LAKE MACQUARIE CITY COUNCIL

NORTH CREEK WARNERS BAY FLOOD STUDY

DRAFT FINAL REPORT

NOVEMBER 2024

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Project **North Creek Warners Bay Flood Study**			**Project Number** **123045**		
Client Lake Macquarie City Council			**Client's Representative** Greg D Jones		
Project Manager Michael Reeves					
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Cover photo: North Creek channel upstream of Walker Street, Warners Bay (October 2023)

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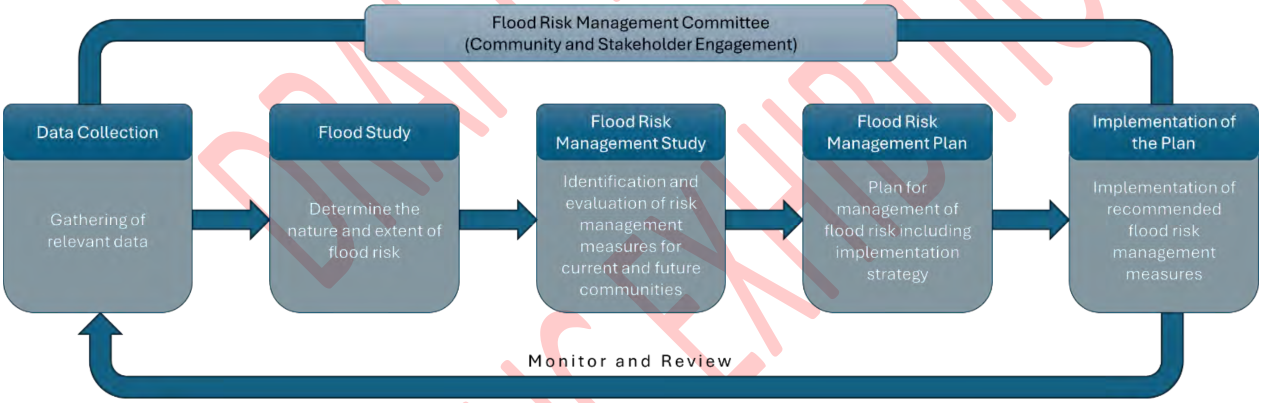
FOREWORD

The NSW State Government's Flood Prone Land Policy, contained in the Flood Risk Management Manual (NSW Department of Planning and Environment 2023), provides a framework to ensure the sustainable use of floodplain environments. The Policy is specifically structured to provide solutions to existing flooding problems in rural and urban areas. In addition, the Policy provides a means of ensuring that any new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

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

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



This document constitutes the first and second stages of the management process for the North Creek catchment. It presents a compilation of the data collected and has defined flood behaviour and flood risk for the catchment area.

This study was commissioned under the 2005 NSW Floodplain Development Manual (Reference 1), however, it is recognised that the 2023 Flood Risk Management Manual (Reference 2) was gazetted shortly after the project commenced. While the study was undertaken in accordance with the 2005 manual, there are elements that are consistent to both the 2005 and 2023 manuals. Where appropriate, the 2023 manual is referenced where project methodology or outputs are consistent with the new 2023 manual.

ACKNOWLEDGEMENTS

This study was undertaken by WMAwater Pty Ltd, on behalf of Lake Macquarie City Council. Lake Macquarie City Council has prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. The document does not necessarily represent the opinions of the NSW Government or the Department of Climate Change, Energy, the Environment and Water.

A number of organisations and individuals have contributed both time and valuable information to this study. The assistance of the following in providing data and/or guidance to the study is gratefully acknowledged:

- Lake Macquarie Coastal Zone Management Committee
- Residents of the study area
- Lake Macquarie City Council
- Department of Climate Change, Energy, the Environment and Water
- NSW State Emergency Service

EXECUTIVE SUMMARY

Introduction

Lake Macquarie City Council engaged WMAwater to undertake the North Creek Warners Bay Flood Study. The study defines the existing flood behaviour within the North Creek catchment using computer models. The objective of this study is to improve understanding of flood behaviour and impacts, and better inform management of flood risk in the study area. It aims to provide an understanding of the full range of flood behaviour and consequences in the study area.

The study area comprises the entire North Creek catchment, located on the northern hinterland of Lake Macquarie. The catchment covers an area of approximately 5.3 km^2 and includes the suburb of Lakelands, the majority of the Warners Bay suburb and a small portion of the Speers Point suburb. The catchment is largely developed for residential and commercial/light industrial purposes. Whie North Creek is the primary waterway, there are a number of unnamed tributaries that traverse the catchment and join North Creek upstream of its outlet to Lake Macquarie at Warners Bay.

The study area can be affected by local overland flows, North Creek and its tributaries and elevated water levels in Lake Macquarie. This study investigates flooding due to overland flows and creeks and provides a significant update on the previous North Creek Flood Study (Webb, McKeown & Associates 2005).

Available Data

As part of the data collection, WMAwater received the previous studies undertaken and models developed by Lake Macquarie Council. Additional data received included a detailed LiDAR dataset captured in 2018 and GIS datasets of stormwater assets, buildings, land use zoning and aerial imagery. Detailed topographic survey of hydraulic structures was available from previous studies. Historic rainfall and water level data was also obtained from the Bureau of Meteorology, Hunter Water and Manly Hydraulics Laboratory.

Community Consultation

At the commencement of the project, the community were informed of the study and provided the opportunity to contribute their observations of flooding within the catchment. A total of 11 responses to a community questionnaire were obtained, with only 2 of these indicating they had experienced flooding in the past.

These respondents described overland flow flood behaviour (i.e. the flood waters were not confined to a defined creek or channel). One respondent described inundation from North Creek at a property that was not their own.

Model Development

The models developed to simulate flood behaviour in the study area consist of a two-stage process:

1. Hydrologic modelling using WBNM to convert rainfall to runoff
2. Hydraulic modelling using TUFLOW to estimate overland flow distributions, flood depths,

levels and velocities.

The WBNM hydrologic model consisted of 245 sub-catchments and covered the entire North Creek catchment. Sub-catchments were delineated to trapped low points, stormwater infrastructure or flow paths. Sub-catchments were assigned an impervious fraction and a catchment lag factor was adopted from other studies in the Lake Macquarie catchment.

The TUFLOW hydraulic model covers the entire North Creek catchment study area. The model consists of a 1 m by 1 m regular grid. The best available terrain and structure data was incorporated into the model, along with model adjustments to ensure that hydraulic features (including gutters and channels) were adequately represented. The simulated runoff hydrographs from the WBNM model were applied to the TUFLOW model as inflows.

Model Calibration

There have been two significant flood events in the North Creek catchment with substantial records – February 1990 and June 2007. Both of these events consisted of widespread inundation from North Creek, followed by elevated levels in Lake Macquarie. Council held surveyed flood mark information for both of these events which were used to undertake model calibration. The simulated flood levels were compared with the recorded flood levels and model parameters were adjusted until a reasonable match was obtained. The North Creek water level gauge at Walker Street was only installed in June 2022 and as such has not captured a significant flood event. However, the gauge recorded elevated water levels in July 2022 which was used to validate the modelled water level with the recorded water level at the gauge.

Overall, the comparisons between the modelled flood behaviour and the observed flood behaviour indicate a reasonable match was achieved. In particular, the calibration to the available flood marks for the 1990 and 2007 events was generally good (within 0.15 m). Where flood levels were not matched, it was considered more likely that the flood mark was not reliable. The model generally replicates the observed flood behaviour in flood photographs. The match with the water level gauge upstream of Walker Street was considered reasonable given the difficulties of simulating a relatively small in-bank event.

Design Flood Modelling

Design flood modelling was undertaken in accordance with Australian Rainfall and Runoff 2019 guidelines, including adoption of design rainfalls from the Bureau of Meteorology simulation of an ensemble of temporal patterns to be run for each duration. Each design storm was simulated in the WBNM hydrologic and TUFLOW hydraulic models. The critical storm duration (duration that produces the highest flood level) was determined based on the mean of the 10 temporal patterns for each duration, and varied across the catchment from 20 minutes to 360 minutes (3 hours). A 60 minute or 90 minute storm was found to adequately represent the typical behaviour across the study area. The design flood events simulated were the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% annual exceedance probability (AEP) events. The Probable Maximum Flood (PMF) was also simulated in accordance with the Generalised Short Duration Method. The critical duration for the PMF was 30 minutes and 90 minutes. Design flood depths, levels, velocities, hydraulic hazard and hydraulic categories were mapped and are provided in Appendix C. Flood results were also tabulated and plotted for tributaries, basins and at key road crossings, with results presented in

Appendix D.

Sensitivity Analysis

A sensitivity analysis was undertaken for key modelling parameters by varying the adopted values and assessing the change in peak flood levels. Peak flood levels are relatively insensitive to changes throughout the catchment (generally within ±0.05 m for the scenarios tested), with North Creek in the vicinity of King Street being the most sensitive. Climate change sensitivity was also undertaken for both sea level rise (0.4 m and 0.9 m) and increase in rainfall intensity (comparing the 0.5% and 0.2% AEP events with the 1% AEP event). For sea level rise, 0.4 m increase in lake levels resulted in 0.02 m increase in overbank areas within the North Creek catchment, while 0.9 m increase resulted in 0.1 m increase. The 0.5% AEP event results in flood levels up to 0.15 m higher than the 1% AEP event along North Creek, while the 0.2% AEP event is up to 0.4 m higher in the vicinity of King Street.

Economic Impacts of Flooding

A flood damages assessment was undertaken to determine at a catchment scale the economic impacts of flooding. A property database was developed using a combination of surveyed and estimated floor levels for 1,843 properties within the catchment. Flood damage curves were applied that estimate the cost of damage for a certain depth of inundation. Damages were estimated for the catchment for each design flood event. The results were used to determine the average annual damage for the North Creek catchment, which was estimated to be $1.6M. Approximately $900,000 of this is attributed to residential flood damages, with the remaining being commercial, industrial and infrastructure damage. In the 1% AEP event, there is estimated to be over 600 properties affected, with 121 of these experiencing above floor flooding.

LIST OF ACRONYMS

1D	One-dimensional
2D	Two-dimensional
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ARR	Australian Rainfall and Runoff
BoM	Bureau of Meteorology
DA	Development Application
DCCEEW	Department of Climate Change, Energy, the Environment and Water
DCP	Development Control Plan
ELVIS	Elevation Information System
ERP	Emergency Response Planning
EY	Exceedances per Year
FEO	Flooded Exit Overland
FER	Flooded Exit Road
FERC	Flood Emergency Response Classification
FIE	Flooded Isolated Elevated Areas
FIS	Flooded Isolated and Submerged Areas
FPL	Flood Planning Level
FRMS	Floodplain Risk Management Study
FRMS&P	Floodplain Risk Management Study and Plan
GIS	Geographic Information System
GPT	Gross Pollutant Trap
GSDM	Generalised Short Duration Method
HW	Hunter Water
IC	Indirect Consequences
IFD	Intensity, Frequency and Duration (Rainfall)
LEP	Local Environmental Plan
LGA	Local Government Area
LiDAR	Light Detection and Ranging (aerial survey technique)
mAHD	metres above Australian Height Datum
MHL	Manly Hydraulics Laboratory
MHLFIT	Manly Hydraulics Laboratory Flood and Coastal Intelligence Tool
MIKE-11	one-dimensional (1D) hydraulic model
OSD	On-Site Detention
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SES	State Emergency Service
SIX	Spatial Information Exchange
TIN	Triangular Irregular Network (3D surface)
TUFLOW	1D and 2D flood and tide simulation software (hydraulic model)
WBNM	Watershed Bounded Network Model (hydrologic model)
WSUD	Water Sensitive Urban Design
WWPS	Waste Water Pumping Station

ADOPTED TERMINOLOGY

Australian Rainfall and Runoff (ARR, ed Ball et al, 2019) recommends terminology that is not misleading to the public and stakeholders. Therefore the use of terms such as “recurrence interval” and “return period” are no longer recommended as they imply that a given event magnitude is only exceeded at regular intervals such as every 100 years. However, rare events may occur in clusters. For example there are several instances of an event with a 1% chance of occurring within a short period, for example the 1949 and 1950 events at Kempsey. Historically the term Average Recurrence Interval (ARI) has been used.

ARR 2019 recommends the use of Annual Exceedance Probability (AEP). AEP is the probability of an event being equalled or exceeded within a year. AEP may be expressed as either a percentage (%) or 1 in X. Floodplain management typically uses the percentage form of terminology. Therefore a 1% AEP event or 1 in 100 AEP has a 1% chance of being equalled or exceeded in any year.

ARI and AEP are often mistaken as being interchangeable for events equal to or more frequent than 10% AEP. The table below describes how they are subtly different.

