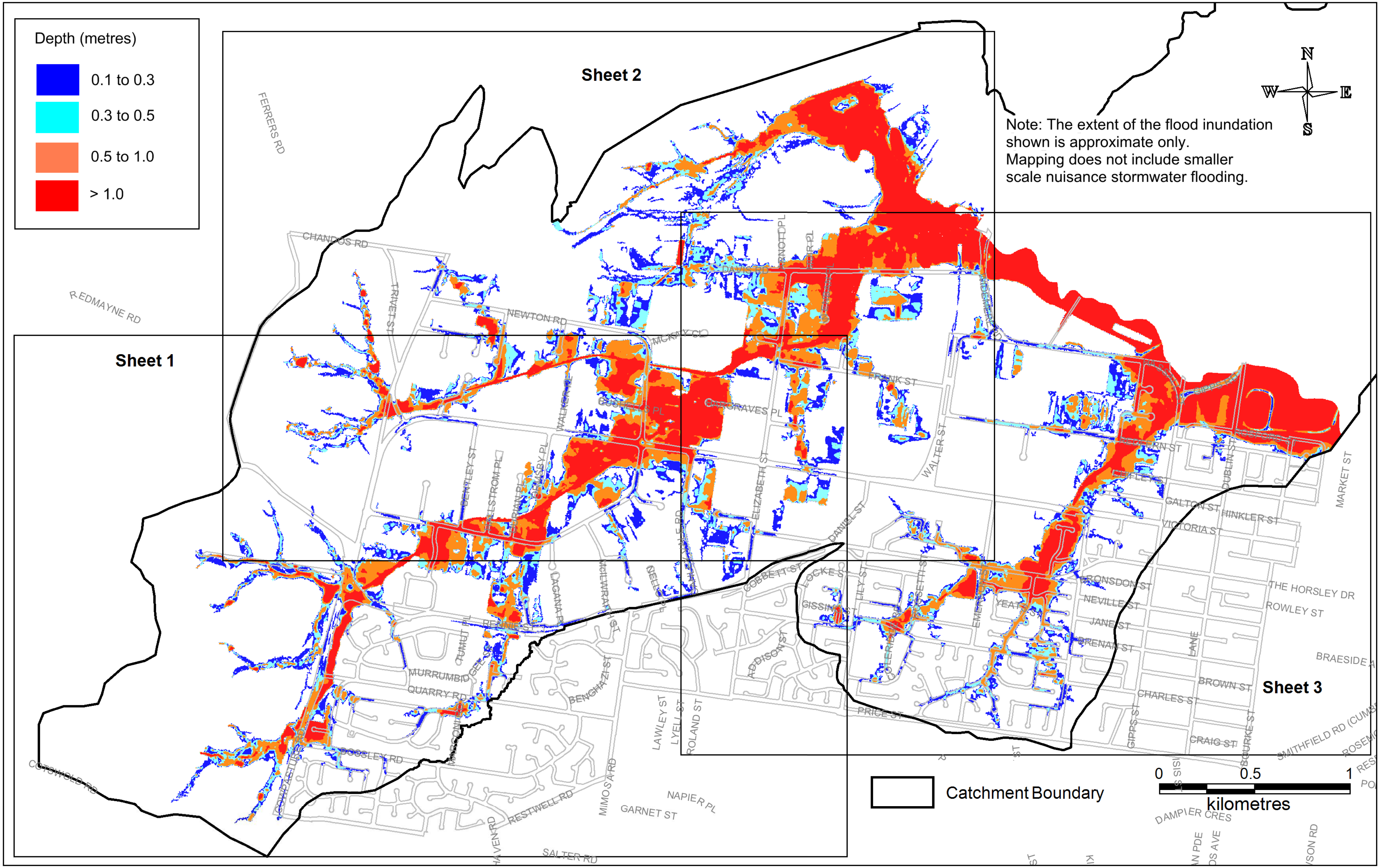

Figure D12 PMF Flood Depths

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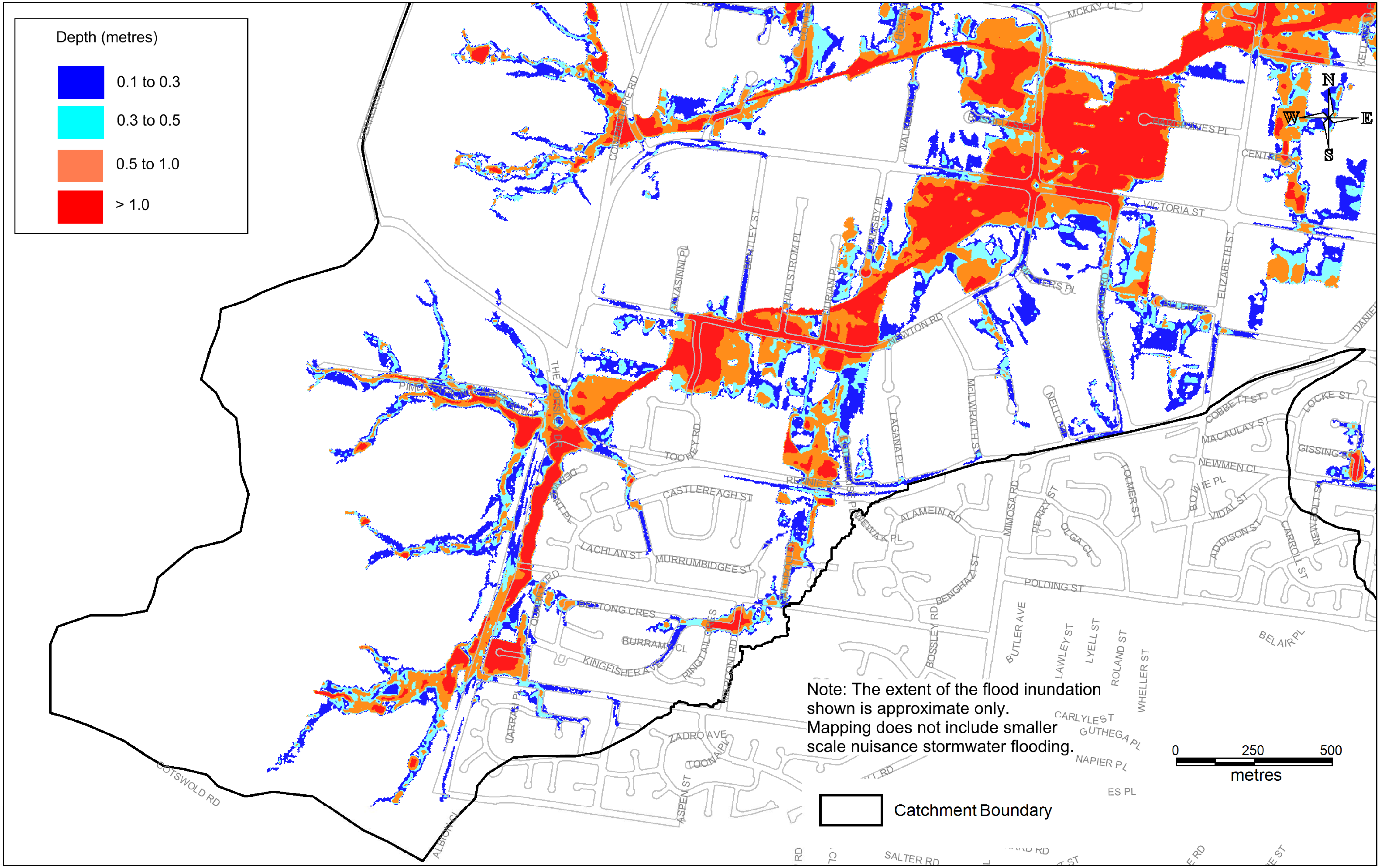

Figure D13 PMF Flood Depths
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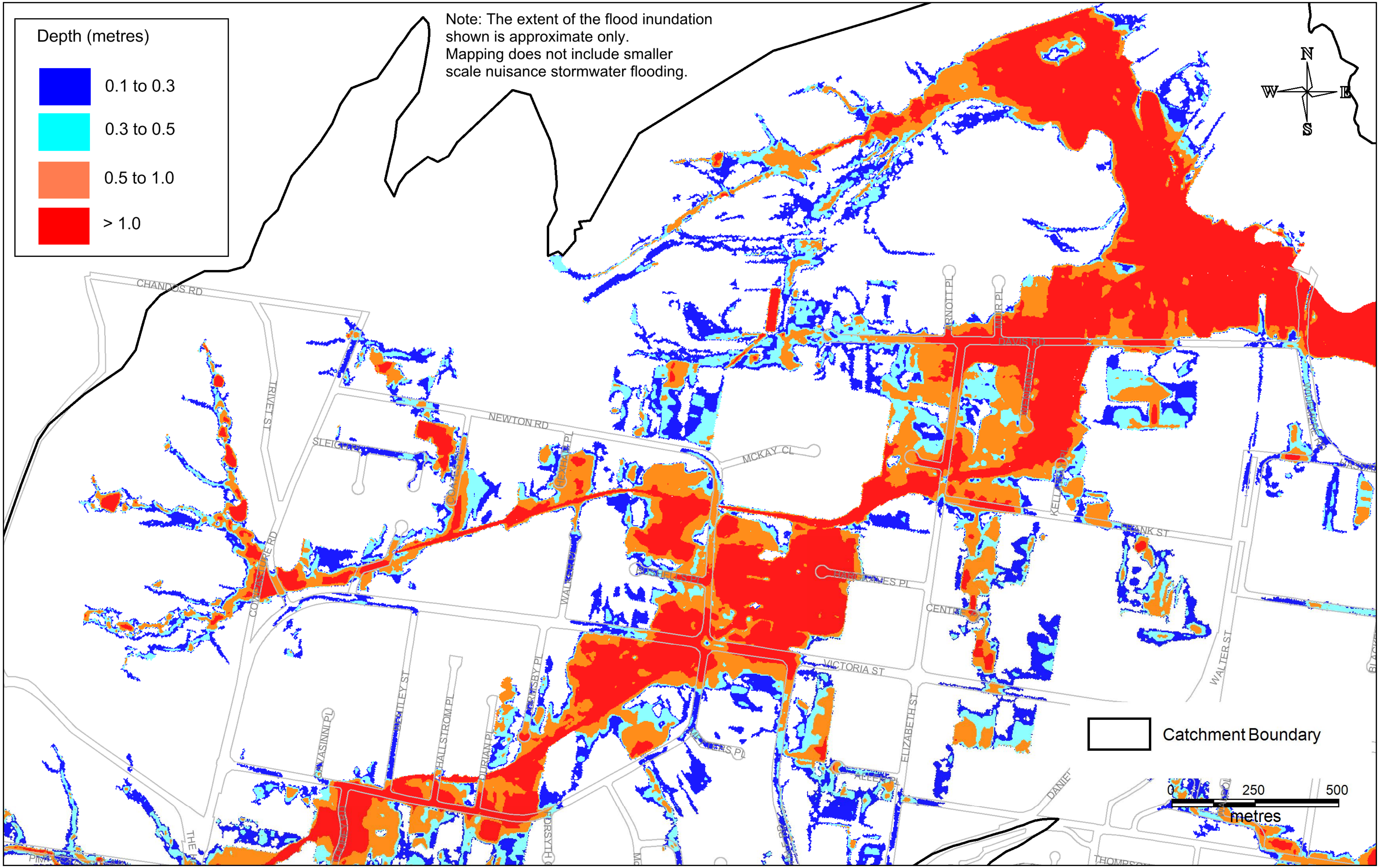