For events more frequent than 50% AEP, expressing frequency in terms of Annual Exceedance Probability is not meaningful and misleading particularly in areas with strong seasonality. Therefore the term Exceedances per Year (EY) is recommended. Statistically a 0.5 EY event is not the same as a 50% AEP event, and likewise an event with a 20% AEP is not the same as a 0.2 EY event. For example an event of 0.5 EY is an event which would, on average, occur every two years. A 2 EY event is equivalent to a design event with a 6 month ARI where there is no seasonality, or an event that is likely to occur twice in one year.

The Probable Maximum Flood (PMF) is the largest flood that could possibly occur on a catchment. It is related to the Probable Maximum Precipitation (PMP). The PMP has an approximate probability. Due to the conservativeness applied to other factors influencing flooding a PMP does not translate to a PMF of the same AEP. Therefore an AEP is not assigned to the PMF.

This report has adopted the approach recommended by ARR and uses % AEP for all events rarer than the 50 % AEP and EY for all events more frequent than this as shown in the table below.

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very Frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	19.5
	0.02	2	50	49.5
	0.01	1	100	99.5
Very Rare	0.005	0.5	200	199.5
	0.002	0.2	500	499.5
	0.001	0.1	1000	999.5
	0.0005	0.05	2000	1999.5
Extreme	0.0002	0.02	5000	4999.5
			↓	
			PMP/ PMP Flood	

1. INTRODUCTION

1.1. Study Objectives

Lake Macquarie City Council (Council) engaged WMAwater to undertake the North Creek Warners Bay Flood Study. This study is jointly funded by the NSW Department of Climate Change, Energy, the Environment and Water (DCCEEW) and Council. The data collection and flood study are the first steps in the NSW flood program and will provide the basis for subsequent steps such as the Floodplain Risk Management Study and Plan (FRMS&P). A flood study (Reference 3) and FRMS&P (Reference 4) were previously undertaken, and this study constitutes a review of the 2005 North Creek Flood Study (Reference 3).

The North Creek Warners Bay Flood Study will define the existing flood behaviour within the North Creek catchment using computer models. Those models will utilise the data available to create an accurate representation of the existing catchment flood behaviour. Once the models are established, calibrated and validated, they can be used to subsequently undertake a FRMS&P to identify existing flood risk and develop mitigation options to reduce this risk. The outputs of the study will also be used in planning for future development of the catchment and providing advice to the community and emergency response agencies.

The objective of this study is to improve understanding of flood behaviour and impacts, and better inform management of flood risk in the study area. It aims to provide an understanding of the full range of flood behaviour and consequences in the study area.

1.2. Study Area

The study area comprises the entire North Creek catchment, located on the northern hinterland of Lake Macquarie. The catchment covers an area of approximately 5.3 km^2 and includes the suburb of Lakelands, the majority of the Warners Bay suburb and a small portion of the Speers Point suburb. The catchment lies entirely within the Lake Macquarie City Council Local Government Area (LGA). The catchment is largely developed for residential and commercial/light industrial purposes. The only remaining areas of natural bushland are located in the eastern portion of the catchment and a narrow band along the catchment divide in the north, in addition to small pockets within the catchment. The majority of the urban development has a road system with kerb and gutter and piped drainage system. There are a number of culvert and bridge crossings of the waterways that flow through the catchment. Other informal structures such as fences have been constructed in the floodplain. The study area is shown in Figure 1.

While the primary waterway is North Creek itself, it has a number of unnamed tributaries. Each of these are described below.

North Creek

North Creek itself drains a portion of the catchment to the east and becomes a formal channel upstream of a commercial area off Hillsborough Road (Photo 1). It crosses under two driveways, through a concrete channel (Photo 2) and under the Hillsborough Road service road (3 x 1.05 m diameter pipes) before running parallel to Hillsborough Road. It is here that North Creek officially commences and downstream of this location the creek is in a semi-natural state. It crosses the service road again (2 x 3.6 m x 0.8 m box culverts) and runs through a low-lying pocket of remnant vegetation where the creek spreads out laterally upstream of King Street (Photo 3). The King Street crossing is located near the roundabout intersecting King Street with Medcalf Street, Macquarie Road and Hillsborough Road. This intersection is quite low-lying and has been subject to inundation in the past (see Section 1.3). The creek is then conveyed under a pedestrian bridge, under King Street (4 x 2.4 m x 1.4 m box culverts) and two driveway crossings (Photo 4). The creek then narrows as it is constricted between development to the Walker Street crossing (3 x 2.55 m x 2.0 m box culverts). Upstream of Walker Street a water level gauge has recently been installed (see Section 2.9). Downstream of Walker Street the creek is flanked by heavy vegetation and the creek has fairly flat banks. Flows in excess of the channel capacity can inundate large areas that include residential development. There is a low-level weir located just upstream of Martin Street (Photo 6) and there are also pedestrian bridges located at Albert Street (Photo 6) and John Street (Photo 7). The lower parts of North Creek are estuarine in character, with the main channel being approximately 10 m wide with an invert at approximately - 1 mAHD. The creek is conveyed under a bridge at The Esplanade (Photo 9) immediately before it discharges into Lake Macquarie (Photo 10).

Photo 1: Channel upstream of Hillsborough Road

Photo 2: Section of concrete channel through the Hillsborough Road commercial area

Photo 3: North Creek upstream of King Street

Photo 4: North Creek crossing King Street

Photo 5: North Creek channel upstream of Walker Street

Photo 6: North Creek at Martin Street (*Google Street View*)

Photo 7: North Creek at Albert Street (*Google Street View*)

Photo 8: North Creek at John Street (*Google Street View*)

Photo 9: The Esplanade bridge over North Creek

Photo 10: North Creek outlet to Lake Macquarie

King Street Branch

This branch drains the southeastern portion of the catchment. The flow path runs through a residential area and fills several low-lying areas. Downstream of Yorston Street there is a drainage swale that conveys overland flows (Photo 11). At King Street (Photo 12), flow is conveyed through a box culvert (2 x 1.8 m x 1.17 m) and into a series of concrete and grass-lined channels that cross through private property (Photo 13). The 2005 flood study (Reference 3) noted that there were several obstructions on this branch such as fences and private pedestrian bridges, which is assumed to still be the case (Photo 14). This branch joins North Creek just upstream of Walker Street.

Photo 11: Drainage swale downstream of Yorston Street (*Google Street View*)

Photo 12: Drainage swale upstream of King Street

Photo 13: King Street Branch downstream of King Street (*Google Street View*)

Photo 14: Example of a fence crossing the King Street Branch (*Reference 3*)

Lakelands Branch

The Lakelands Branch commences downstream of the Lakelands Pond which captures runoff from a portion of the Lakelands suburb. A surcharge pit (Photo 15) conveys flow to a box culvert (2.7 m x 0.75 m) when the water level in the pond rises high enough. The culvert discharges into a concrete lined open channel (Photo 16). The Lakelands Branch joins north Creek just upstream of the Martin Street weir.

Photo 15: Lakelands Pond surcharge pit

Photo 16: Concrete lined open channel downstream of Lakelands Pond

Western Tributary (Biddabah Creek)

The largest tributary of North Creek is referred to as the western tributary in the previous flood study (Reference 3), although is understood to be referred to as Biddabah Creek. The tributary originates from the headwaters of Munibung Hill and flows from north to south through the western portion of the North Creek catchment. A channel forms near Grasmere Way and traverses grassed and bushland areas around recent development in the upper catchment. Upstream of Windross Drive there is a small wetland area (Photo 17). Flows are conveyed under Windross Drive (Photo 18) via shallow box culverts (5 x 1.5 m x 0.6 m) and through a bushland area to the Medcalf Street culvert (2 x 1.05 m diameter pipes, Photo 19). Downstream of Medcalf Street, the creek is conveyed through a vegetated channel bordered by residential development (Photo 20). The tributary joins North Creek near Feighan Oval.

Photo 17: Biddabah Creek wetland

Photo 18: Biddabah Creek crossing Windross Drive

Photo 19: Biddabah Creek crossing Medcalf Street

Photo 20: Biddabah Creek channel downstream of Medcalf Street

Seaman Avenue Branch

The Seaman Avenue Branch (as it was called in the previous flood study, Reference 3) is also known as Bangalow Palm Creek, and is a branch of the Western Tributary. The upper portion of the branch is part of the urban drainage system, which discharges into a concrete-lined open channel downstream of Ruswell Avenue. This channel runs in a southeast direction and crosses Medcalf Street (3 x 0.95 m x 0.8 m box culvert, Photo 21) and Seaman Avenue (3.25 m x 0.8 m box culvert, Photo 22). Downstream of Seaman Avenue the channel is grass-lined and runs adjacent to Feighan Oval (Photo 23). This branch joins the Western Tributary at the northern corner of Feighan Park.

Photo 21: Concrete-lined open channel downstream of Medcalf Street (*Google Street View*)

Photo 22: Concrete-lined open channel upstream of Seaman Avenue

Photo 23: Open channel downstream of Seaman Avenue

Vermont Place Branch

The Vermont Place Branch begins at the Vermont Place basin in the east of the catchment. The basin outlets into an open concrete invert and grassed swale. The swale flows northwest and under Myles Avenue (2 x 1.85 m x 0.45 m box culvert, Photo 24). The channel continues northwest towards the New York Avenue basins (Photo 25). A Gross Pollutant Trap (GPT) diverts low flows to the basins with greater flows continuing in the channel. From New York Avenue the path flows through a vegetated section of channel (Photo 26), southwest of the Hillsborough commercial area and northeast of Warners Bay High School sports fields. The branch converges with North Creek immediately downstream of the Hillsborough Road, service road exit.


Photo 24: Vermont Place Branch crossing Myles Avenue


Photo 25: Open channel downstream of Myles Avenue


Photo 26: Vegetated channel, north of New York Avenue basins


Photo 27: Looking upstream over New York Avenue GPT on Vermont Place Branch

1.3. The Flood Problem

Many catchments entering the Lake Macquarie waterway experience poor drainage due to a combination of flat terrain, densely developed areas which can restrict flow paths, restricted openings under road or rail crossings, the presence of localised sag points that are not drained adequately, and/or periodically elevated lake levels. Localised problems also arise due to the age, design and maintenance of urban drainage systems. These problems will likely be exacerbated as ocean and lake levels rise as a result of projected sea level rise or rainfall increase due to anthropogenic climate change.

1.3.1. Causes of Flooding

Flooding within the study area may occur due to three key mechanisms:

1. Intense rainfall over the local catchment which exceeds the capacity of the urban stormwater (pit and pipe network) and flows overland to creeks and waterways. This is known as overland flow. Runoff in excess of the stormwater network capacity can accumulate at sag points and in areas with very little ground slope to facilitate drainage.

2. Elevated water levels within North Creek and its tributaries as a result of intense rainfall over the North Creek catchment. This is known as mainstream flooding. The levels in the creek are driven by the amount of runoff produced by the catchment but can be affected by constrictions along the channel (such as culverts, blockages, vegetation, fences, etc).
3. Elevated levels in the Lake Macquarie waterway due to intense widespread rainfall over the Lake Macquarie catchment. The water level in the lake rises when the rate of inflow into the lake is greater than the outflow to the ocean. The Swansea Channel, the outlet of Lake Macquarie to the ocean, can act as a significant constriction to outflows. Elevated ocean levels (for example a storm surge occurring at high tide) and local wind conditions (wind wave action) can affect also affect the levels in Lake Macquarie. The elevated levels in the lake cause a backwater effect up the North Creek channel. These elevated levels can exacerbate flooding due to local rainfall runoff flooding.

These mechanisms may occur in isolation or in combination with each other. Generally, the peak water level in Lake Macquarie will occur some 8 to 12 hours (or longer) after the peak rainfall over the lake catchment, while peak local catchment flood levels will typically have a much shorter response time (in the order of 1 to 2 hours). This means that even in large rainfall events, the two peaks are unlikely to coincide. For example, in the event of 2-4 February 1990 the peak rainfall intensities for durations up to 6 hours occurred around 10:00 am on 2 February 1990. However, the lake rose in response to several days of rain and peaked around midday on 4 February 1990. The rainfall event causing flooding of the waterways within the North Creek catchment may occur as part of a longer duration storm that causes flooding on Lake Macquarie (as occurred in February 1990), or may occur due to an isolated short duration storm event that does not cause any appreciable rise in lake levels (as occurred in February 2023).

Local overland or stormwater flooding in the North Creek catchment is more frequent, with storms and nuisance local stormwater flooding often occurring several times a year. It is typically mainstream flooding, however, that causes significant issues such as roads being cut off and buildings being inundated, although this occurs less frequently. While the previous flood study (Reference 3) investigated only mainstream flooding from North Creek and its tributaries, this study has investigated overland flow flooding in addition to mainstream flooding.

1.3.2. Historical Flood Occurrences

In large rainfall events where the capacity of the pit and pipe system is exceeded, overland flow paths are activated in North Creek and its tributaries causing inundation of low-lying land adjacent to the creeks. Some roads are also prone to flooding and present a risk to motorists. There have been numerous reports of flooding within the catchment, the most recent being June 2007.

Lake Macquarie City Council has maintained a comprehensive database of peak flood levels in Lake Macquarie waterway since the 1930’s. However, this is not a significant factor for the North Creek catchment as the peak lake level typically occurs several hours after the peak flow in North Creek and the rise in lake level is relatively small, with peak levels at approximately 1 mAHD in the February 1990 and June 2007 events. Council also holds information related to flooding specifically in the North Creek catchment and this was supplied for the North Creek Flood Study (Reference 3) and North Creek FRMS&P (Reference 4). This primarily covered the events of

February 1990 and June 2007, with photographs and peak flood levels recorded. Council does not hold any information regarding flooding within the North Creek catchment after the 2007 event. A water level gauge was installed upstream of Walker Street on North Creek in June 2022, however, there have been no significant flood events since the gauge was installed. A summary of the historical events that have occurred in the North Creek catchment is provided in Table 1.

Table 1: Historic Flood Events in the North Creek Catchment

Event	Description	Source
1946	Appears to be inundation of low lying areas most likely from elevated water levels in Lake Macquarie.	North Creek Flood Study (Reference 3)
1949	Appears to be inundation of low lying areas most likely from elevated water levels in Lake Macquarie.	North Creek Flood Study (Reference 3)
1951	Three low lying properties affected. Minimal information available.	North Creek Flood Study (Reference 3)
February 1982	Photos provided of flooding in the vicinity of Fairfax Road.	North Creek Flood Study (Reference 3)
May 1988	One photo provided of inundation on Martin Street.	North Creek Flood Study (Reference 3)
February 1990	Flooding in the catchment due to intense rainfall. Inundation of King Street and the Hillsborough roundabout. Numerous properties downstream of King Street affected by flooding from North Creek in addition to overland flooding in the vicinity of Campbell Street.	North Creek Flood Study (Reference 3)
April 2001	Three photos provided of inundation on Sweet Street.	North Creek Flood Study (Reference 3)
June 2007	At least 16 building floors were inundated in the North Creek catchment causing significant damage to both commercial and residential properties. King Street was overtopped, with inundation of the roundabout at Hillsborough Road. Numerous low-lying properties downstream of King Street were affected with several properties affected by overland flows (i.e not inundated from elevated water levels in a creek or channel).	North Creek FRMS&P (Reference 4)
April 2015	Known to be a large storm event in the wider Lake Macquarie and Hunter Valley region. Likely that the North Creek catchment experienced flooding to some extent.	-
February 2020	Known to be a large storm event in the wider Lake Macquarie and Hunter Valley region. Likely that the North Creek catchment experienced flooding to some extent.	-
February 2023	Largest event since the installation of the North Creek water level gauge. Water was still in bank at this location, so it is likely that the extent of flooding was minor.	-

1.4. Changes to the Catchment

Since the first European settlement there have been many changes to the landforms and building outlines on the floodplain (extensions to existing houses, new houses) as well as other significant developments on the floodplain, including installation of bridges and the pit and pipe network. Development and upgrades to the pit and pipe network have also occurred. No chronological details of these changes are available.

Some of the changes to the floodplain may be temporary and others permanent. It is likely that these developments on the floodplain will have affected the runoff regime as well as the extent of local catchment flooding. It is impossible to accurately account for these changes over the years due to the lack of detailed information.

Some of the key changes to the catchment since the 2005 Flood Study (Reference 3) include:

- Construction of the Warners Bay Homemakers Centre off Hillsborough Road.
- Development of two new residential subdivision areas between Fairfax Road and Gosforth Grove.
- Rani Close residential subdivision off Thompson Road.
- Smaller commercial and residential developments on greenfield sites in the catchment.
- Infill development (such as knock-down rebuilds, construction of secondary dwellings, new commercial buildings or strata developments).

This assessment was based on a comparison of available aerial imagery from 2007 and 2023 (Figure 2).

2. AVAILABLE DATA

2.1. Previous Studies

2.1.1. North Creek Flood Study, Webb, McKeown & Associates, 2005

Webb, McKeown & Associates (now WMAwater) completed the North Creek Flood Study for Lake Macquarie Council in 2005 (Reference 3). A WBNM hydrologic model and MIKE-11 hydraulic model were developed to simulate flood behaviour for North Creek and its tributaries. The WBNM model consisted of 35 sub-areas that simulated rainfall runoff for the North Creek catchment. The WBNM model adopted a catchment lag (C) value of 1.29, initial loss of 0 mm and continuing loss of 2.5 mm/h. The 1D MIKE-11 model covered the North Creek channel and main tributaries (as outlined in Section 1.2). A calibration was not undertaken due to the lack of available data and certainty of location combined with the limitations of the 1D modelling.

Design storms were simulated used Australian Rainfall and Runoff (ARR) 1987 (Reference 5). The 20%, 10%, 5%, 2%, 1% and 0.5% Annual Exceedance Probability (AEP) events and the Probable Maximum Flood (PMF) events were simulated. The study provided peak flood level profiles, hydraulic categorisation and hazard categorisation. A sensitivity analysis included testing of structure blockage, Mannings ‘n’ roughness, rainfall depths, catchment lag factor and Lake Macquarie tailwater level. A flood damages assessment was also undertaken for 262 properties, with average annual damages (AAD) estimated to be $440,000 (2005 dollars).

While this modelling is now considered to be outdated, there are many components of this study that are relevant to the current study, including the survey of major cross-drainage structures, approximately 70 cross sections and floor levels for 160 buildings across the catchment, in addition to the flood information collected regarding the February 1990 event.

2.1.2. North Creek Floodplain Risk Management Study, WMAwater, 2010

WMAwater undertook the North Creek Floodplain Risk Management Study (FRMS) for City of Lake Macquarie Council in 2010 (Reference 4). The FRMS undertook a review of the Flood Study (Reference 3) modelling, considering the June 2007 event which occurred after the Flood Study was completed. Flood information from the 2007 event was obtained through a questionnaire sent out to residents, with 33 flood marks being subsequently surveyed. Parameters in the hydraulic model were adjusted in order to match the flood marks for the 2007 event. Design flood events were re-simulated with the updated flood model, including 100% blockage of key hydraulic structures.

The FRMS assessed that even in relatively frequent events (such as the 10% AEP), over 20 building floor levels would be inundated. The estimate of AAD for the catchment was $700,000 with the updated modelling. The FRMS investigated a range of flood risk mitigation measures including flood modification measures, property modification measures and response modification measures. Some of the recommended measures include:

- Retarding basins (for future development only)

- Removal of structures within channels and introduction of a maintenance scheme
- Maintain a database of local drainage issues
- Local Flood Plan to be prepared by the NSW State Emergency Service (SES)
- Implement a flood awareness program
- Update development control plan (DCP) to include variable flood planning levels (FPLs) for commercial and industrial development, and include minimum crest levels for basement carparks
- Review on-site detention (OSD) policy and ensure that all development applications (DAs) in the floodplain are supported by a flood study
- House raising to be investigated for 15 houses inundated in the 10% AEP event
- Flood proofing to be promoted as a means of reducing flood damages for non-residential buildings
- Undertaking a detailed flood study to identify overland flow areas
- Progressively upgrade pipes when redevelopment occurs
- Periodically review planning controls and optimise the policy on managing overland flow
- Council to incorporate sea level rise and climate change into FPLs
- Continued use of water sensitive urban design (WSUD) measures were supported

2.1.3. Lake Macquarie Waterway Flood Study, WMAwater 2012

The 2012 Lake Macquarie Waterway Flood Study (Reference 6) was initiated by Council to research and update the prior 1998 Lake Macquarie Flood Study, to incorporate predicted impacts of climate change. It is of relevance to the present study as it provides design flood levels within Lake Macquarie waterway.

The study included modelling of the June 2007 long weekend event and incorporated detailed bathymetric survey within the Swansea Channel. The study established a hydrologic model (WBNM) and hydraulic model (TUFLOW), which were calibrated and validated to the February 1990 and June 2007 long weekend events. The following conditions were adopted for the design lake flood analysis:

- 0.1 mAHD initial water level in the Lake Macquarie waterway (average lake level);
- 48 hour critical rainfall storm duration inflows (for all design events except the PMF) in conjunction with the respective ocean tides;
- design ocean levels based on the design levels in Fort Denison/Sydney Harbour plus a wave setup component (0.2 m assumed for the 1% AEP event);
- all design tides assume the “shape” of the tidal hydrograph of the May 1974 east coast low event as recorded at Fort Denison in Sydney Harbour. This tidal hydrograph approximates the 1% AEP design ocean event;
- the wave setup component was assumed to increase linearly to peak at the same time as the ocean peak;
- the peak ocean level was coincided with the peak rainfall burst in the 48 hour duration event.

Peak ocean levels and peak catchment runoff are unlikely to coincide. The study used an envelope (i.e. whichever event produced the highest level) of the:

- 1% AEP catchment flood ("rain dominated event") in conjunction with a 5% AEP elevated ocean level ("ocean dominated event"); and
- 5% AEP catchment flood ("rain dominated event") in conjunction with a 1% AEP elevated ocean level ("ocean dominated event"),

to establish the 1% AEP design flood level for the lake. A similar approach was used for the other design events.

Design lake flood levels in Lake Macquarie waterway from Reference 6 are reproduced in Table 2 and are based on ARR 1987 (Reference 5) rainfall data. Climate change scenarios were analysed for the 20%, 5% and 1% AEP events and are also summarised in Table 2. The lake flood levels shown in Table 2 exclude wave runup on the foreshore areas within the lake or adjoining the Swansea Channel.

Table 2: Design Lake Flood Levels from Lake Macquarie Waterway Flood Study (Reference 6)

Event (AEP)	Peak Lake Level (m AHD)					
	Existing	Sea Level Rise		Rainfall Increase		
		+ 0.4m	+ 0.9m	10%	20%	30%
50%	0.65	1.04	1.54	0.71	0.77	0.83
20%	0.82	1.21	1.71	0.88	0.94	1.00
10%	0.94	1.32	1.81	1.03	1.11	1.19
5%	1.23	1.61	2.10	1.32	1.40	1.49
2%	1.38	1.74	2.20	1.50	1.61	1.72
1%	1.50	1.86	2.32	1.62	1.73	1.84
0.5%	1.69	2.05	2.51	1.81	1.92	2.03
0.2%	1.87	2.23	2.69	1.99	2.10	2.21
PMF	2.45	2.81	3.27	2.57	2.68	2.79

Note: Underlined levels have been derived from interpolation from model results rather than actual modelling

It should be noted that the application of the 2019 revision of ARR will change the design flood levels shown in Table 2.

2.2. Topographic Data

The Digital Elevation Model (DEM) is a representation of the ground topography and one of the primary inputs into a flood model. The study area DEM was developed using the Light Detection and Ranging (LiDAR) data and detailed topographic survey data, as outlined in the following sections.

2.2.1. LiDAR

The majority of the study area DEM was developed using the LiDAR data. This is a form of aerial survey that uses a laser scanner mounted to an aircraft. This data produces a high-resolution model of the ground elevation over large areas. A gridded DEM is derived from a series of survey points which are filtered to represent the ground surface (removing points with elevations of trees, buildings, cars, etc). Council provided WMAwater with a high resolution DEM derived from LiDAR data. The LiDAR was captured in August 2018 and was provided as a DEM with a 0.2 m resolution.

It is noted that the NSW Government Spatial Services also hold LiDAR data, however, the most recent capture for this area was 2014 and only a 1 m resolution DEM was available. For this reason the Council LiDAR data was adopted for this study.

The terrain across the study area is shown in Figure 3. The LiDAR-derived DEM has limitations in accuracy where there is dense vegetation or waterbodies. Due to this limitation, the LiDAR was supplemented with survey data, as outlined in Section 2.2.2 below.

2.2.2. Detailed Topographic Survey

Detailed topographic survey was undertaken as part of the 2005 Flood Study (Reference 3). This included survey of hydraulic structures such as culverts and bridges in addition to cross sections of opens channels (including North Creek). Further details of the survey can be found in Reference 3 and the locations of the major hydraulic structures are shown in Figure 4.

2.3. Aerial Imagery

Aerial imagery was provided by Council in jpg format. This dataset consists of 1 m resolution aerial imagery captured in 2007, 2010, 2012, 2014, 2016 and 2023. Aerial imagery was also available on platforms such as Google Maps (www.maps.google.com.au), Nearmap (www.nearmap.com) and SIX Maps (six.maps.nsw.gov.au).

2.4. Land Use Zoning

The Lake Macquarie Local Environmental Plan (LEP) 2014 applies to the study area. The LEP zoning was provided by Council for the study area as well as cadastral boundaries. This information identifies lot boundaries and the zoning (such as residential, commercial, industrial and recreational areas). The study area mostly comprises R2 (low density residential), R3 (medium density residential), with small portions of B2 (local centre), B4 (mixed use) and B7 (business park). Fields such as Feighan Oval are classified as RE1 (public recreation), with small areas of RE2 (private recreation). Bushland areas primarily along the catchment boundary are C2 (environmental conservation) and C4 (environmental living). Other GIS boundaries, such as catchments, road corridors and creek lines were also provided.