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Figure D14 PMF Flood Depths
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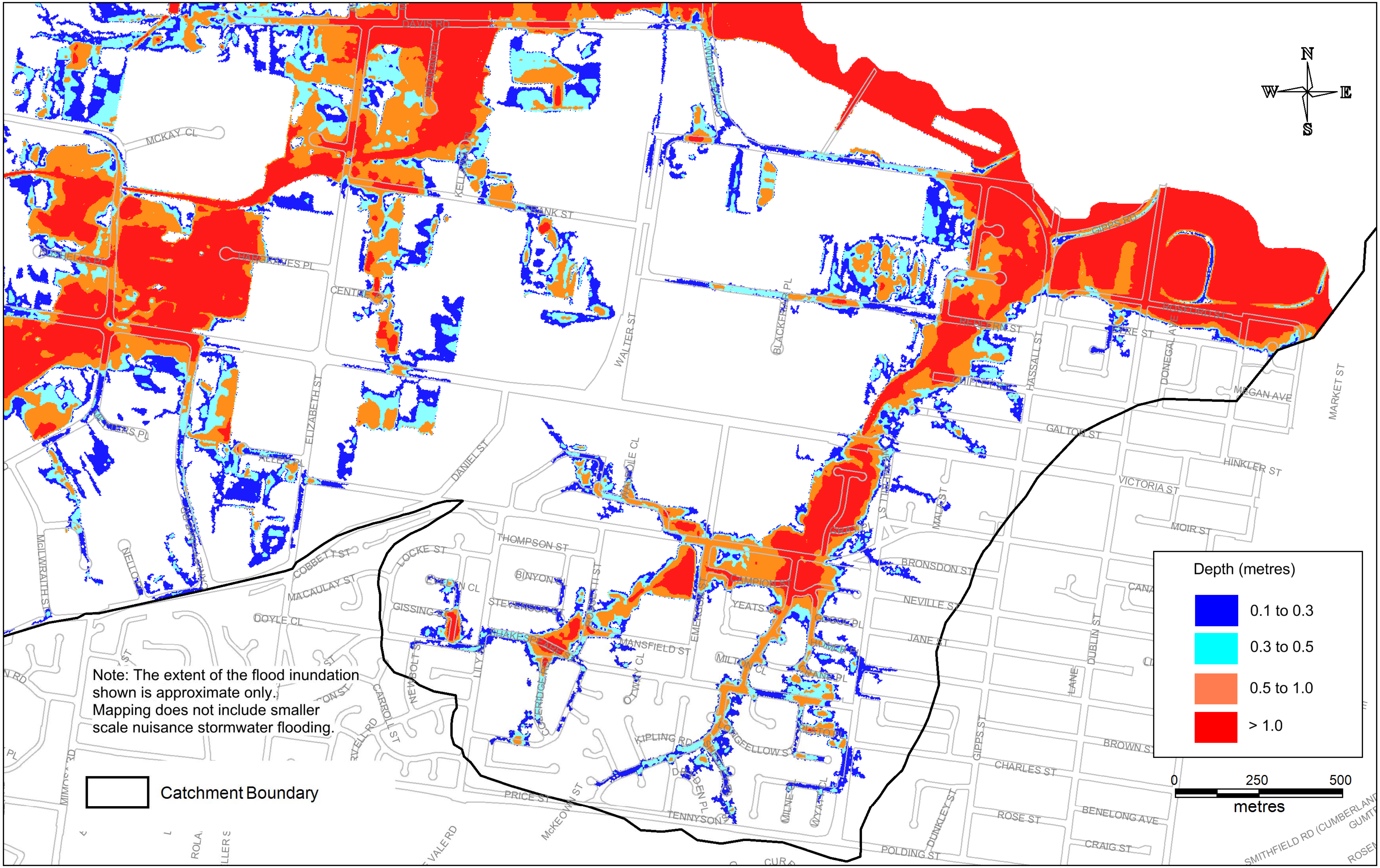

Figure D15 PMF Flood Depths Sheet 3

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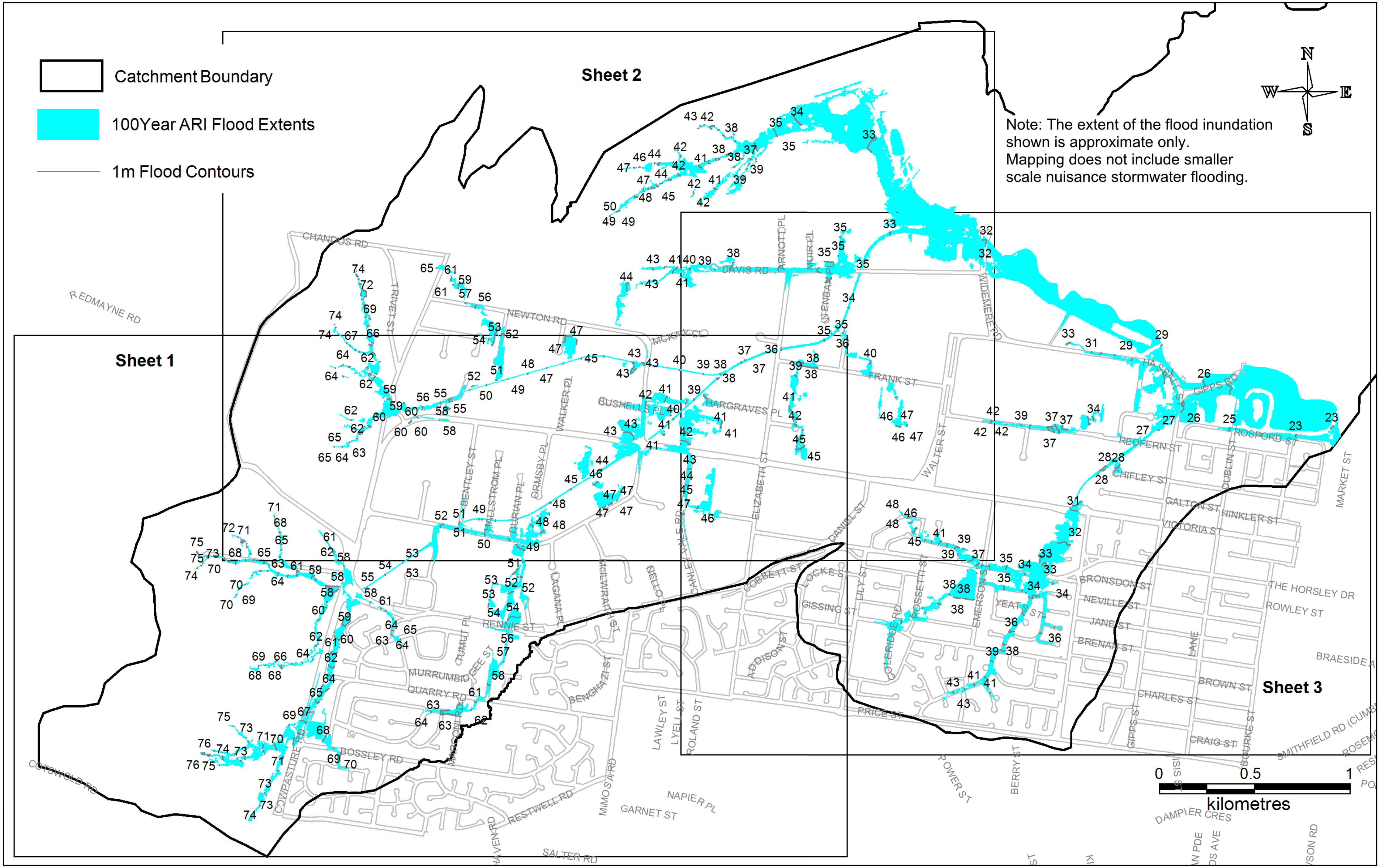

Figure D16 100Year ARI Flood Level Contours

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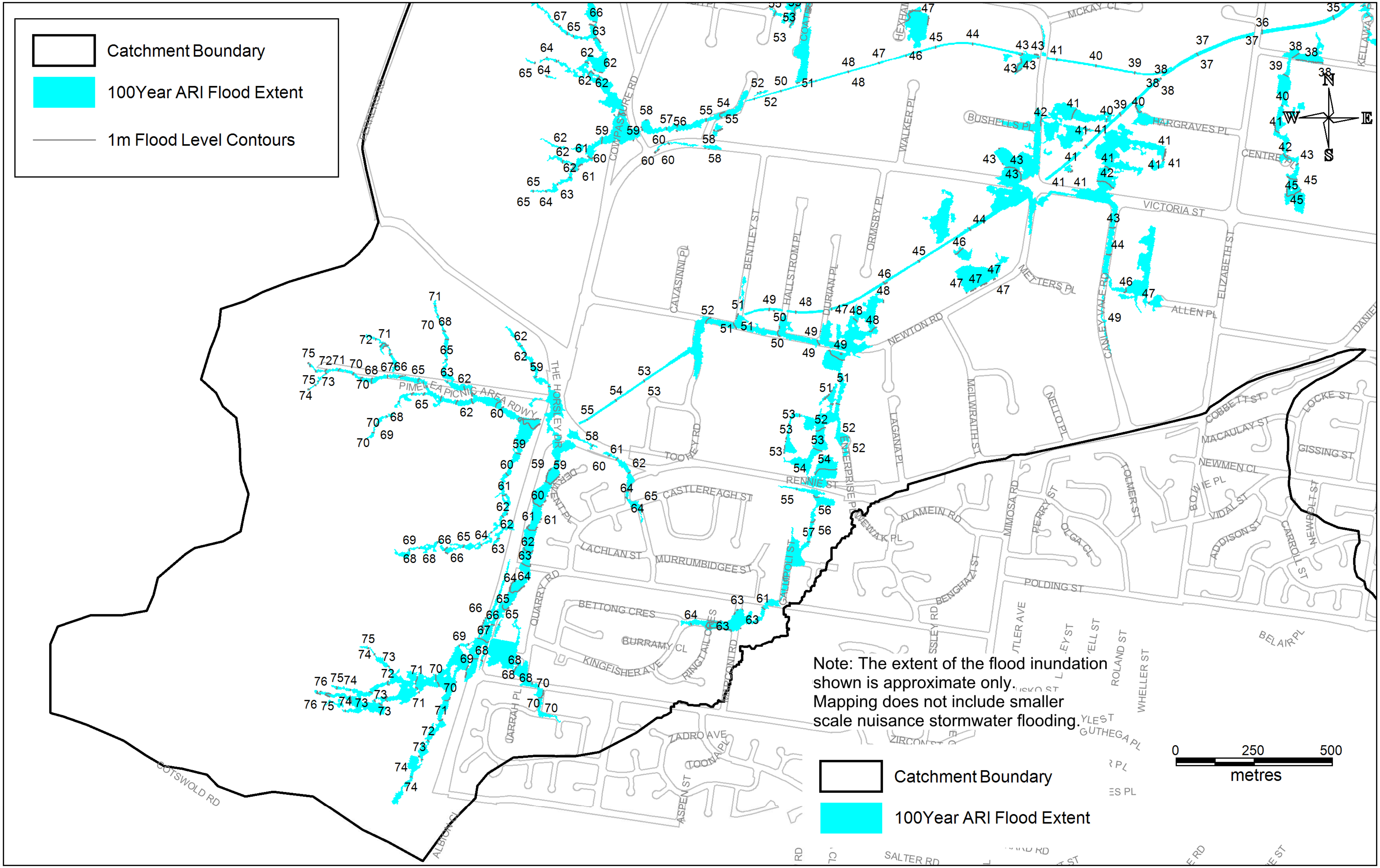