2.5. Hydraulic Structures

Council provided GIS layers of their stormwater database for the study area. A total of 1,335 pipes and 1,288 pits are located within the study area. Typically, only pipe sizes were available from the dataset. There were sparse data related to invert levels and pit inlet sizes, however, this data did not appear to be reliable. GIS layers of other stormwater features such as trash racks, culverts, headwalls, weirs and channels were also provided. These layers typically only provided the location of such features. Where details were not available, WMAwater and Council measured these structures in the field (see Section 2.7). Major hydraulic structure locations can be found in Figure 4.

2.6. Buildings

Buildings are a major hydraulic feature in the study area. Their presence within overland flow paths and on the banks of North Creek (for example, downstream of King Street) can cause flow constrictions and/or divert floodwaters. Council provided a building layer indicating building footprints. Approximatively 4,700 buildings are within the study area.

2.7. Site Visit

A site visit was conducted on 25 October 2023 by WMAwater and Council staff. The site visit was conducted of the whole study area, with a focus on the major creeks and where these intersect with urban development. The aim of the site visit was to gain an appreciation of the study area including the topography, waterways and urban development. In particular, features that are not readily seen in aerial imagery, LiDAR data or stormwater datasets was able to be observed. Two examples of detention basins are shown in Photo 28 and Photo 29. In addition to this, Council staff inspected several areas where additional information was required to understand how water would be influenced by specific features. This included the trash rack at the end of Derwent Crescent (upstream of Lakelands Drive, Photo 30), Forrester Close detention basin and Wellham Close structures (Photo 31).

Photo 28: Whitehaven Drive Detention Basins

Photo 29: New York Avenue Detention Basins

Photo 30: Derwent Crescent trash rack

Photo 31: Wellham Close hydraulic structures

2.8. Historic Rainfall Data

2.8.1. Overview

Rainfall data is recorded either daily (24-hour rainfall totals to 9:00 am) or continuously (pluviometers measuring rainfall in small increments – less than 1 mm). Daily rainfall data has been recorded for over 100 years at many locations on the east coast of NSW. However, pluviometers have generally only been installed for widespread use since the 1970s. Together these records provide a picture of when and how often large rainfall events have occurred in the past.

Care must be taken when interpreting historical rainfall measurements. Rainfall records may not provide an accurate representation of past flooding due to a combination of factors including local site conditions, human error or limitations inherent to the type of recording instrument used.

Examples of limitations that may impact the quality of data used for the present study are highlighted in the following:

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

- In the past, rainfall over weekends was often erroneously accumulated and recorded as a combined Monday 9:00 am reading.
- The duration of intense rainfall required to produce overland flooding in the study area is typically less than 6 hours (though this rainfall may be contained within a longer period of rainfall). This is termed the "critical storm duration". For a larger catchment (such as the adjacent Cockle Creek catchment) the critical storm duration may be greater than 12 hours. For the study area a short intense period of rainfall can produce flooding but if the rain starts and stops quickly, the daily rainfall total may not necessarily reflect the magnitude of the intensity and subsequent flooding. Alternatively, the rainfall may be relatively consistent throughout the day, producing a large total but only minor flooding.
- Rainfall records can frequently have "gaps" ranging from a few days to several weeks or even years.
- Pluviometer (continuous) records provide a much greater insight into the intensity (depth vs. time) of rainfall events and have the advantage that the data can generally be analysed electronically. This data has much fewer limitations than daily read data. Pluviometers, however, can also fail during storm events due to the extreme weather conditions.

Intense rainfall events which cause overland flooding in highly urbanised catchments are usually localised and as such are only accurately represented by a nearby gauge, preferably within the catchment. Gauges sited even only a kilometre away can show very different intensities and total rainfall depths.

The rainfall data described in the following sections pertains to information that was used in model calibration.

2.8.2. Daily Rainfall Stations

There are a number of daily rainfall stations available around the study area operated by the Bureau of Meteorology (BoM). Stations within approximately 10 kilometres of the study area with useful data for recent years (from 1990, the earliest calibration event) have been selected. This resulted in a total of 9 stations that were analysed as listed in Table 3 and shown in Figure 5. Diagram 1 also shows the operating period of these stations.

Table 3: Daily rainfall stations around the study area

Station Number	Station Name	Start Date	End Date	Length of Record (years)	% Complete
61011	COCKLE CREEK (PASMINCO METALS)	2/01/1900	16/08/2003	103.7	97.9
61133	**BOLTON POINT (THE RIDGE WAY)**	**1/04/1962**	**20/04/2023**	**61.1**	**71.3**
61299	**BELMONT WWTP**	**1/01/1990**	**14/12/2018**	**29.0**	**92.0**
61322	**TORONTO WWTP**	**1/10/1972**	**14/12/2018**	**46.2**	**91.8**
61359	MT HUTTON (AUKLET RD)	1/07/1987	28/02/2005	17.7	99.3
61367	BELMONT NORTH (WOMMARA AVE)	1/04/1990	30/11/1997	7.7	21.7
61370	BARNSLEY (BENDIGO STREET)	1/01/1991	30/04/1997	6.3	99.5
61391	**MEREWETHER (BURWOOD BEACH WWTP)**	**1/01/1990**	**17/09/2019**	**29.7**	**96.0**
61393	**EDGEWORTH WWTP**	**1/01/1990**	**24/10/2019**	**29.8**	**91.1**

Note: Stations in **bold** are currently 'open' however, data is not available up to the current day

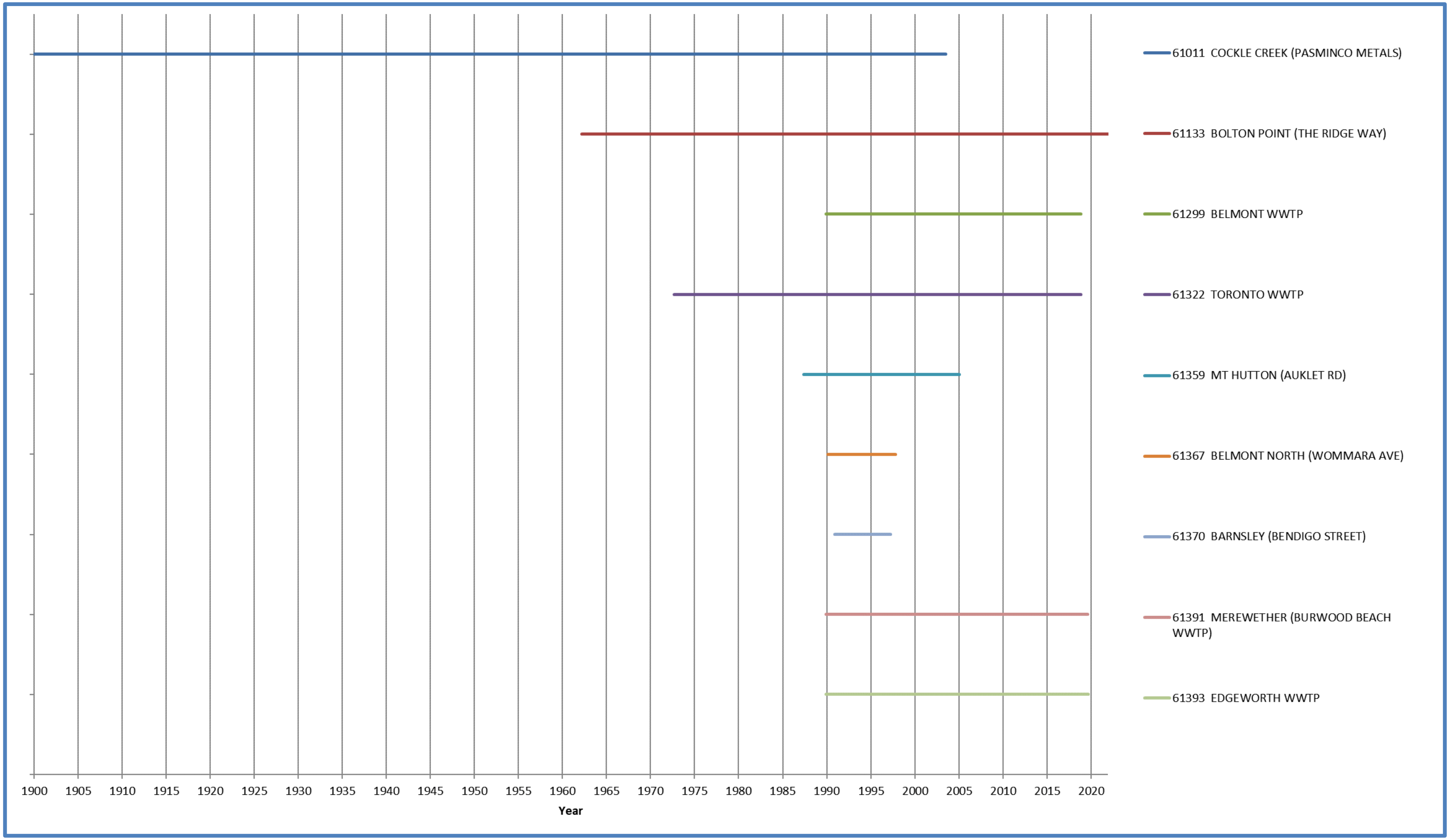


Diagram 1: Available daily rainfall station records for the study area

2.8.3. Sub-Daily Rainfall Stations

A number of continuous rainfall stations (pluviometers) that record data at a sub-daily level are available around the study area. These include stations operated by BoM, Manly Hydraulics Laboratory (MHL) and Hunter Water (HW). MHL operates a number of rainfall and water level gauges on behalf of Council as part of the MHL Flood and Coastal Intelligence Tool (MHLFIT) Lake Macquarie FloodWatch network (https://www.mhl.nsw.gov.au/users/LakeMac/). Rainfall gauges relevant to the North Creek catchment are located to the south of the catchment, at Eleebana Reservoir (561158) and Windale No 2 Waste Water Pumping Station (WWPS) (561172), within 5 km of the catchment centroid. Since these stations were only installed recently, a station at Barnsley (561067) was also utilised for earlier calibration events. HW operates a network of sub-daily rainfall stations as part of their wastewater treatment works and stormwater infrastructure. It is understood that the HW gauges were not operational in 1990, however, data were available for the 2007 event, obtained from Reference 6 and stations within 5 km of the catchment centroid were analysed. These stations are summarised in Table 4 and shown in Figure 5.

Table 4: Sub-daily rainfall stations around the study area

Station Number	Station Name	Start Date	End Date	Operating Authority	Distance from centroid of Catchment (km)
561158	Eleebana Reservoir	19/11/2015	current	MHL	3.2
561172	Windale No. 2 WWPS	31/08/2021	current	MHL	4.6
561067	Barnsley	22/01/1988	current	MHL	5.9
R7	Taralba 1 WWPS Rain Gauge	2007 event only		HW	3.8
R39	Cardiff Chlorinator Rain Gauge	2007 event only		HW	5.1
TR100	Eleebana	2007 event only		HW	1.6
TR105	Windale	2007 event only		HW	5.0
TR106	Dudley	2007 event only		HW	3.4

The MHL stations, being part of the MHLFIT network, send alerts if the gauge records:

- 20 mm or greater of rainfall in 1 hour
- 70 mm or greater of rainfall in 3 hours
- 150 mm or greater of rainfall in 24 hours

The closest sub-daily rainfall stations operated by BoM are the Newcastle University pluviometer station (061390) and the Newcastle Nobbys Signal Station AWS station (061055), located more than 10 km away. No BoM sub-daily rainfall stations were included in the current analysis for this reason.

2.9. Water Level Gauges and Flood Marks

A water level gauge owned by Council and operated and maintained by MHL was installed on 9 June 2022 (No. 2114110) upstream of Walker Street (Photo 32) as part of the MHLFIT Lake Macquarie FloodWatch network. The gauge does not have a rating curve and hence only water levels are recorded (stream flow cannot be estimated). The gauge continuously (15 minute interval) records water levels and automatically issues alerts to registered users when the creek water level reaches 4.7 mAHD (water spills onto Walker Street at approximately 3 mAHD).

Photo 32: Water level gauge installed immediately upstream of Walker Street

There are various water level gauges (Figure 5) installed on Lake Macquarie itself as part of the MHLFIT Lake Macquarie FloodWatch network. The closest station to Warners Bay is located at Marmong Point (No. 211460), on the western side of the lake but the water level would be representative of the lake level at the outlet of North Creek. The gauge was installed in June 1986 and is currently operational. Flood warnings for the lake are issued from the Belmont water level gauge rather than from this gauge. The following NSW SES/BoM flood classifications apply to Lake Macquarie waterway:

- Minor flood classification = 0.7 mAHD
- Moderate flood classification = 0.9 mAHD
- Major flood classification = 1.1 mAHD

Council provided a flood mark database that covers the North Creek catchment and a summary of the available flood marks is provided in Table 5. The 1990 and 2007 events have the largest number of flood marks available across the catchment, indicating that these are likely the largest events that have occurred in the North Creek catchment in the last 30+ years.