Figure D17 100Year ARI Flood Level Contours
Sheet 1

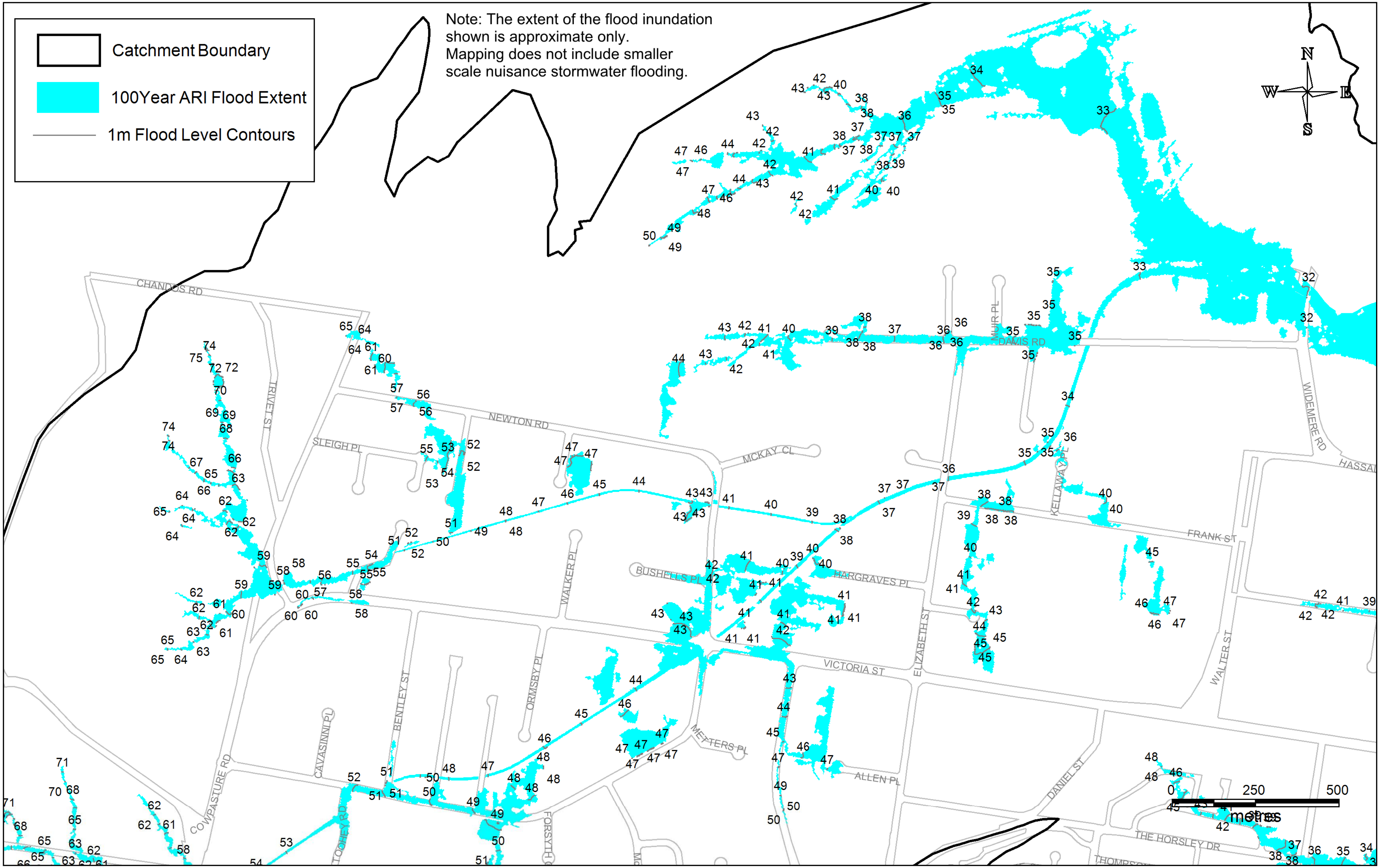

Figure D18 100Year ARI Flood Level Contours
Sheet 2

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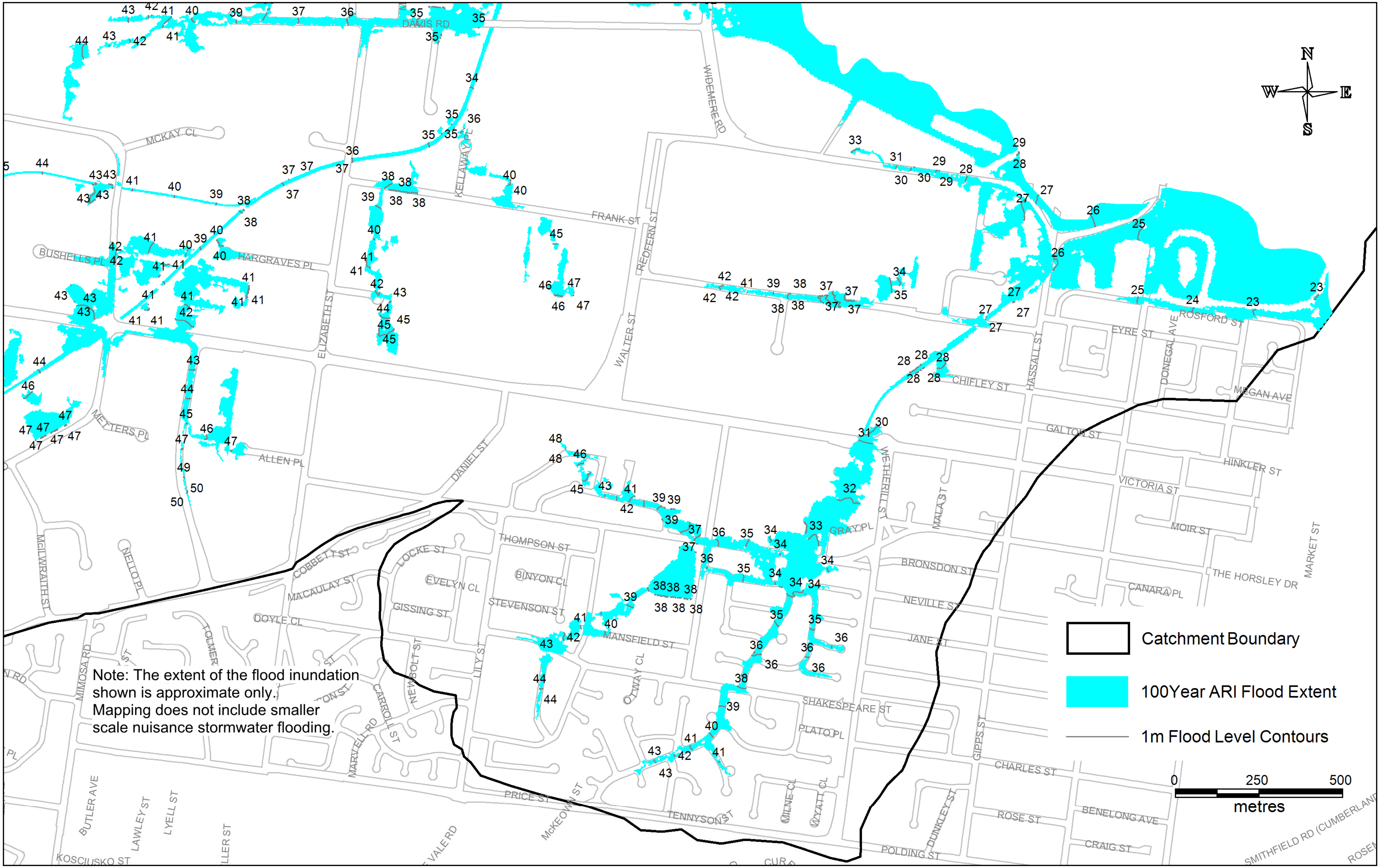

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Figure D19 100Year ARI Flood Level Contours Sheet 3

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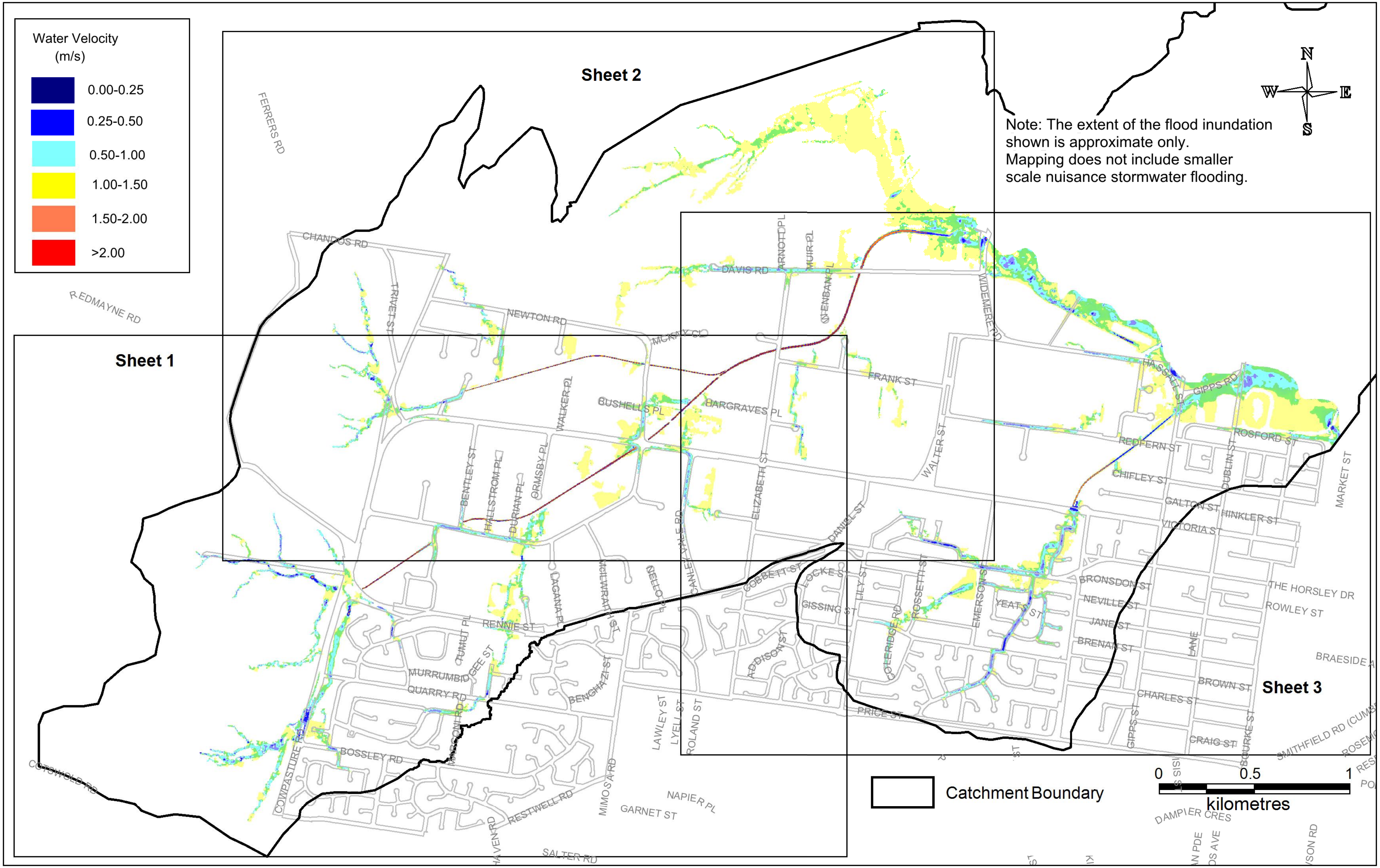