Table 5: Summary of available flood marks

Event	Number of Flood Marks
1946	1
1949	1
1951	3
1990	36
1995	1
2007	41
2015	4

2.10. Calibration Events

There have been two major floods in recent history within the North Creek catchment, namely the February 1990 and June 2007 events. These are the only flood events which have a sufficient number of flood marks to enable model calibration. The water level gauge recently installed on North Creek at Walker Street has recorded no major flood events. The largest event occurred on 22 February 2023 where the water level reached 2.08 mAHD with several events reaching approximately 1.8 mAHD. The July 2022 event was also selected as a calibration event to match the in-bank flood behaviour of North Creek using the Walker Street gauge.

3. COMMUNITY CONSULTATION

At the commencement of the project, the community were informed of the study and provided the opportunity to contribute their observations of flooding within the catchment. Information on the study was provided on Council's 'Shape Lake Mac' website and a questionnaire was available from 29 November 2023 to 10 January 2024 to enable residents to share their experiences of flooding.

A newsletter was also produced and sent to approximately 2,800 residents in the catchment. A summary of the engagement can be found in Appendix B. There was a total of 345 visits to the webpage and 11 questionnaire responses were received. Respondents have lived around North Creek for varying periods from 12 months to 24 years, and experienced flood events in February 1990, April 2001, June 2007 and April 2015. The majority of respondents (9) indicated that their property had not been affected by flooding, while two indicated that the front/back yard and/or garage/shed were affected. Three responses contained information that could be used to validate the flood model. One specifically referred to the June 2007 event and two mentioned overland flows in heavy rainfall events. The location of the respondents and a summary of useful information can be found in Figure 6, noting that one respondent (that indicated they were not flood affected) was located in Speers Point, outside of the study area.

4. HYDROLOGIC MODELLING

4.1. Background

A hydrological model is a computer-based software tool for estimating the amount of runoff that flows from a catchment for a given amount of rainfall, and the timing of this runoff. Water level gauges are a means of measuring this information, but they are expensive to setup and maintain. They also require a long record (several decades) and measurements of the velocity of flow during flood events (known as gaugings) required to determine the flows (m^3/s).

In the case of North Creek, while there is a gauge at Walker Street, it has a very short record and there are no gaugings such that the recorded water level cannot be converted to a flow. As such, using a computer-based hydrologic model is the best practice method for determining how much flow occurs from rainfall information (which is more widely available from rain gauges). This type of hydrologic model is referred to as a runoff-routing model.

A range of runoff-routing hydrologic models is available as described in Australian Rainfall and Runoff (ARR) 2019 (Reference 7). These models allow the rainfall to vary in both space and time over the catchment and will calculate the runoff generated by each sub-catchment. The generated flow hydrographs then serve as inputs at the boundaries of the hydraulic model, which provides details about flood levels and velocities.

The WBNM package was selected for this study and is widely used throughout Australia to estimate runoff from both rural and urban areas. The WBNM model has a relatively simple but well supported method, where the routing behaviour of the catchment is primarily assumed to be correlated with the catchment area. If flow data are available at a stream gauge, then the WBNM model can be calibrated to this data through adjustment of various model parameters including the stream lag factor, catchment lag factor, and/or rainfall losses. Further details regarding the WBNM software can be found in the WBNM User Guide (Reference 8).

WMAwater developed a WBNM model covering the entire North Creek catchment, with details of this process provided in the following sections.

4.2. Sub-catchment Delineation

The study area was divided into 245 sub-catchments (shown in Figure 7) by using the available LiDAR data (see Section 2.2.1). The discretisation of sub-catchments within the study area has a direct influence on the scale of flooding to be simulated in the hydraulic model. For this study the sub-catchments were delineated to the stormwater pits and other key hydraulic controls such as culverts and channels. The LiDAR data was used to determine the upper catchment boundary and the gutter and stormwater network locations were used to determine the location of flow paths.

The total catchment area contained in the WBNM model is approximately 5.4 km^2, resulting in an average sub-catchment size of 2.2 hectares. There are generally smaller, more detailed sub-catchments within the urbanised parts of the catchment, with coarser sub-catchments within the forested upstream areas. The largest sub-catchment is 13 hectares and the smallest is

0.2 hectares. This relatively fine-resolution sub-catchment delineation, particularly within the urban areas, ensures that where significant overland flow paths exist in the catchment, they are accounted for and incorporated into the TUFLOW hydraulic model.

4.3. Adopted Hydrologic Model Parameters

The model input parameters to represent each sub-catchment are:

- A catchment lag factor (termed 'C'), which can be used to accelerate or delay the runoff response to rainfall;
- A stream lag factor, which can accelerate or decelerate in-channel flows occurring through each sub-catchment;
- An impervious area lag factor;
- Catchment area; and
- The percentage of catchment area with a pervious/impervious surface.

WBNM requires a catchment lag parameter and a stream lag factor to be selected which describes the average travel time for runoff from the catchment surface. The catchment lag parameter is applied to pervious surfaces and adjusted to apply to impervious surfaces by multiplication by an impervious lag factor. A catchment lag factor ('C') of 2.4 was adopted. This is higher than the value recommended for NSW ungauged catchments (1.6), however, is consistent with studies or catchments draining to the northern portion of Lake Macquarie, as outlined in Table 6.

A sensitivity analysis was undertaken for the catchment lag factor, with the results presented in Section 9.3. A stream lag factor of 1.0 was adopted, representing a natural channel. The default impervious lag factor of 0.1 was adopted for the routing of flows from impervious areas. Catchment areas were calculated based on sub-catchment boundaries in a GIS program. The impervious fractions within the catchment are discussed in Section 4.5 below.

Table 6: WBNM Catchment Lag Factors for Nearby Catchments

Catchment	Catchment Lag Factor ('C')	Study	Reference
Lake Macquarie	2.4	Lake Macquarie Waterway Flood Study	WMAwater, 2012
Sheppards Creek	2.4	Flood Studies for Eight Residual Lake Macquarie Waterway Tributary Catchments	WMAwater, 2021
South Creek	N/A - adopted 'direct rainfall' approach	South Creek Flood Study	Cardno Lawson Treloar, 2010
Winding Creek and Lower Cockle Creek	2.4 (1.7 for urban areas) (3.0 for dense forest)	Winding Creek and Lower Cockle Creek Flood Study	WMAwater, 2017
Upper Cockle Creek	2.4	Upper Cockle Creek Flood Study	WMAwater, 2019

Catchment	Catchment Lag Factor ('C')	Study	Reference
LT Creek	N/A – adopted RAFTS modelling	LT Creek Flood Study	BMT WBM, 2011
Stony Creek	N/A – adopted RAFTS modelling	Stony Creek Flood Study	Cardno Lawson Treloar, 2005

4.4. Rainfall Losses

Methods for modelling the proportion of rainfall that is "lost" to infiltration are outlined in ARR 2019 (Reference 7). The methods are of varying degrees of complexity, with the more complex options only suitable if sufficient data is available. The method most typically used for design flood estimation and adopted in this study is to apply an initial and continuing loss to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur, including interception, infiltration and the filling of localised depressions. The continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

4.5. Impervious Fraction

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete surfaces occurs significantly faster than from vegetated surfaces. There is proportionally a greater volume of runoff as there is reduced infiltration into the ground from these surfaces. This results in a faster concentration of flow within the downstream area of the catchment, and increased peak flow in some situations compared to the pre-developed state. ARR 2019 (Reference 7) identifies three types of surfaces for the purpose of estimating urban storm losses and routing:

- Directly connected impervious areas which are impervious areas directly connected to the drainage system;
- Pervious areas consisting of parks and bushland areas; and
- Indirectly connected areas which consist of impervious areas which are not directly connected to the drainage system and the pervious areas which interact with indirectly connected impervious areas.

These three surface types were categorised for each sub-catchment and derived from land use within each sub-catchment. The assumed proportion of each of the above categories for each land use type is outlined in Table 7.

Table 7: Land use categories and adopted fraction of each surface type for the WBNM model

Land Use	Directly connected impervious fraction (%)	Pervious fraction (%)	Indirectly connected fraction (%)
Vegetation/Grass/Field	0	100	0
Main channels and open water	100	0	0
Commercial and Industrial	50	33.3	16.7
Residential	25	50	25
Roads/Pavements	90	0	10
Building	100	0	0

WBNM, however, only accounts for pervious and impervious components of each sub-catchment. As such, the surface types defined above were implemented in the WBNM model as follows:

- **Directly connected impervious areas**: This is applied as the impervious fraction of each sub-catchment. These impervious areas have impervious losses applied and are routed through the impervious fraction of the sub-catchment.
- **Pervious areas**: This is applied as part of the pervious fraction of each sub-catchment. These pervious areas have pervious losses applied and are routed through the pervious fraction of the sub-catchment.
- **Indirectly connected areas**: This is applied as part of the pervious fraction of each sub-catchment, being routed through the pervious portion of the sub-catchment. The losses for the pervious area of the sub-catchment, however, are factored to account for the impervious surfaces within the indirectly connected areas. It has been assumed that the initial loss over the indirectly connected areas is approximately 70% of that compared to a purely pervious area (i.e. approximately 30% of the indirectly connected area contains impervious surfaces). Similarly, the continuing losses are also adjusted by the same factor to account for the indirectly connected areas.

The proportion of each land use type within each sub-catchment was determined, and an overall percentage of each surface type was calculated. An example is provided in Diagram 2.

Diagram 2: Land types within sub-subcatchment NC98

For sub-catchment NC98, the proportion of each land use type is shown in Table 8.

Table 8: Land use categories and percentages for NC98

Land Use	Percentage of sub-catchment area (%)
Residential	5
Road corridor	15
Commercial and Industrial	41
Building	35
North Creek channel	4

Using the adopted percentage of each surface type for each land use category (Table 7), an overall fraction for each surface type was then computed. The overall fraction of each surface type for NC98 is as follows:

- Directly connected impervious area = 74%
- Pervious area = 16%
- Indirectly connected area = 10%

5. HYDRAULIC MODEL

5.1. Background

Hydraulic modelling is the simulation of how flow moves across the terrain. A hydraulic model can estimate the flood levels, depths, velocities and extents across the floodplain. It can also provide information about how the flooding changes over time. The hydraulic model can simulate floodwater both within the creek banks, and when it breaks out and flows overland, including flows through structures (such as bridges and culverts), over roads and around buildings.

Two-dimensional (2D) hydraulic modelling is currently the best practice standard for flood modelling. It requires high resolution information about the topography, which is available for this study from the LiDAR aerial survey (Section 2.2.1). Various 2D software packages are available (SOBEK, TUFLOW, RMA-2, MIKE FLOOD). The TUFLOW package was adopted as it meets requirements for best practice and is currently the most widely used flood model of this type in Australia.

The TUFLOW modelling package includes a finite difference or finite volume numerical model for the solution of the depth averaged shallow water equations in two dimensions. The TUFLOW software has been widely used for a range of similar floodplain projects both internationally and within Australia and is capable of dynamically simulating complex overland flow regimes. The TUFLOW model version used in this study was 2023-03-AB-w64 (using the finite volume HPC solver in single precision mode). The TUFLOW Heavily Parallelised Compute (HPC) solver can run on a Graphics Processor Unit (GPU) and is significantly faster than TUFLOW Classic models, which rely on a Central Processing Unit (CPU). HPC models can be run across thousands of cores within a GPU. This scheme is also more robust, being a finite volume scheme. Further details regarding TUFLOW software can be found in the User Manual (Reference 9).

In TUFLOW the ground topography is represented as a uniform grid with ground elevations and Mannings 'n' roughness value assigned to each grid cell. The grid size is determined as a balance between the model result definition required and the computer processing time needed to run the simulations. The greater the definition (i.e. the smaller the grid size) the greater the processing time need to run the simulation. TUFLOW also has the ability to dynamically link to the 1D ESTRY engine, making it useful for simulating both overland flow in the 2D domain, and flows through underground pipes and culverts in the 1D domain.

The hydraulic model covers the entire study area including the suburbs of Lakelands and Warners Bay (see Figure 8) from the catchment divide to Warners Bay. The model covers an area of approximately 5.8 km^2 and utilises a 1 m by 1 m grid resolution.

5.1. Model Topography

The hydraulic model reliability is heavily dependent on the underlying terrain data. The 2D topography for the TUFLOW model was based on the 0.2 m DEM derived from LiDAR data, captured in August 2018 (see Section 2.2.1 for details, shown in Figure 3). The LiDAR data is most accurate on hard surfaces exposed to the sky (for example roads). The data is less accurate where there is dense vegetation covering the ground (such as within vegetated creek channels). Flow paths within the upper catchment urban areas are typically along streets where the LiDAR is reliable. The open channels and creeks that receive and convey this water are less defined in the LiDAR data. This is due to the size of the channels (for example the narrow concrete lined channels of the Lakelands Branch and Seaman Avenue Branch), and dense vegetation that lines the creeks along North Creek. Open channels were incorporated into the topography based on the available information, as detailed in Section 5.2.5. Additional topographic modifications were made to the terrain to ensure correct representation of hydraulic features, as outlined in the following sections.