Figure D20 100Year ARI Flood Velocity

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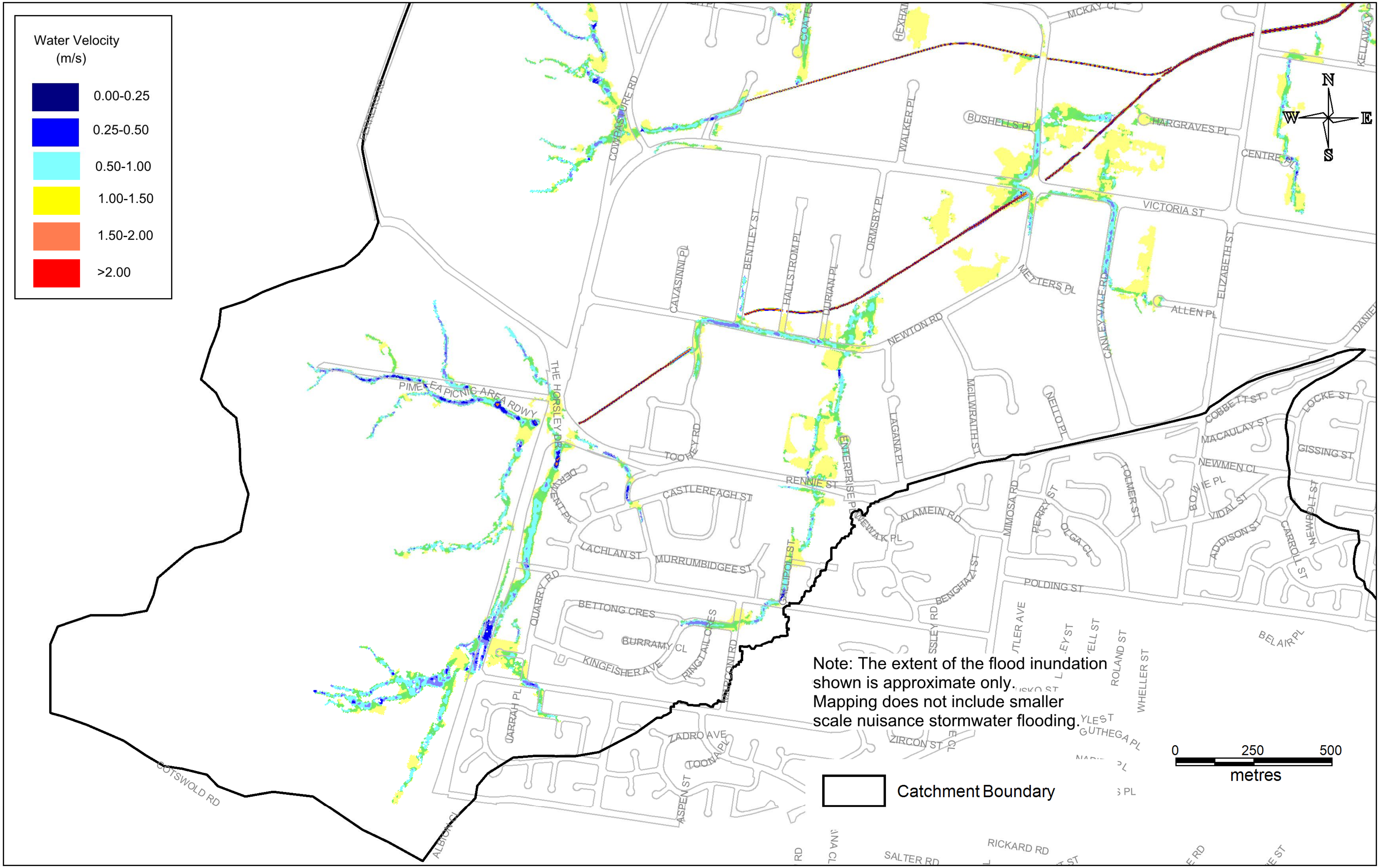

Figure 21 100Year ARI Flood Velocity
Sheet 1

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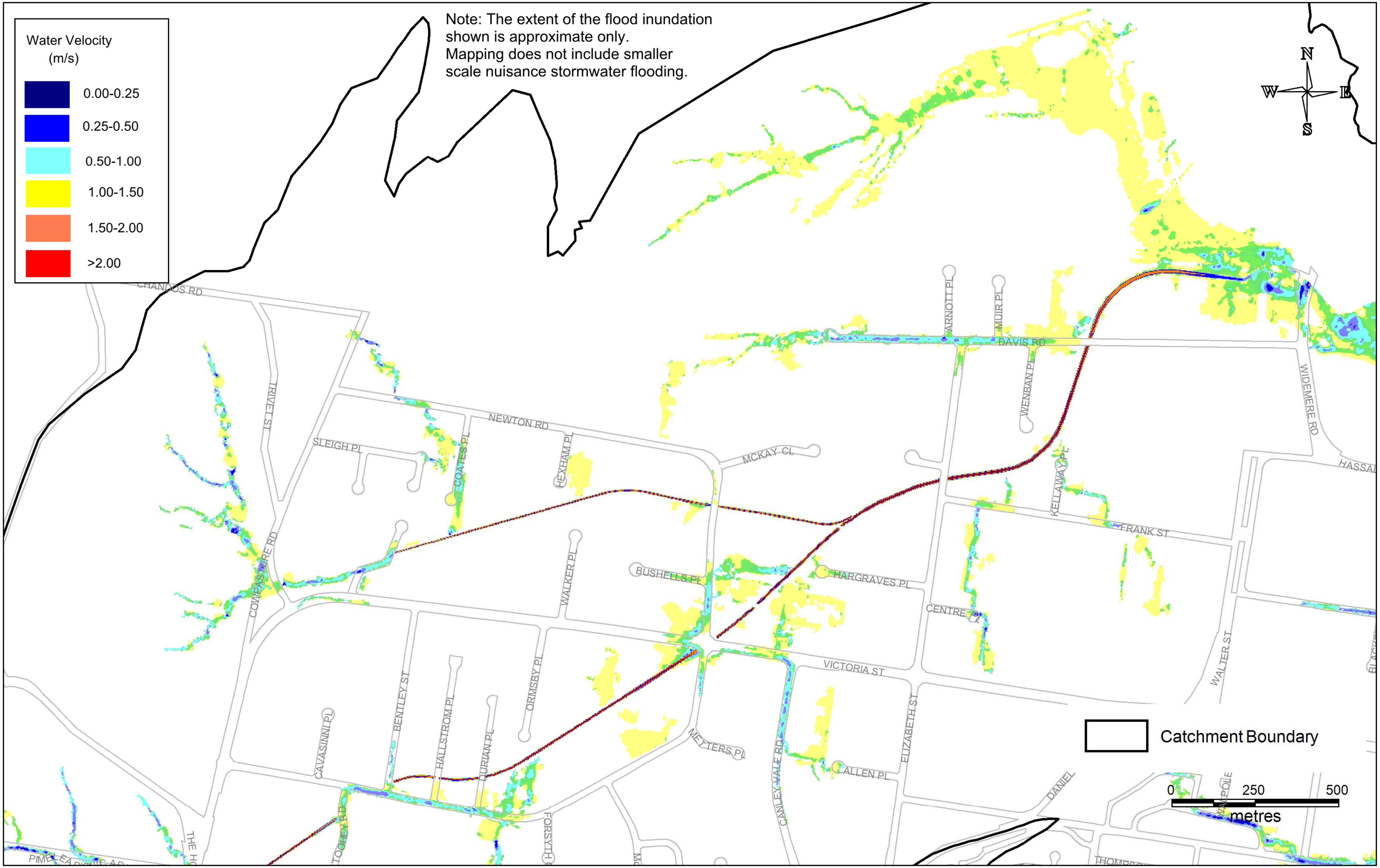

Figure D22 100Year ARI Flood Velocity
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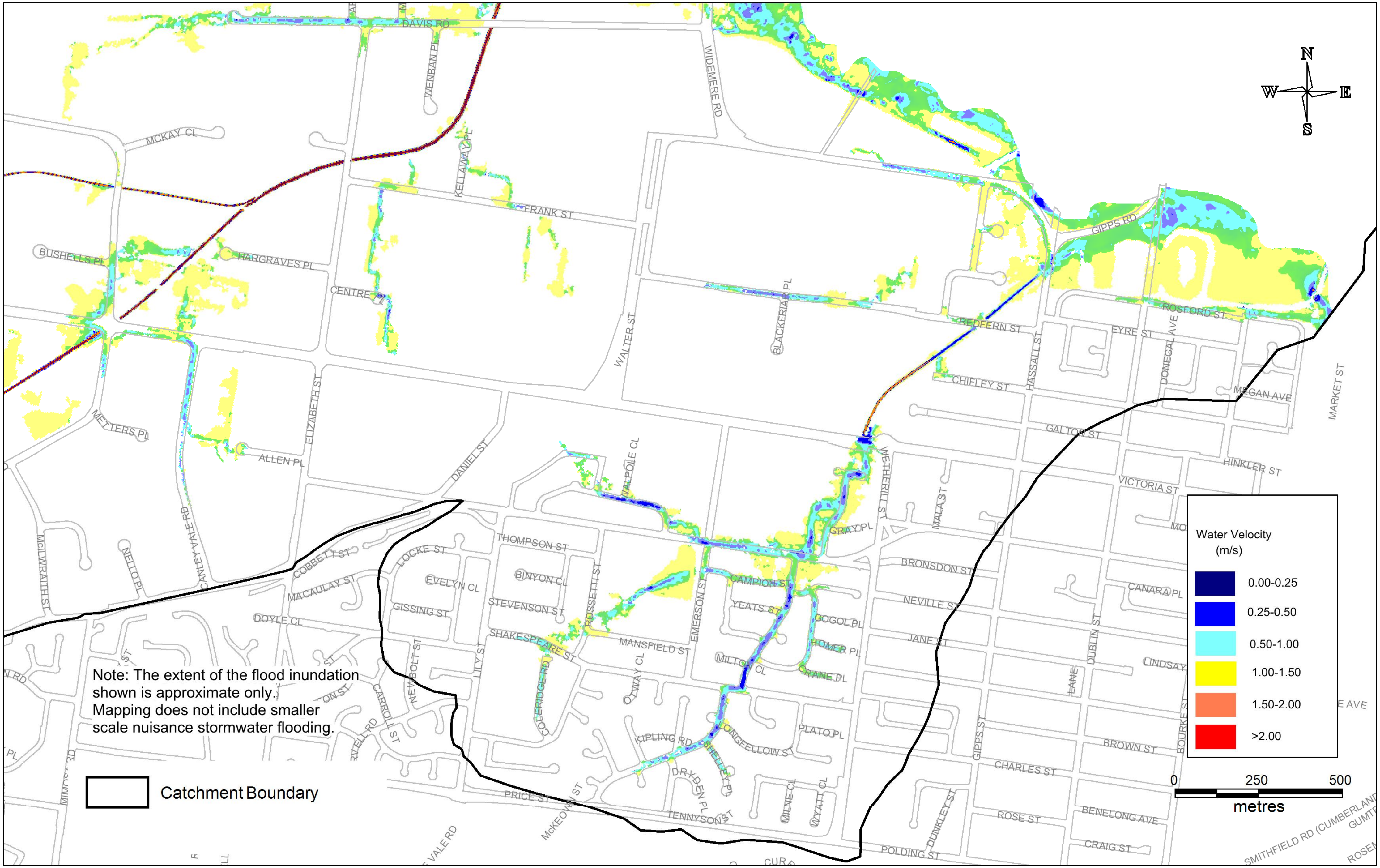