5.2. Hydraulic Structures

5.2.1. Road Kerbs and Gutters

The road gutter network plays a key role for overland flow in the urbanised parts of the study area. Roadways typically capture the runoff from properties and convey flow within the gutter. Representation of the kerb and gutter system in the model is therefore an important feature to accurately simulate overland flows.

LiDAR typically does not have sufficient resolution to adequately define the kerb and gutter system within roadways. The kerb/gutter feature is of a scale that is smaller than the underlying 1 m LiDAR grid used in the model, and even use of the LiDAR return points do not pick up a continuous line of low points defining the drainage line along the edge of the kerb. Project 15 of ARR 2019 – *Two Dimensional Modelling in Urban and Rural Floodplains* (Reference 10) provides the following guidance:

> *"Stamping a preferred flow path into a model grid/mesh (at the location of the physical kerb/gutter system) may produce more realistic model results, particularly with respect to smaller flood events that are of similar magnitude to the design capacity of the kerb and gutter. Stamping of the kerb/gutter alignment begins by digitising the kerb and gutter interval in a GIS environment. This interval is then used to select the model grid/mesh elements that it overlays in such a way that a connected flow path is selected (i.e. element linkage is orthogonal). These selected elements may then be lowered relative to the remaining grid/mesh."*

In order to model the road drainage system effectively, the gutters were stamped into the mesh using the method described above. The road corridor layer provided by Council was used to create a gutter line which was reviewed using aerial imagery and modified where required to ensure a reasonable alignment. A total of 79 km of gutter lines were included in the model.

The method used was to inspect the LiDAR by automatically generating points a maximum of every 2 metres along the gutter lines and sampling the lowest elevation value within a 1 metre radius of the point. The elevation of those points was lowered by 0.1 m to simulate a continuous flow path in the gutter.

5.2.2. Pits and Pipes

Urban areas that are developed over natural watercourses typically have a drainage system that consists of pit inlets to capture surface water and a pipe system to convey that water underground to a downstream outlet location, such as an open channel or creek. The hydraulic model incorporates this stormwater, or 'pit and pipe' network.

The stormwater drainage network was modelled in TUFLOW as a 1D network dynamically linked to the 2D overland flow domain. This stormwater network includes conduits such as pipes / box culverts, and stormwater pits including inlet pits and junction manholes. The schematisation of the stormwater network was undertaken using the stormwater GIS layers supplied by Council (see Section 2.5). Pipe sizes were obtained from the GIS layer and checked for consistency along each branch (ensuring pipe sizes do not get smaller downstream). Invert levels were not available and hence were estimated based on the ground level (from LiDAR data) minus the pipe width and cover (0.3 m assumed). The alignment of the pipes and location of the stormwater pits were reviewed against the provided aerial imagery to ensure pits were located in the gutter. The locations of pits and pipes included as 1D elements in the hydraulic model are shown on Figure 9 and a summary of pipes by size is provided in Table 9 (also includes culvert structures as detailed in Section 5.2.3). A total of 1,288 pits were included in the model and where these had a surface inlet, a 2.1 m kerb inlet was assumed.

Table 9: Stormwater pipes and culverts included in the model

Pipe Diameter (mm)	Number of Pipes in Model
< 375	22
375	676
450	217
525	101
600	99
675	27
750	35
900	64
> 900	60
Box Culvert	35
Total	**1,336**

5.2.3. Culverts

Cross drainage culverts were identified in the Council stormwater database (see Section 2.5). The culvert dimensions and invert levels were checked and updated based on the topographic survey (Section 2.2.2) and site visit (Section 2.7). The culverts were included in the hydraulic model as 1D elements. The locations of headwalls were modified to align with the terrain low points. Local terrain modifications were made at culvert inlets and outlets where necessary to ensure the transfer of water between the 2D domain and the 1D culverts.

5.2.4. Bridges

There are several bridge structures that cross North Creek. The primary one is The Esplanade bridge, as shown in Photo 9 located at the outlet of North Creek. The structure was included in the 2D domain of the hydraulic model. The obstruction of the deck and handrail was included in the model, as well as form losses for the piers. Losses were applied in accordance with recent research from TUFLOW (Reference 11). Several other bridge structures were also included in the TUFLOW model in a similar manner. These included a pedestrian bridge upstream of King Street (Photo 33), two clear spanning driveway bridges downstream of King Street (Photo 34), a small bridge upstream of Walker Street (Photo 35) and pedestrian bridges at Albert Street (Photo 7) and John Street (Photo 8).

Photo 33: Pedestrian bridge upstream of King Street

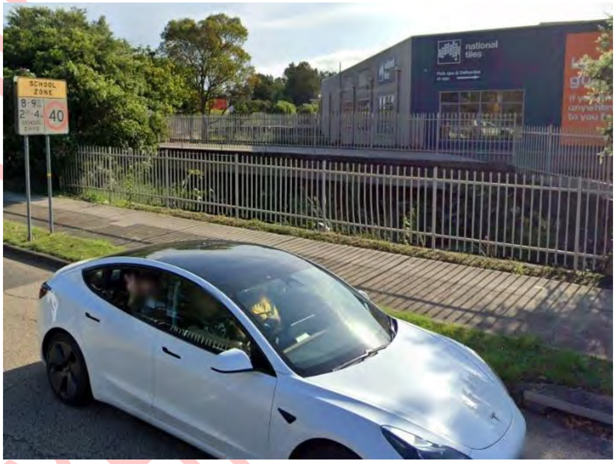


Photo 34: Driveway crossings downstream of King Street (*Google Street* View)

Photo 35: Bridge upstream of Walker Street

There are numerous other bridge structures that span minor waterways throughout the study area. These are typically small single-spanning footbridges (for example those crossing the King Street Branch). These bridges are likely to only have very localised hydraulic effects and there are too many to quantify and model at the catchment-wide scale. The hydraulic effects would only be initiated when the flow comes into contact with the structure. At this level, the flow breaks out of the main channel and the overbank flow is of more concern rather than the specifics of the flow within the channel and any interaction with small structures.

5.2.5. Channels

As outlined in Section 5.1, open channels were incorporated into the 2D domain where the LiDAR data was less reliable. The available detailed topographic survey cross sections (see Section 2.2.2) were used to generate a detailed TIN. The surveyed points were used to represent the channel invert and width, ensuring that the bank levels aligned with the LiDAR data outside of the channel. This was done for the Lakelands Branch (downstream of Medcalf Street), Seaman Avenue Branch and North Creek (downstream of Albert Street). An example cross section is provided in Diagram 3, showing the LiDAR data, surveyed cross section and the resulting TUFLOW DEM.

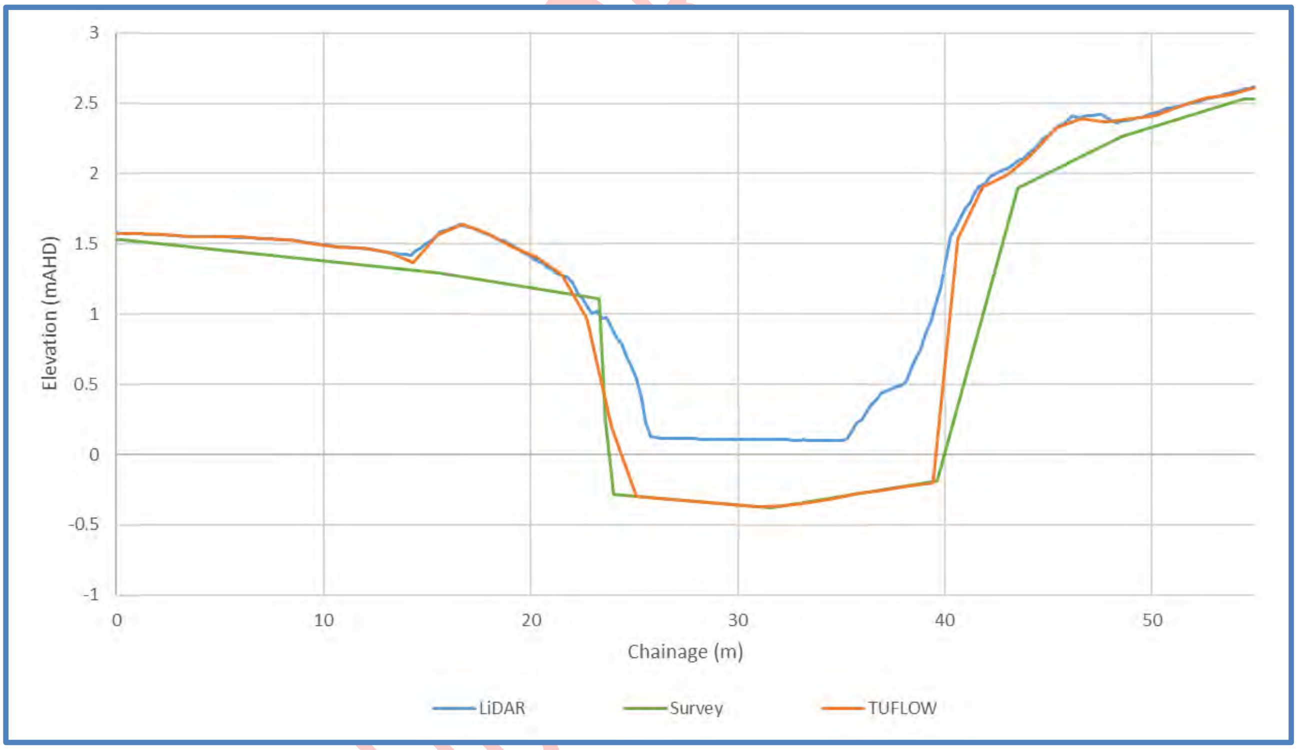


Diagram 3: Example cross-section for North Creek, downstream of the weir where survey data was adopted

For several other channels, the survey data was not available or not detailed enough to generate a representative TIN of the channel and the LiDAR data was taken as a reasonable representation of the channel shape. The lowest elevations from the LiDAR data were sampled at a 5 m interval to generate a breakline to ensure a continuous flow path is represented along the creek. Where reasonable surveyed invert levels could be extracted, these were used to enforce the channel invert. This method was applied to the King Street Branch (downstream of King Street), the Western Tributary (downstream of Medcalf Street) and North Creek (upstream of Albert Street to the first driveway crossing of the creek in the commercial area off Hillsborough Road). These modifications for open channels more accurately define the in-channel shape and capacity the 2D domain.

An example cross section is provided in Diagram 4, showing the LiDAR data, surveyed cross section and the resulting TUFLOW DEM.

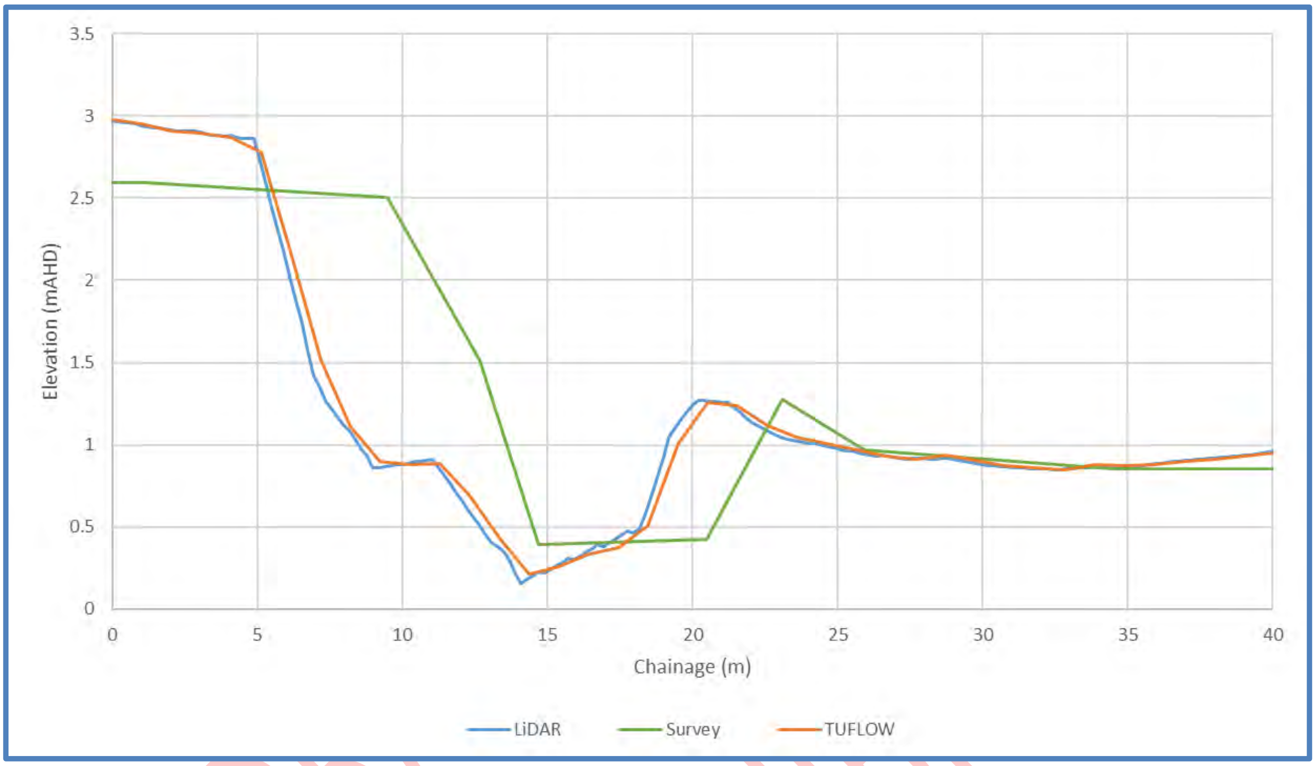


Diagram 4: Example cross-section for North Creek, upstream of the weir where LiDAR data was adopted

5.2.6. Gross Pollutant Traps

There were several GPTs identified in the Council stormwater database. Many of these water quality devices are typically small inline structures that would not have a significant impact on flood behaviour (examples shown in Photo 36 and Photo 37). One large device, however, is located upstream of the Lakelands Pond, at Derwent Cresent (Photo 30). This GPT would impede flows due to the rack itself and any debris that would build up in a flood event. As such, this structure was modelled with a 50% blockage applied.