Figure D23 100Year ARI Flood Velocity
Sheet 3

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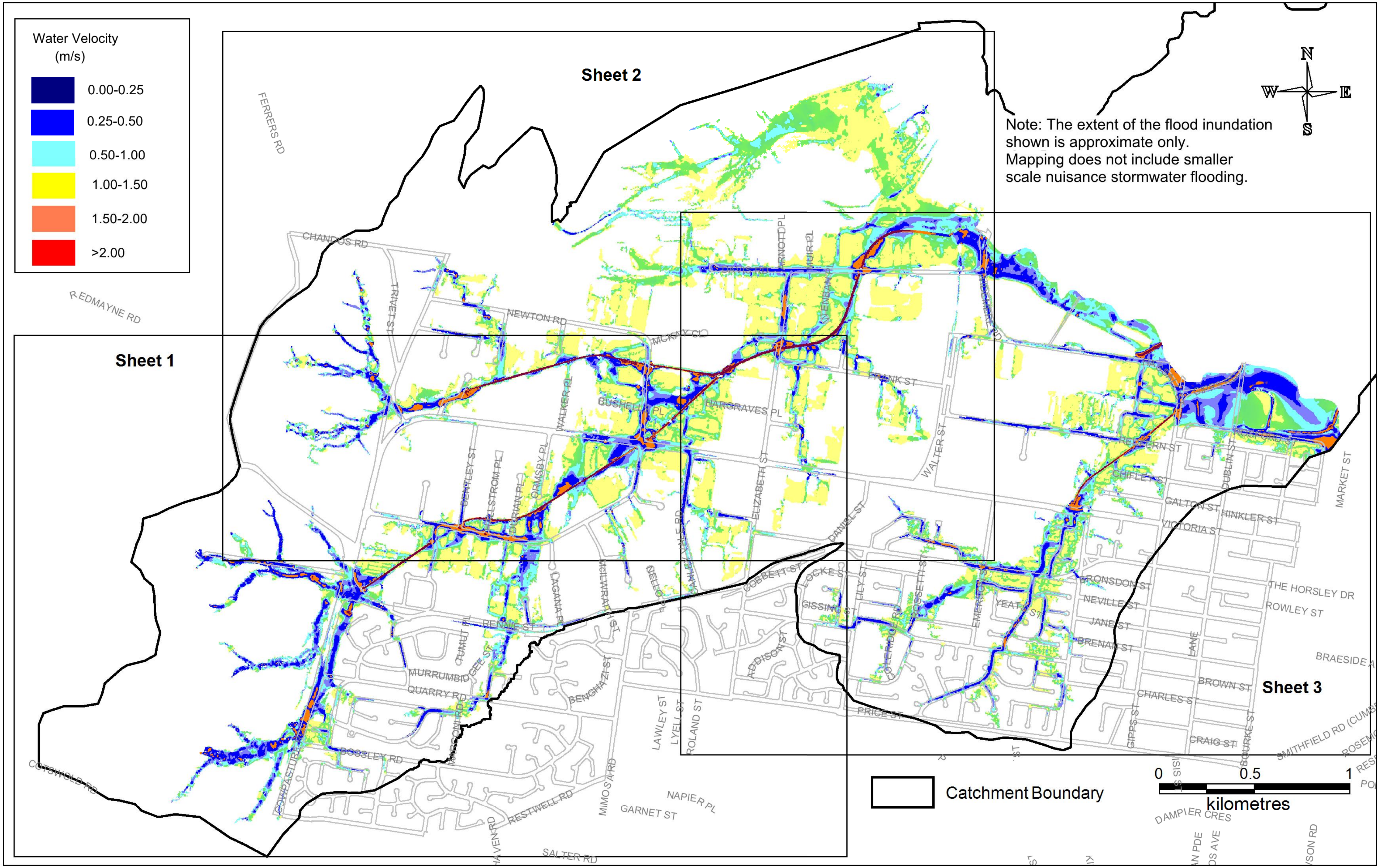

Figure D24 PMF Flood Velocity

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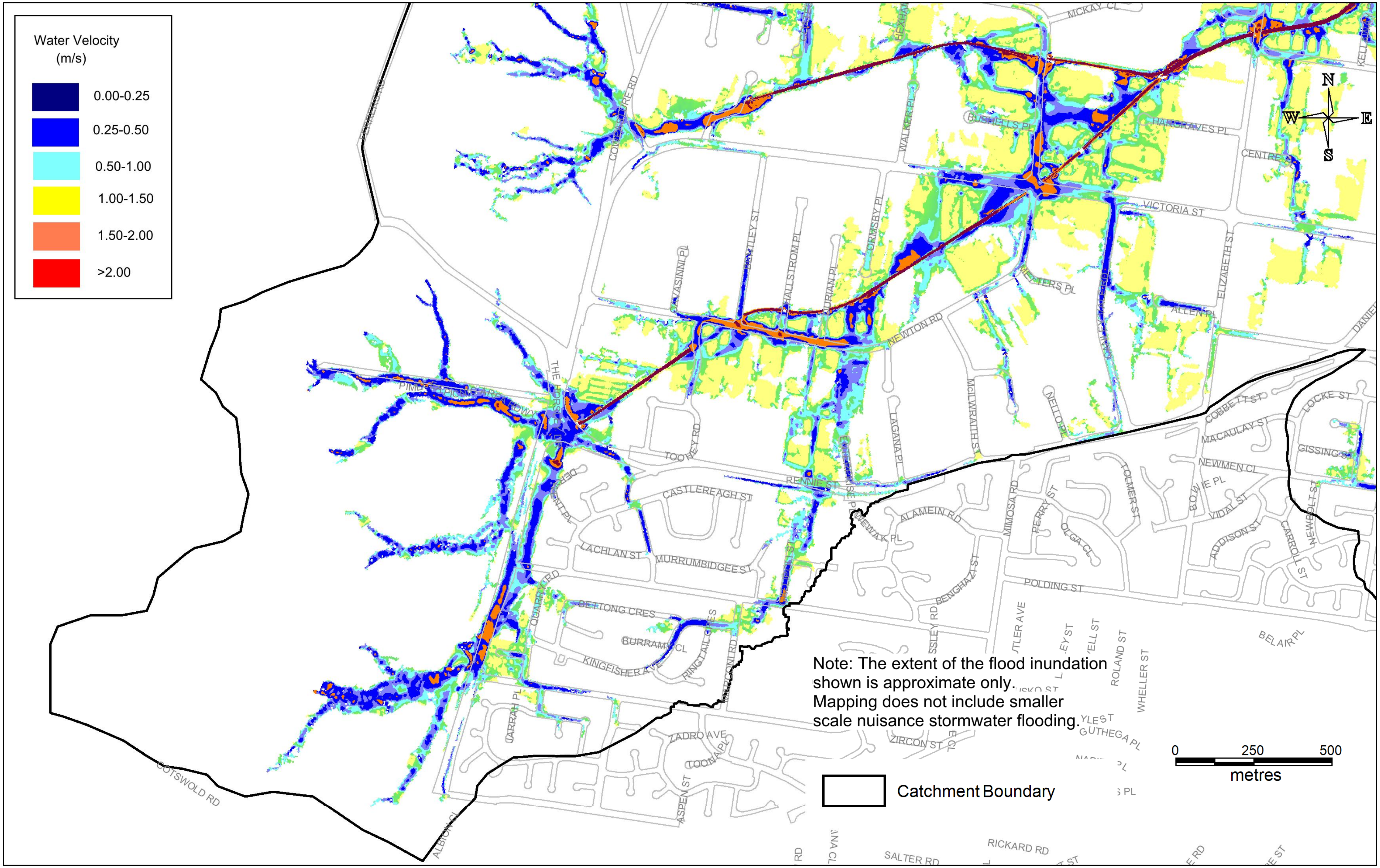

Figure 25 PMF Flood Velocity
Sheet 1

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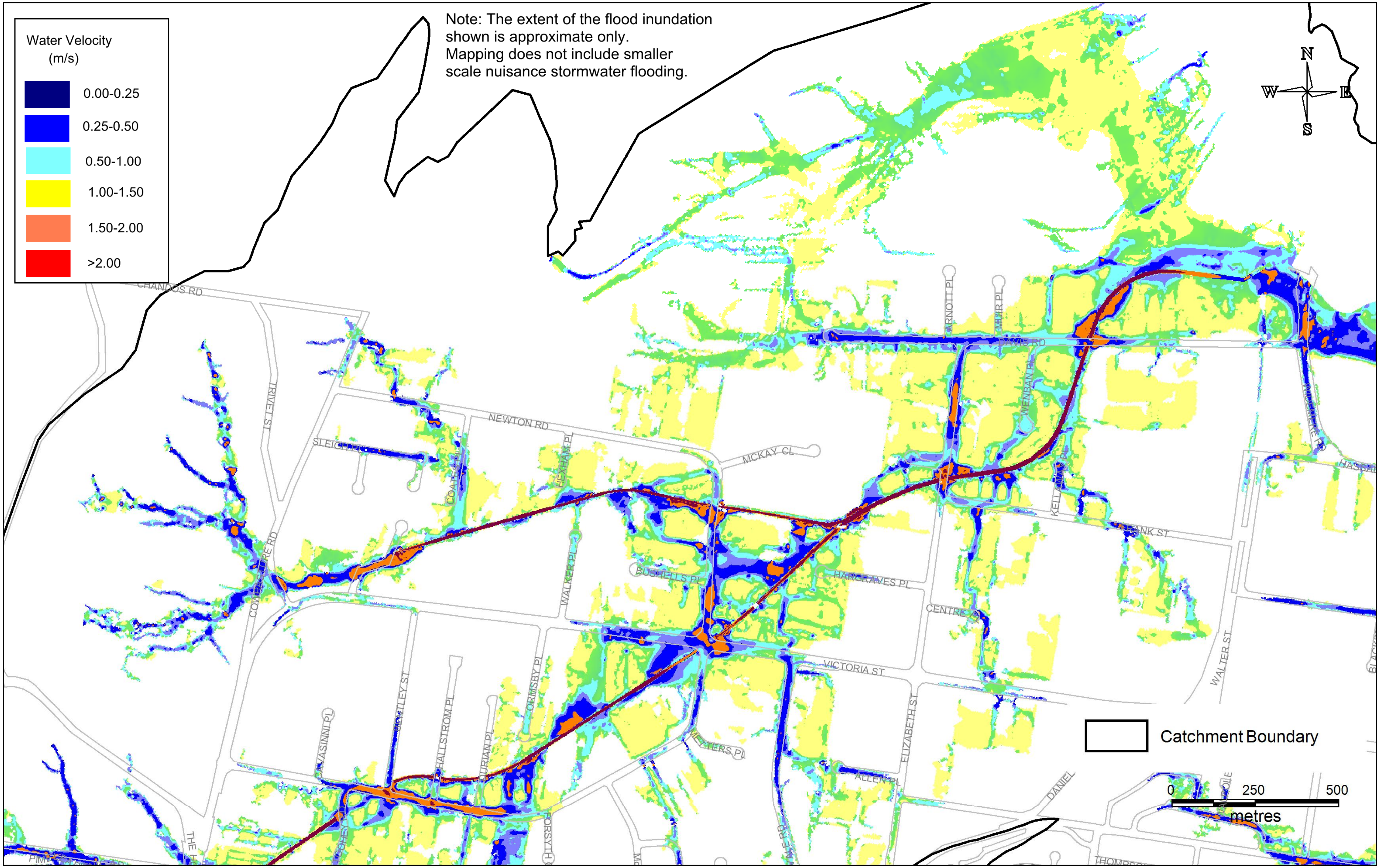

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Figure D26 PMF Flood Velocity
Sheet 2