Photo 36: Water quality device at New York Avenue

Photo 37: Water quality device at Whitehaven Drive

5.2.7. Buildings

Buildings in overland flow paths can have a significant influence on surrounding flood levels and can redirect floodwater. Buildings and other significant features likely to obstruct flow were incorporated into the model and represented as solid obstructions to flow by blocking them out of the TUFLOW grid. The buildings were based on a building footprint layer provided by Council (see Section 2.6). The building footprints were reviewed using aerial imagery and Google Street View in key overland flow areas, and modified where required. Approximately 4,700 buildings were included in the model and are shown in Figure 10.

5.2.8. Fences

Smaller localised obstructions (such as fences) can be represented in TUFLOW in several ways including as impermeable obstructions, a percentage blockage or as an energy loss. The obstructions may also be approximated generally by increasing Mannings roughness for certain land use areas (such as residential) to represent the typical type of fencing used in such areas.

Individual fences in the catchment were not explicitly modelled, as they are difficult to identify and relatively impermanent (since people can change their fences without Council approval). Fences in urbanised areas were therefore accounted for by applying a slightly higher Mannings roughness for the residential land-use type to simulate the obstruction to flow.

5.3. Hydraulic Roughness

Roughness, represented by the Mannings ‘n’ coefficient, is a key parameter in hydraulic modelling. It models the resistance that floodwaters experience when flowing over a surface. For example, floodwater flows more easily over a concrete carpark surface than through dense vegetation in a natural creek channel. As part of the calibration process, roughness values are adjusted within the ranges defined in the literature so that the model better matches observed peak flood levels at a variety of locations. The typical ranges of the Mannings ‘n’ coefficient for different surface types are discussed in Project 15 of ARR 2019 – *Two Dimensional Modelling Urban and Rural*

Floodplains (Reference 10). Chow (Reference 12) also provides some information with regards to the setting of the roughness values for hydraulic calculations.

The Mannings 'n' values adopted for the study area are shown in Table 10. These values have been adopted based on the site inspection, past experience in similar floodplain environments, consideration of the above references and the model calibration process. The spatial variation in Mannings 'n' is shown in Figure 10. The land use planning layer was used to determine the land use type across the study area and was reviewed against the available aerial imagery.

Table 10: Mannings 'n' values adopted in the TUFLOW model

Land Type	Mannings 'n'
Light vegetation/grass/field (RE1 RE2 RU4 RU6)	0.04
Heavy vegetation (C2 C3 C4)	0.08
Concrete-lined channels	0.025
Main channels with minimal vegetation	0.035
Open water (Lakelands Pond and Lake Macquarie)	0.04
Overbank areas with dense vegetation, channels with reed type vegetation	0.12
Commercial and industrial lots (B1 B2 B4 B7)	0.07
Residential lots (R2 R3)	0.05
SP2 Roads and paved areas (SP2)	0.02

5.4. Boundary Conditions

5.4.1. Inflows

The WBNM hydrologic model (Section 4) simulates the runoff that occurs for a particular rainfall event and runoff hydrographs are generated for each sub-catchment area. These hydrographs are applied at the downstream end of each sub-catchment, within the TUFLOW 2D domain (see Figure 8). These inflow locations correspond with stormwater inlet pits (146 inflow points) or drainage reserves and open watercourses (99 inflow points). These inflow points typically receive inter-allotment drainage and sheet runoff flows from upstream catchment areas. Flows for an additional three sub-catchments were distributed across seven inflow points, to obtain better resolution of flooding within the sub-catchment. The flows were assigned based on the proportion of the sub-catchment area that drains to each inflow location.

5.4.2. Downstream Boundary Condition

The southwestern edge of the study area is located along Warners Bay (Lake Macquarie). A water level boundary was applied within Warners Bay as shown in Figure 8. This allows water from North Creek to discharge into Warners Bay (and flow out of the model), as well as simulating the backwater effects of Lake Macquarie water levels up North Creek.

6. MODEL CALIBRATION

6.1. Approach

The aim of the calibration process is to ensure the modelling system can replicate historical flood behaviour. There are assumptions in the modelling inputs, such as the effect of vegetation on flow and the amount of infiltration into the soil, which can be adjusted to improve the match between observed and modelled flood levels. A good match to historical flood behaviour provides confidence that the modelling methodology and schematisation can accurately represent the important flood processes in the catchment.

The choice of calibration events for flood modelling depends on a combination of the severity of the flood event and the quality of the data available. Ideally, data would be available from streamflow and rainfall gauges in addition to records of flood marks or inundation extents. Typically, in urban catchments both gauge records and reliable calibration information is lacking. The following limitations prevent a comprehensive calibration of the hydrologic and hydraulic models for this study.

- There is only a limited amount of historical flood information available for the study area. The available water level gauge at Walker Street has only been operational for two years and there are only a limited number of surveyed flood marks and flood photographs available.
- Rainfall records and particularly pluviometer records for past floods within the catchment are limited. Rain gauges are sparsely distributed and may not accurately capture the spatial and temporal distribution of rainfall during the storm event.
- Changes to the catchment over time due to urban development may result in significant changes to land uses and drainage structures. The models have been developed for current conditions, and the simulation of historic events too far into the past may not be accurate due to changes in the catchment.

These limitations are typical of the majority of urban catchments on the NSW coast and the calibration exercise undertaken here constitutes recommended practice as outlined in Reference 10. All of the available historic flood data has been used to undertake a model calibration as far as practically possible. In addition to this, a detailed sensitivity analysis (refer to Section 9) was undertaken to understand how variations to the adopted model parameters influence the modelled flood behaviour.

The following events were selected for calibration based on the available data and relative size of the events:

- **February 1990** – selected as it is a large flood event that has occurred relatively recently in the catchment with a number of flood marks available.
- **June 2007** – selected as it is a large flood event that has occurred relatively recently in the catchment with a number of flood marks available.
- **July 2022** – selected as it is within the period that the Walker Street gauge has been operational. Although a relatively minor event with in-bank flows, calibration to the Walker Street gauge was considered important.

6.2. Summary of Historical Event Rainfall Data

6.2.1. The February 1990 Storm Event

The storm of 2nd – 6th of February 1990 impacted the study area and a spatial analysis of the daily rainfall gauges was undertaken, using daily and sub-daily rainfall recorded at 9am on the 2nd – 6th February 1990 is shown in Figure 11. This indicates that the storm was relatively consistent across the North Creek catchment, with approximately 480 mm of rainfall over 5 days. An analysis of the nearby sub-daily rainfall data was also undertaken. Only the Barnsley station (561067) had data available for this event and cumulative rainfall is shown in Figure 12. The majority of rainfall occurred on the 2nd and 3rd February, with the most intense burst occurring around midday on the 2nd. The sub-daily rainfall was compared to the ARR 2019 design Intensity Frequency Durations (IFDs) at the catchment centroid with the comparison shown in Figure 13. This indicates that the rainfall burst of approximately 3 to 18 hours was approximately a 5% AEP event, while the rarity increases for longer durations, up to 1% AEP for a 48 hour duration. It is noted that the rainfall total at Barnsley was approximately 10% lower than the estimated rainfall over the North Creek catchment.

6.2.2. The June 2007 Storm Event

The June 2007 storm event is commonly known as the “Pasha Bulker” storm as due to this storm event the Pasha Bulker coal ship ran aground off Newcastle’s Nobbys Beach. The storm, driven by an east coast low, was most intense in the Newcastle region. The 24-hour rainfall totals for the 2007 event (recorded at 9am on 9th of June) show a gradient from east to west across the North Creek catchment, with the rainfall reaching 300 mm in the east and 220 mm in the west, as shown in Figure 14. The sub-daily rainfall records show a reasonably similar temporal pattern for the gauges, although the gauges to the west (R7 and Barnsley) indicate a larger proportion of rain falling at the start of the event, but with lower overall rainfall totals, as shown in Figure 15. A comparison of the sub-daily rainfall records with the ARR 2019 IFD at the catchment centroid shows a similar peak burst intensity around 6 to 12 hours, being between a 2% AEP and 0.2% AEP event for the nearby gauges, as shown in Figure 16.

6.2.3. The July 2022 Storm Event

The July 2022 storm event was a reasonably small rainfall event. Daily rainfall totals from nearby available gauges were analysed and the event occurred between the 2nd and 6th of July 2022. The main burst of the storm occurred on the 5th and 6th of July. Rainfall recorded at 9am on the 5th, 6th and 7th of July indicated rainfall totals of between 200 mm and 240 mm across the catchment. Higher rainfalls occurred in the west and lower rainfalls in the east, as shown in Figure 17. The sub-daily rainfall records at the two nearby gauges show a similar temporal pattern, although rainfall totals vary, as shown in Figure 18. A comparison of the sub-daily rainfall records with ARR 2019 IFD at the catchment centroid shows the rainfalls for this event were less than a 50% AEP event for durations less than 8 hours, with the maximum being a 20% AEP event for a 48 hour duration, as shown in Figure 19.

6.3. Recorded Flood Observations

There were flood marks available from Council, as outlined in Section 2.9. For the 1990 event, 36 flood marks were recorded and 40 flood marks for the 2007 event within the North Creek catchment. The North Creek water level gauge located upstream of Walker Street recorded a water level time series with a 15 minute temporal resolution for the 2022 event. These records provide the most accurate information that can be used to calibrate a flood model.

There were also 4 flood photos available from Reference 3 for the 1990 event, and 13 photos available from Reference 4 for the 2007 event. As part of the community consultation phase of this project (see Section 3), residents shared their knowledge of flooding in the North Creek catchment. Of the 11 responses, 3 were considered useful for validating modelled flood behaviour, with 2 of these referring specifically to the June 2007 event.

6.4. Event Simulation

The historic rainfall events were simulated in the WBNM hydrologic model using the spatial distributions of rainfall presented in Figure 11, Figure 14 and Figure 17 for the February 1990, June 2007 and July 2022 storm events. The rainfall total for each of the WBNM sub-catchments was sampled from the spatial rainfall grid produced for each event.

This rainfall depth was distributed temporally using the available sub-daily rainfall data from nearby gauges that were considered to be most representative of the rainfall over the catchment. There was not sufficient data to calibrate the hydrologic model or rainfall losses. As such, a joint calibration was undertaken of the WBNM model and the TUFLOW model. Design continuing losses were adopted and initial losses were varied in conjunction with the Mannings 'n' values in the TUFLOW model to calibrate the models. The adopted parameters for each storm event are outlined in Table 11 below.

Table 11: Simulation Parameters of the Historic Storm Events

Parameter	1990	2007	2022
Storm Start Date/Time	2/02/1990 9AM	8/06/2007 9AM	4/07/2022 9AM
Storm End Date/Time	7/02/1990 9AM	9/06/2007 9AM	7/07/2022 9AM
Temporal Pattern	Eleebana Reservoir 561158	Barnsley 561067	Eleebana Reservoir 561158
Initial Loss (mm)	7	4	18
Continuing Loss (mm/h)	1.08	1.08	1.08

The resultant runoff hydrographs produced by the WBNM model were then applied to the TUFLOW hydraulic model to simulate the flood behaviour for the duration of each event. In the TUFLOW model, it was assumed that stormwater pits were blocked by 20% for on-grade pits and 50% for sag pits. Culverts with headwalls were assumed to be blocked by 20%. The recorded water level at the Marmong Point gauge (211460) was adopted as the downstream boundary in Warners Bay. The model results were then compared to the flood marks, flood photographs, observations and water level gauge for the relevant events. The modelled flood depths for the 1990, 2007 and 2022 events are shown in Figure 20, Figure 21 and Figure 22, respectively.