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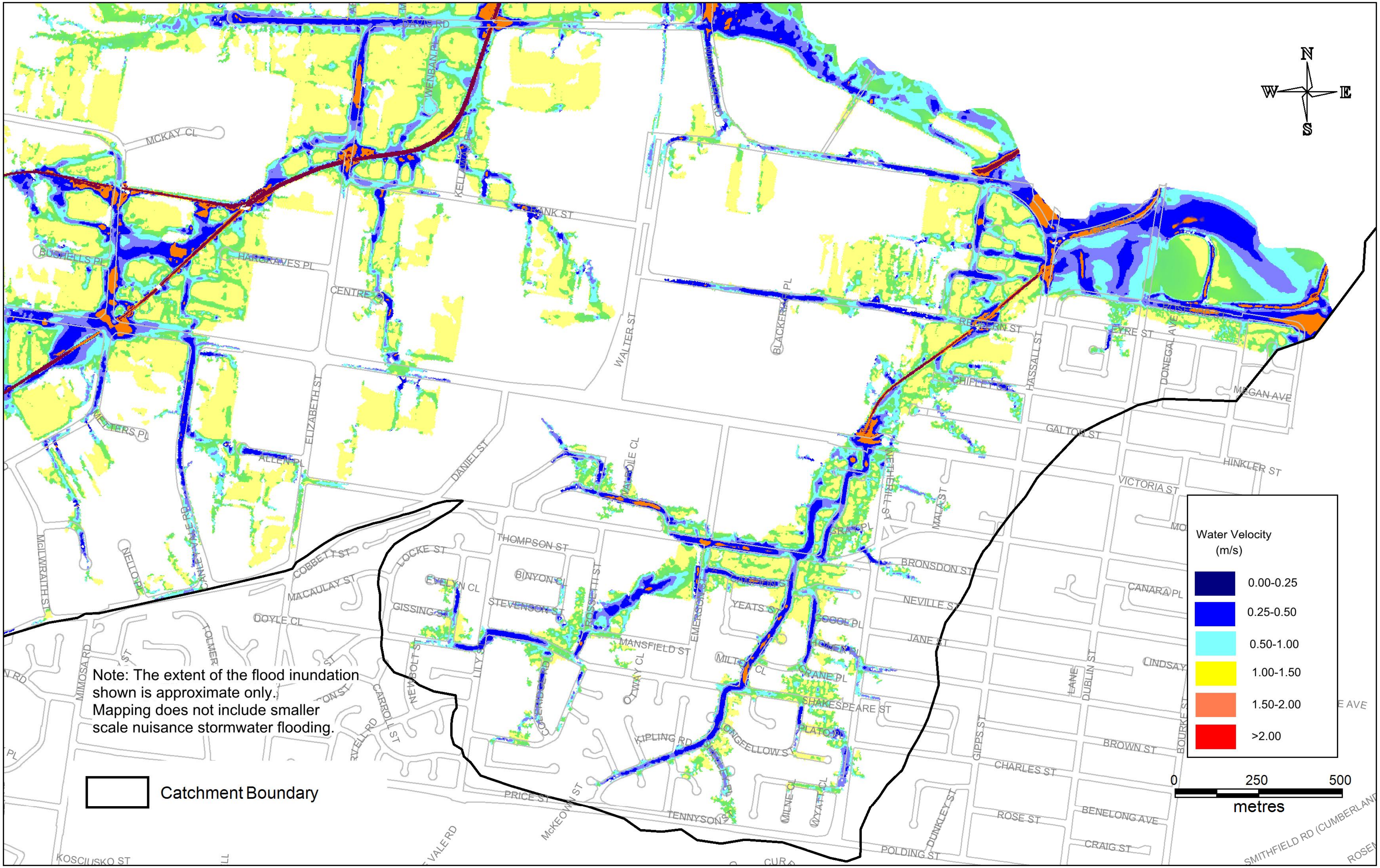

Figure D27 PMF Flood Velocity
Sheet 3

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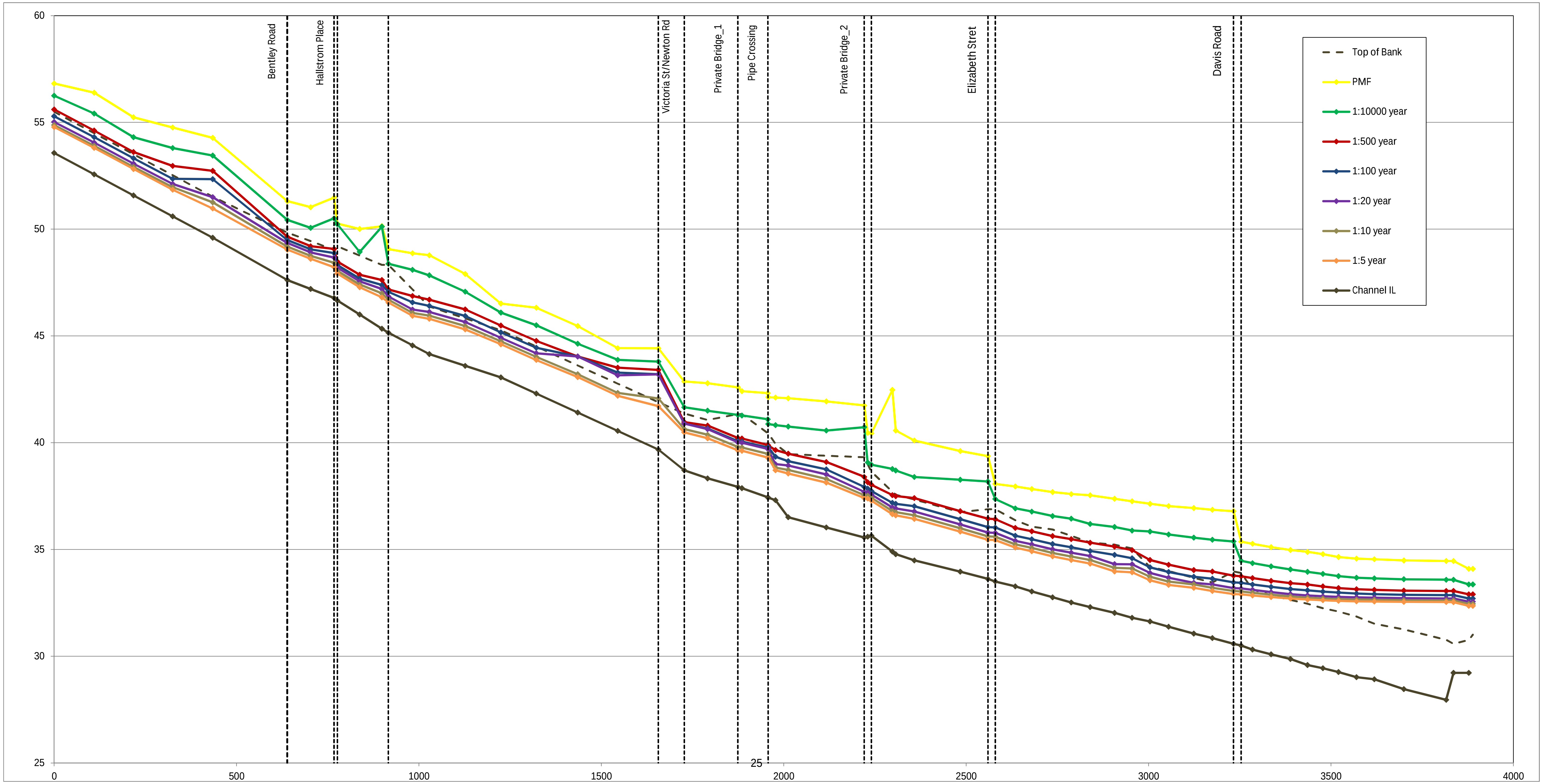

Figure D28 Design Flood Levels - Main Channel Long Profile

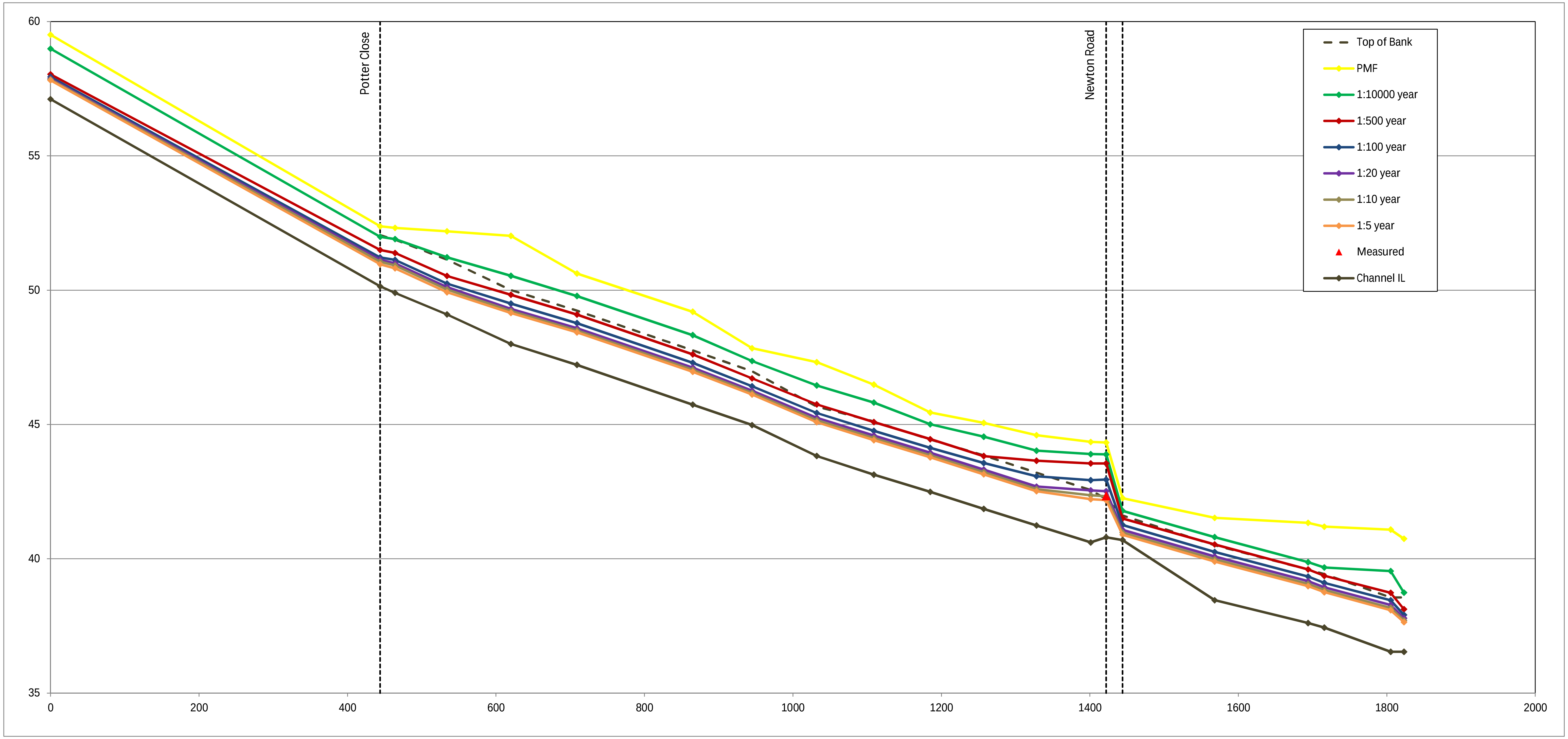

Figure 4-2 Design Flood Levels – Tributary 1 Long Profile

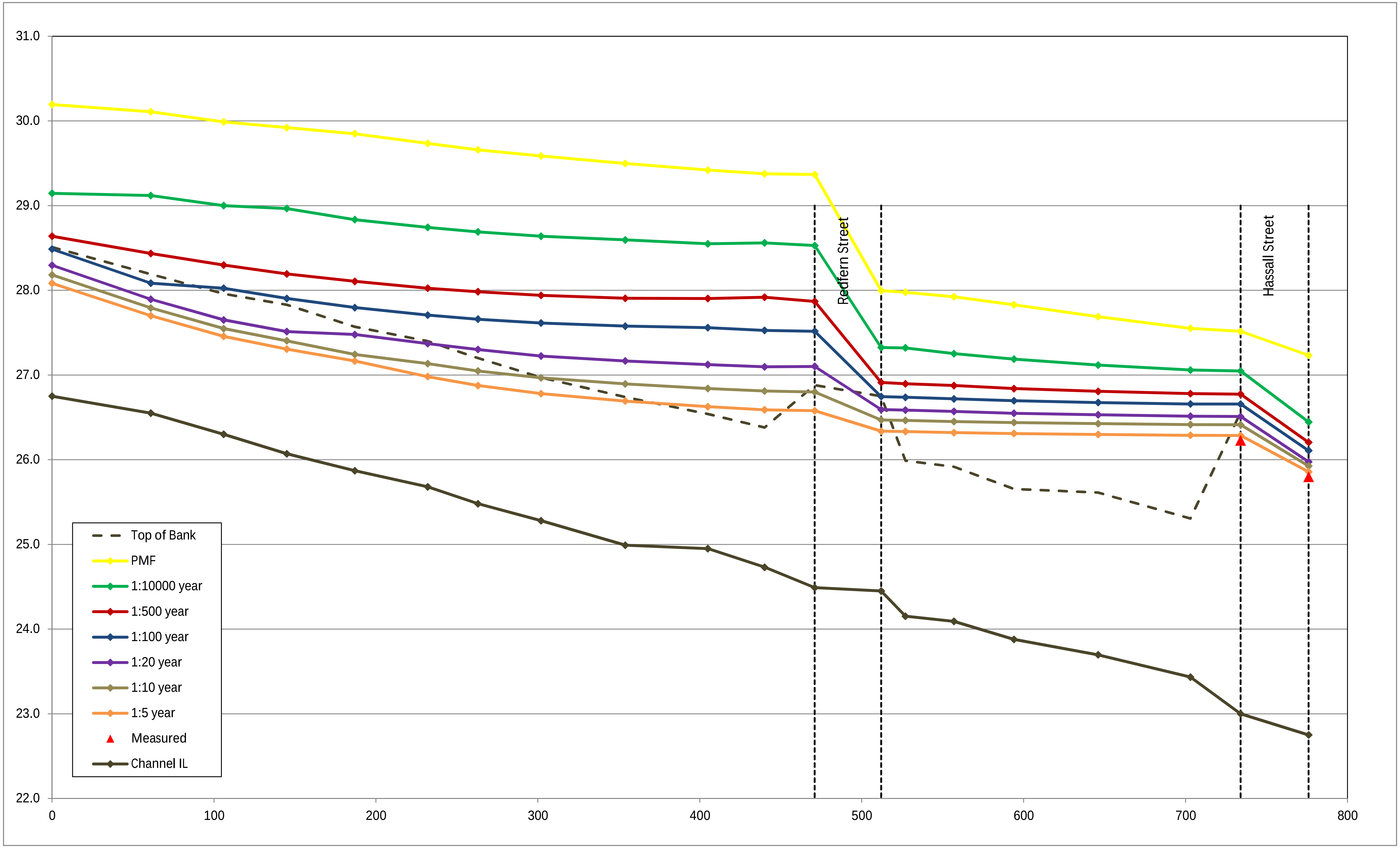

Figure 4-3 Design Flood Levels - Rosford Channel Long Profile

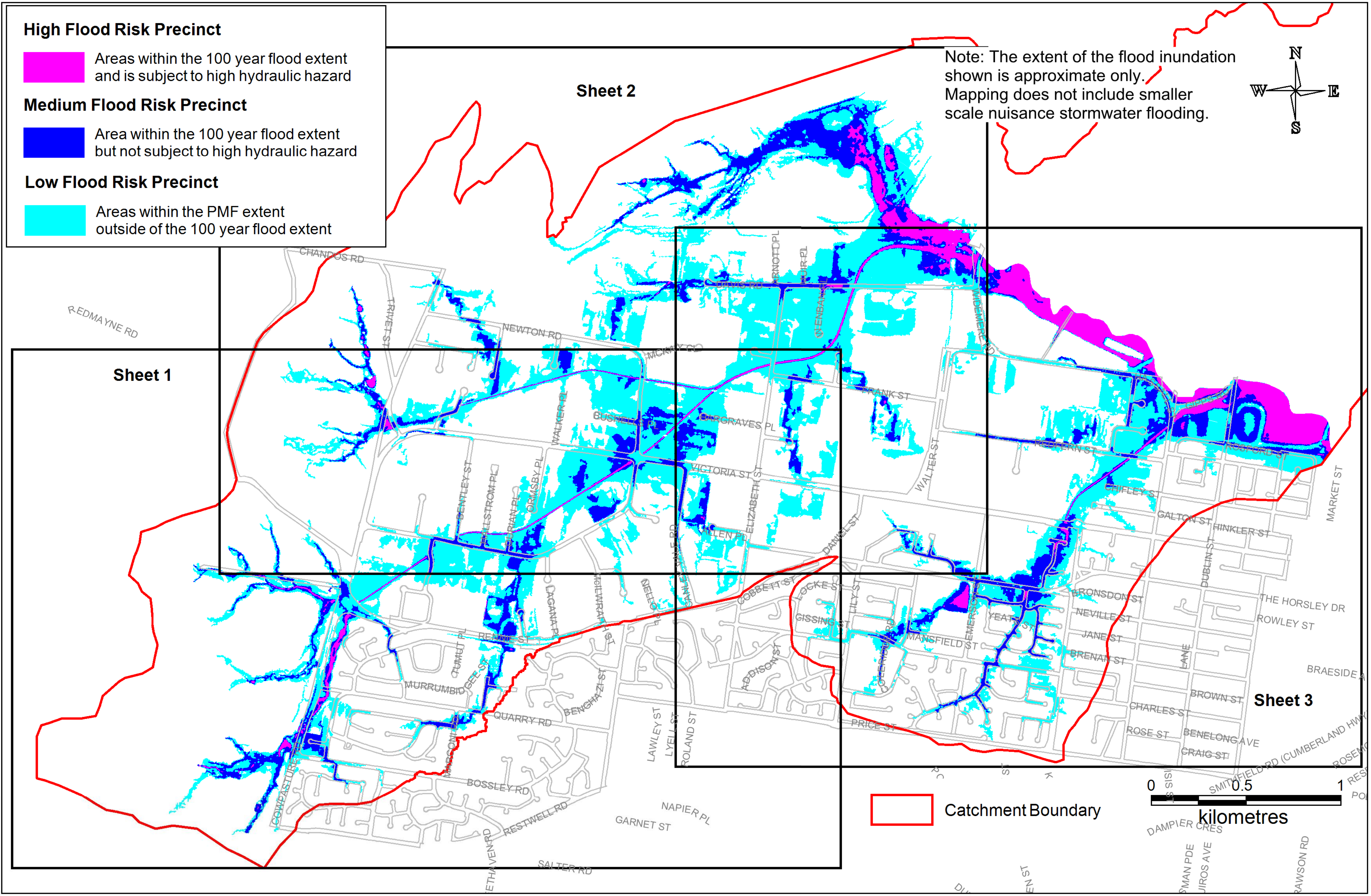

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Figure D31 Provisional Flood Risk Precincts

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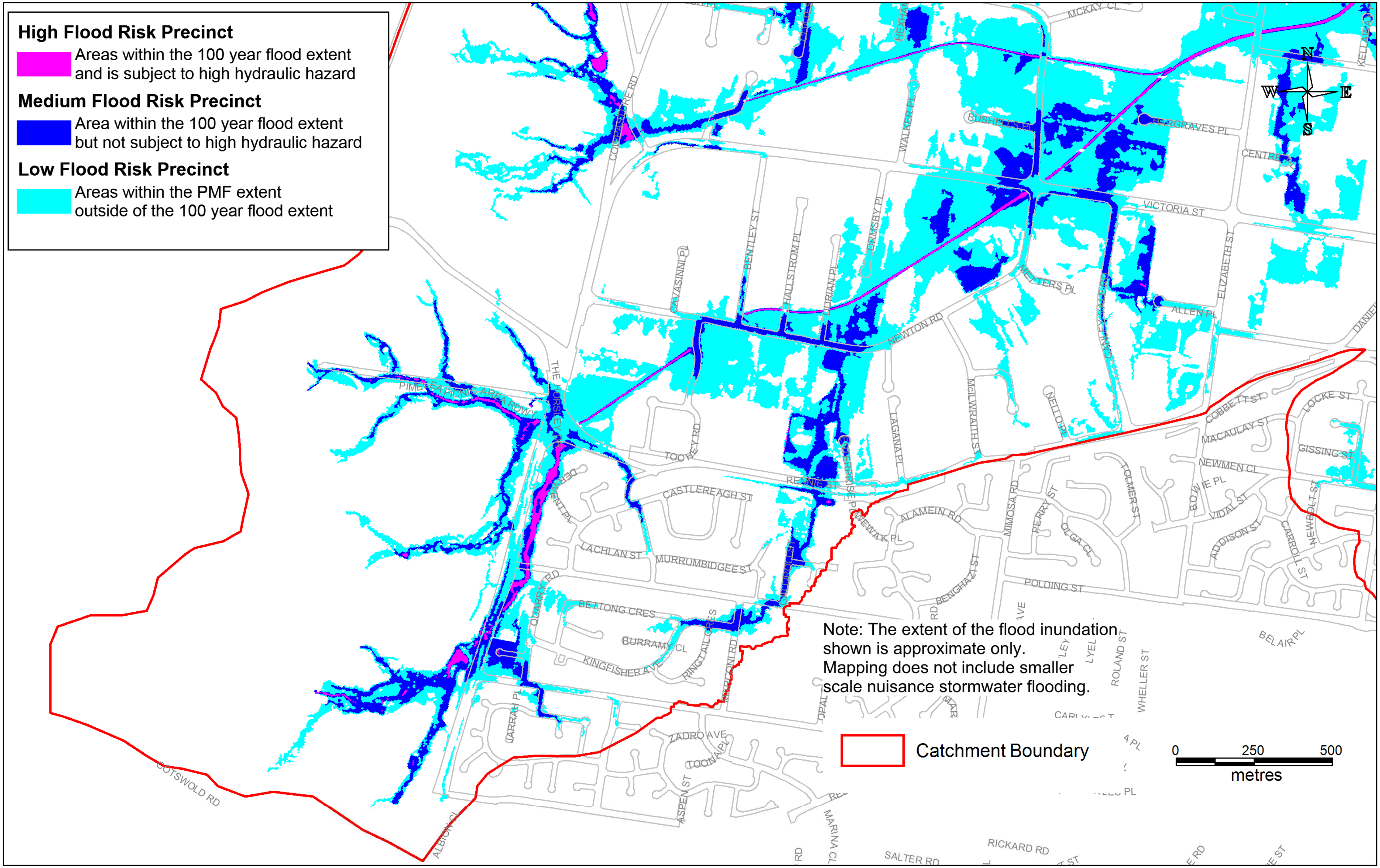

Figure D32 Provisional Flood Risk Precincts
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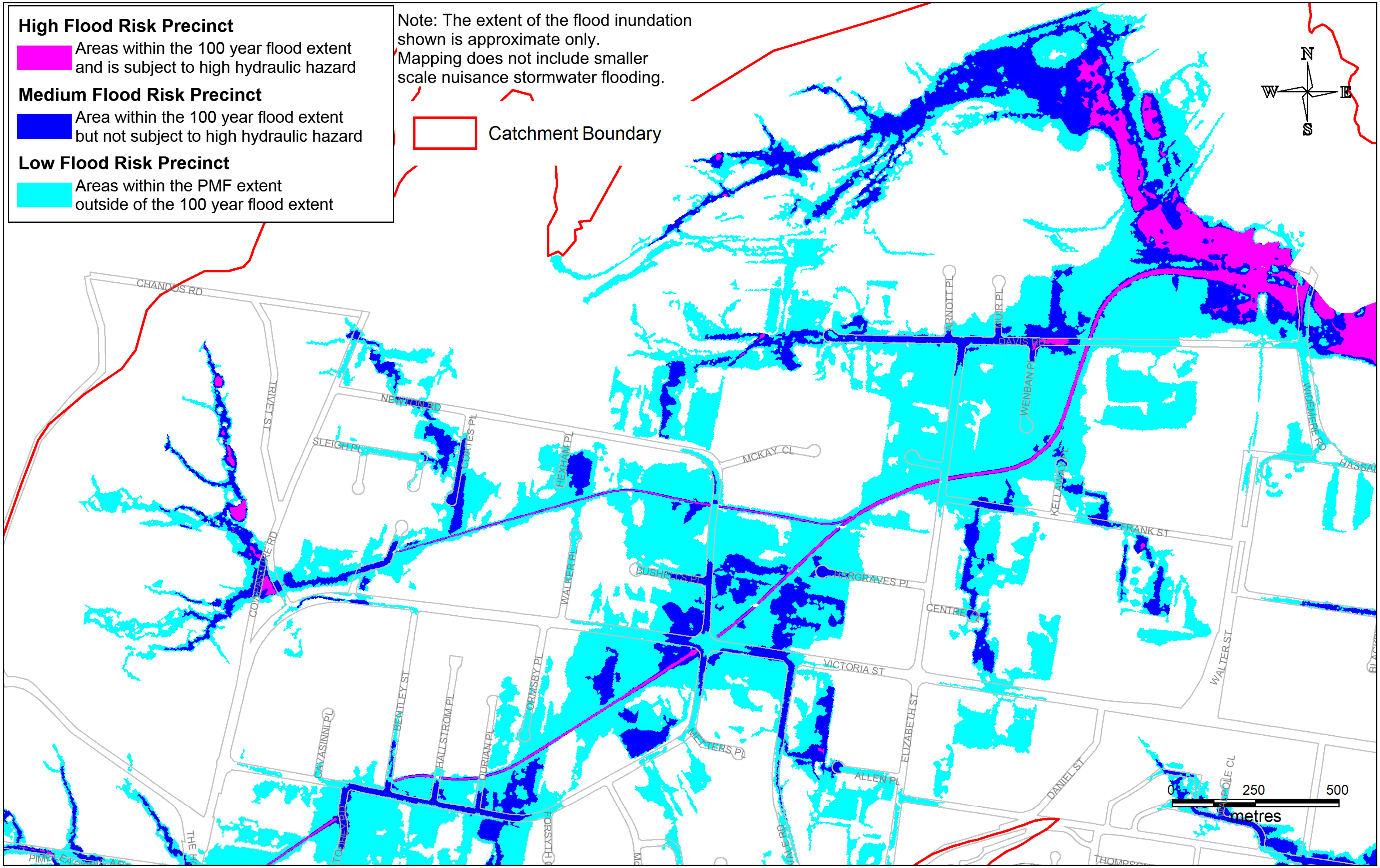

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Figure D33 Provisional Flood Risk Precincts Sheet 2

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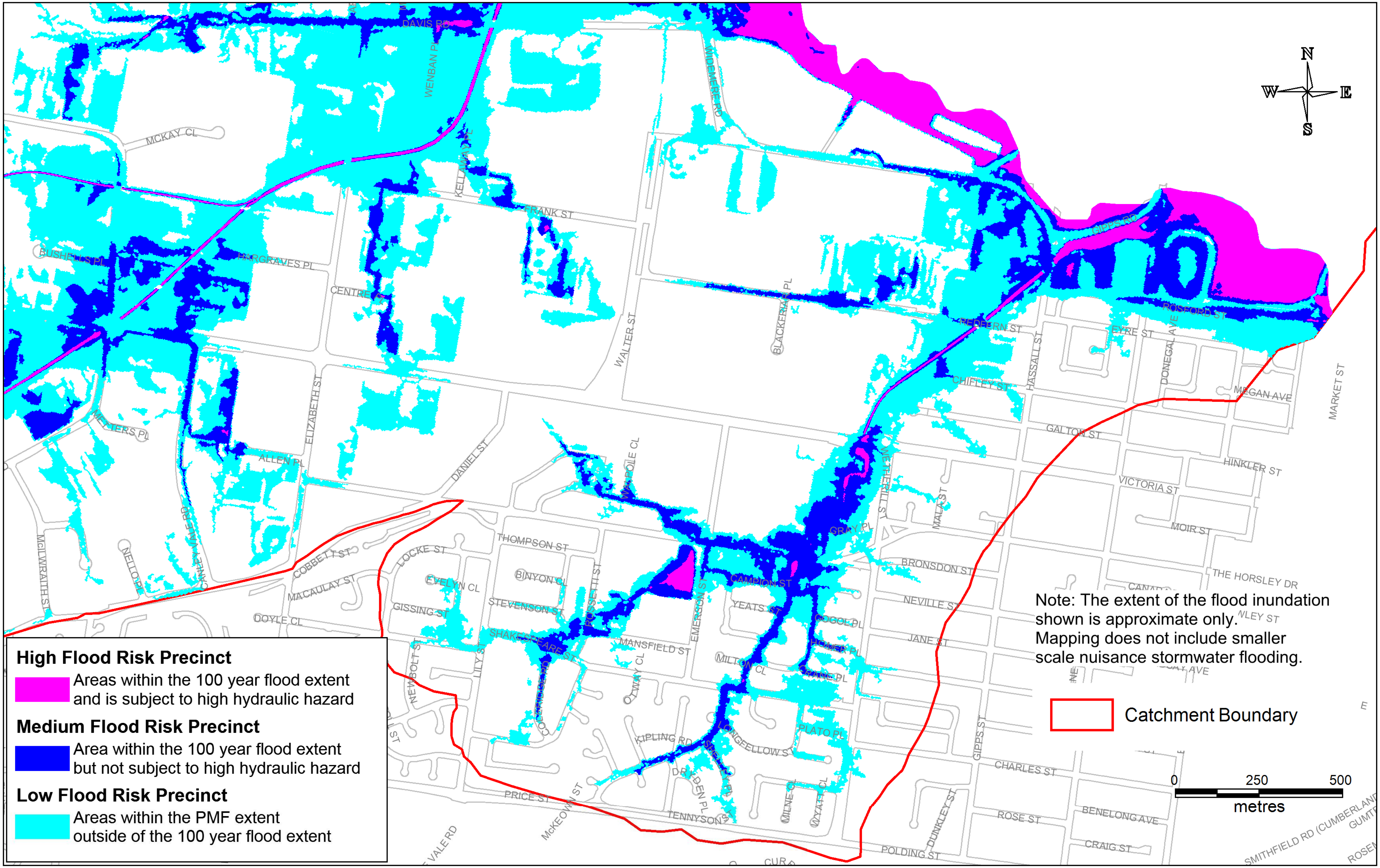

Figure D34 Provisional Flood Risk Precincts
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Wetherill Park Catchment Management Plan

Glossary

Term	Description
Annual Exceedance Probability (AEP)	Term used to describe the chance of a flood of a given or larger size occurring in any one year, expressed as a percentage. Eg. a 1% AEP flood means there is a 1% (ie. one-in-100) chance of a flood of that size or larger occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national plain of level corresponding approximately to mean sea level. All flood levels, floor levels and ground levels are normally provided in metres AHD (m AHD)
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.
catchment	A catchment is the area of land from which rainwater drains into a common point such as a reservoir, pond, lake, river or creek. In urban areas such as Fairfield, the majority of the rainwater is collected by gutters and pipes and then flows through stormwater drains into the stormwater system.
conveyance	A direct measure of the flow carrying capacity of a particular cross-section of a stream or stormwater channel. (For example, if the conveyance of a channel cross-section is reduced by half, then the flow carrying capacity of that channel cross-section will also be halved).
discharge	The rate of flow of water measured in terms of volume per unit time, eg. cubic metres per second (m^3/s). Also known as **flow**. Discharge is different from the speed/velocity of flow which is a measure of how fast the water is moving.
extreme flood	An estimate of the probable maximum flood, which is the largest flood likely to ever occur.
flood	A 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 as defined by the FDM before entering a watercourse.
flood awareness	An appreciation of the likely effects of flooding and a knowledge of the relevant flood warning and evacuation procedures.
flood hazard	The potential for damage to property or harm to persons during a flood or a situation with a potential to cause loss. In relation to this study, the hazard is flooding which has the potential to cause harm or loss to the community. Flood hazard is a key tool used to determine flood severity and is used for assessing the suitability of future types of land use.
flood level	The height of the flood described as either a depth of water above a particular location (eg. 1m above floor level) or as a depth of water related to a standard level such as Australian Height Datum (eg. flood level is 5m AHD).
flood liable/flood prone land	Land susceptible to flooding up to the PMF. The term flood liable or flood prone land covers the entire floodplain.
floodplain	The area of land that is subject to inundation by floods up to and including the PMF event.

Term	Description
Floodplain Development Manual (FDM)	Refers to the document dated April 2005, published by the New South Wales Government and entitled "Floodplain Development Manual: the management of flood liable land".
Floodplain Risk Management Plan (FRMP)	A plan prepared for one or more floodplains in accordance with the requirements of the FDM or its predecessors.
Floodplain Risk Management Study (FRMS)	A study prepared for one or more floodplains in accordance with the requirements of the FDM or its predecessors.
flood risk	The chance of something happening that will have an impact. It is measured in terms of consequences and probability (likelihood). In the context of this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
flood risk precinct	An area of land with similar flood risks and where similar development controls may be applied by a Council to manage the flood risk. The flood risk is determined based on the existing development in the precinct or assuming the precinct is developed with normal residential uses. Usually the floodplain is categorised into three flood risk precincts 'low', 'medium' and 'high', although other classifications can sometimes be used. *High Flood Risk: T*his has been defined as the area of land below the 100-year flood event that is either subject to a high hydraulic hazard or where there are significant evacuation difficulties. *Medium Flood Risk:* This has been defined as land below the 100-year flood level that is not within a high flood risk precinct. This is land that is not subject to a high hydraulic hazard or where there are no significant evacuation difficulties. *Low Flood Risk:* This has been defined as all land within the floodplain (i.e. within the extent of the probable maximum flood) but not identified within either a high flood risk or a medium flood risk precinct. The low flood risk precinct is that area above the 100-year flood event.
flood study	A study that investigates flood behaviour, including identification of flood extents, flood levels and flood velocities for a range of flood events.
hydraulics	The study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydraulic hazard	The hazard as determined by the provisional criteria outlined in the FDM in a 100 year flood event.
hydrology	The study of rainfall and runoff process; in particular, the evaluation of peak discharges, flow volumes and the derivation of hydrographs (graphs that show how the discharge or stage/flood level at any particular location varies with time during a flood).
local drainage	Term given to small scale inundation in urban areas outside the definition of major drainage as defined in the FDM. Local drainage problem invariably involve shallow depths (less than 0.3m) with generally little danger to personal safety.
local overland flooding	The inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
mainstream flooding	The inundation of normally dry land occurring when water overflows

Term	Description
	the natural or artificial banks of a stream, river, estuary, lake or dam.
overland flow path	The path that floodwaters can follow if they leave the confines of the main flow channel or pipe system. Overland flow paths can occur through private properties or along roads.
peak discharge	The maximum discharge or flow during a flood measured in cubic metres per second (m^3/s).
probable maximum flood (PMF)	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation.
probable maximum precipitation (PMP)	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 the estimation of the probable maximum flood.
probability	A statistical measure of the expected chance of flooding (see ARI).
risk	See *flood risk.*
runoff	The amount of rainfall that ends up as flow in a stream. Also known as rainfall excess.
velocity	The term used to describe the speed of floodwaters, usually in metres per second (m/s).
water level	See *flood level.*
water surface profile	A graph showing the height of the flood (ie. water level or flood level) at any given location along a watercourse at a particular time.
zone of significant flow	The area of the floodplain where a significant discharge of water occurs during floods. Should the area within this boundary be fully or partially blocked, a significant distribution of flood flows or increase in flood levels would occur.