6.5. Calibration Results

6.5.1. February 1990 Calibration Results

The calibration to the available flood marks for the 1990 event is summarised in Table 12, and shown in Figure 20. The flood marks were based on the information provided in Reference 3, with Council's flood mark database used to check locations and levels.

Table 12: February 1990 Calibration to Flood Marks

ID	Location	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
1	20 Lake Street	1.71	1.77	0.06
2	11 Charles Street	1.47	1.80	0.33
3	436 The Esplanade	1.76	1.73	-0.03
4	4 John Street	1.79	1.64	-0.15
5	12 John Street	1.60	1.64	0.04
6	6 John Street	1.50	1.64	0.14
7	53 Albert Street	1.97	1.97	0.00
8	55 Albert Street	1.94	1.99	0.05
9	49 Albert Street	1.81	1.94	0.13
10	45 Albert Street	1.81	1.89	0.08
11	17 Martin Street	2.29	2.25	-0.04
12	19 Martin Street	2.12	2.25	0.13
13	26 Martin Street	2.16	2.25	0.09
14	28 Martin Street	2.15	2.25	0.10
15	24 Martin Street	2.44	2.29	-0.15
16	52 Medcalf Street	3.82	3.79	-0.03
17	54 Medcalf Street	3.07	3.11	0.04
18	5/7 Walker Street	2.74	2.72	-0.02
19	7 King Street	5.25	5.48	0.23
20	14 Medcalf Street	4.75	4.94	0.19
21	12 Walker Street	3.21	3.10	-0.11
22	2-4 Margaret Street	2.37	2.47	0.10
23	3 Margaret Street	2.39	2.39	0.00
24	30 Martin Street	2.18	2.25	0.07
25	381 Hillsborough Road	6.63	6.58	-0.05

ID	Location	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
26	314 Hillsborough Road	7.70	7.73	0.03
27	342 Hillsborough Road	6.47	6.50	0.03
28	22 Walker Street	5.68	4.95	-0.73
29	82 Medcalf Street	7.37	NF	-
30	80 Medcalf Street	7.30	NF	-
31	76 Medcalf Street	6.65	NF	-
32	16 Albert Street	7.22	7.18	-0.04
33	2 Campbell Street	6.70	6.63	-0.07
34	6 Campbell Street	6.39	5.92	-0.47
35	14 Campbell Street	6.37	5.19	-1.18
36	20 Campbell Street	4.92	4.60	-0.32

Note: NF means 'Not Flooded'

Modelled flood levels for three marks (25, 26 and 27) upstream of King Street matched well with the observed levels, being within 0.05 m. There are two points (19 and 20) downstream of King Street that are approximately 0.2 m over. The previous Flood Study (Reference 3) noted that the site at 7 King Street was redeveloped in 2003 and hence a comparison of levels may not be valid. The location of the point for 14 Medcalf Street is also not certain, and the flood level along North Creek varies such that if the level was taken further downstream, a match could be obtained. At Walker Street (point 21) the flood level was matched. On the King Street Branch (point 28), the flood level was modelled to be 0.7 m lower than the observed. This may be due to the obstructions (such as fences and bridges) along this reach that were not explicitly modelled. Downstream of Walker Street, the modelled flood level is within 0.15 m of the observed flood level, across 20 points. There is only one point with a larger difference of 0.33 m (point 2). The flood level is modelled to be approximately 1.8 m on Charles Street and 1.65 m on John Street. The observed flood level of 1.47 mAHD is lower than the downstream flood levels observed on John Street (between 1.5 mAHD and 1.8 mAHD). It is unlikely that the flood level on Charles Street is reliable in this case.

There are 8 points bounded by Medcalf Street, Albert Street and Campbell Street that were observed to be affected by flooding. In the model, however, flow is simulated to occur within the streets only, with the exception of minor affectation on the corner of Albert Street and Campbell Street. It is thought that in reality, flooding on Medcalf Street overtopped the gutter and flowed through properties down to Campbell Street. This behaviour is not replicated in the model. The runoff from the upstream catchment does not spill over the Medcalf Street crest at this location. There are some locations that are not flooded in the model (points 30 and 31), some locations where the modelled flood level is much lower than the observed (for example the flooding modelled on Campbell Street is much lower than that which was observed at the properties which are at a higher elevation), and some locations where the flood level in the street is similar to that observed on the lot (for example points 32 and 33 along Albert Street).

The average difference across reliable flood points (neglecting points discussed above) is 0.02 m, with the model generally simulating flooding from North Creek reasonably well.

Below are photos from Reference 3 and commentary is provided about how the model replicates flooding observed in the photos.

Photo 38: 4 Margaret Street – 4 February 1990

The flooding at 4 Margaret Street is due to North Creek. The peak flood depth in the yard on the northern side of the house is modelled to be approximately 0.6 m deep. This appears to be reasonable given the peak flood level marked in the photograph. The model simulated a peak flood level 0.1 m higher than the observed flood mark 22 which is also at this location.

Photo 39: Medcalf/Albert Street – 2 February 1990

The flooding at this location was modelled to be approximately 0.05 m to 0.15 m deep. This appears to align well with the photograph, with generally shallow inundation on the road, and deeper flooding (up to gutter height) where the turn bay joins Medcalf Street. This is the location in the model where deeper flooding (0.15 m deep) is simulated.

Photo 40: The Esplanade Bridge – 4 February 1990

The modelled peak flood level at the Esplanade Bridge is approximately 1.02 m, the peak level recorded in Lake Macquarie. This is approximately 0.7 m from the soffit of the bridge, which appears to be a reasonable estimate of the flood level in the photograph.

Photo 41: Foreshore of Warners Bay – 4 February 1990

The model is not intended to simulate flooding of the foreshore from Warners Bay. This was simulated as part of Reference 6.

6.5.2. June 2007 Calibration Results

The calibration to the available flood marks for the 2007 event is summarised in Table 13, and shown in Figure 21.

Table 13: June 2007 Calibration to Flood Marks

ID	Location	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
1	59 Abert Street	2.07	2.08	0.01
2	38/3 Abert Street	2.16	1.99	-0.17
3	55 Albert Street	2.01	2.06	0.05
4	49 Albert Street	2.02	1.99	-0.03
5	50 Albert Street	2.13	2.09	-0.04
6	38/1 Albert Street	2.16	1.99	-0.17
7	37 Albert Street	2.16	1.97	-0.19
8	45 Albert Street	2.39	2.03	-0.36
9	23 Charles Street	1.86	2.03	0.17
10	22 Charles Street	1.94	1.91	-0.03
11	16 Charles Street	1.96	1.93	-0.03
12	18 Charles Street	1.98	1.91	-0.07
13	8 Charles Street	1.99	1.94	-0.05
14	19 Charles Street	1.99	1.91	-0.08
15	4 Charles Street	2.00	1.92	-0.08
16	6 Charles Street	2.00	1.92	-0.08
17	15 Charles Street	2.06	1.91	-0.15
18	342 Hillsborough Road	6.05	6.08	0.03
19	342 Hillsborough Road	6.65	6.62	-0.03

ID	Location	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
20	16 John Street	1.79	1.74	-0.05
21	4 Margaret Street	2.66	2.63	-0.03
22	3 Margaret Street	2.68	2.59	-0.09
23	17 Martin Street	2.33	2.27	-0.06
24	21 Martin Street	2.37	2.36	-0.01
25	28 Martin Street	2.40	2.36	-0.04
26	26 Martin Street	2.43	2.42	-0.01
27	22 Martin Street	2.90	2.81	-0.09
28	52 Medcalf Street	4.67	4.62	-0.05
29	18 Medcalf Street	5.47	5.35	-0.12
30	10 Medcalf Street	5.83	5.87	0.04
31	130 Medcalf Street	6.15	6.14	-0.01
32	27 Seaman Avenue	4.64	4.66	0.02
33	398 The Esplanade	1.71	1.61	-0.10
34	9/11 Walker Street	2.87	2.76	-0.11
35	9A Walker Street	2.91	2.88	-0.11
36	12A Walker Street	3.68	3.79	0.11
37	12 Walker Street	3.79	3.78	-0.01
38	278 Macquarie Road	5.69	5.89	0.20
39	King Street	5.37	5.87	0.50
40	12 John Street	1.99	1.74	-0.25
41	7 John Street	1.70	1.72	0.02

Note: NF means 'Not Flooded'

Modelled flood levels for two marks (18 and 19) upstream of King Street matched well with the observed levels, being within 0.03 m. At King Street, one flood mark at McDonalds (38) is modelled to be 0.2 m higher than the observed level. Another flood mark (39) was estimated to be 0.5 m higher than the flood mark. However, it appears that this flood mark is based on a photo of a car flooded on King Street, and the exact location of the car is unknown. Between King Street and Walker Street, there are four flood marks (29, 30, 36 and 37) which are all within 0.1 m of the recorded levels. Downstream of Walker Street, there are 27 marks all within 0.17 m of the recorded levels. There is one flood mark (8) where the modelled level was 0.36 m lower than the recorded level. However, the modelled water level in this area is approximately 1.95 mAHD to 2.0 mAHD, and the surrounding 7 flood marks (upstream and downstream) all match the recorded flood level within 0.17 m, and hence this is considered to be an outlier and the flood mark may not be reliable. There is one flood mark on Charles Street (9) that is approximately 0.17 m higher than the recorded level. At this location, the recorded flood level is lower than those affected by a relatively level pool of inundation from North Creek to the northwest. The ponded level (2 mAHD) would have extended up Charles Street to this property and hence the recorded level (1.86 mAHD) is not considered reliable. There is one flood mark on John Street (40) that is approximately 0.25 m lower than the recorded level. The adjacent property (20) recorded a higher flood level by 0.2 m that was matched well by the model. It is unlikely that both of these levels would be correct and it is considered that this flood mark (40) is not reliable. There are two flood marks located on the Seaman Avenue Branch (31 and 32) subject to overland flows, and these were matched well by the model (within 0.02 m).

The average difference across reliable flood points (neglecting unreliable points discussed above) is -0.04 m, with the model generally simulating flooding from North Creek well.

Below are photos from Reference 4 and commentary is provided about how the model replicates flooding observed in the photos.

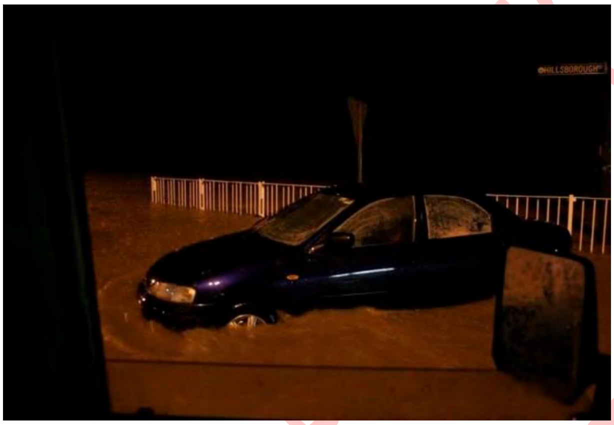


Photo 42: Flooding at the Hillsborough Road Roundabout – June 2007

The location of this photograph is uncertain. The modelled flood depth in the vicinity of the Hillsborough Roundabout is up to approximately 0.6 m. The flood depth observed in the photo is possible at a number of locations in the vicinity of the roundabout.

Photo 43: Flooding adjacent to McDonalds at Hillsborough Road – June 2007

As above.

Photo 44: Hillsborough Road – June 2007

As above.

Photo 45: Approximate flood height at 3 Margaret Street – June 2007

The modelled peak flood depth on the northern side of the building at 3 Margaret Street is approximately 0.8 m, which is likely to be lower than that shown in the photograph (perhaps 1 m). The match to the flood mark at this location (22) noted a modelled level approximately 0.1 m lower than that recorded.


Photo 46: Flood damage at 4 Margaret Street – June 2007

The modelled peak flood depth at 4 Margaret Street is approximately 0.7 m, and hence the damage observed is likely.


Photo 47: Flood water approaching floor level at Margaret Street – June 2007

The location of this photograph is uncertain (although assumed to be 4 Margaret Street). The modelled flooding at 4 Margaret Street encroaches on the property, although likely to not inundate the main floor level due to the height of the building, which aligns with the photograph. The match to the flood mark at 4 Margaret Street (mark 21) was very good (within 0.03 m).

Photo 48: Aftermath of flooding at Margaret Street – June 2007

The location of this photograph is uncertain, although assumed to be at 3 or 4 Margaret Street. Modelled peak flood depths at these properties is approximately 0.7 m, which is approximately the depth observed in the photograph using the wheelie bin for scale.

Photo 49: Charles Street – June 2007

The location of this photograph is uncertain. The modelled flood depth on Charles Street is up to 0.8 m, and hence the flood depth observed in the photo is possible at a number of properties on Charles Street.

Photo 50: Floodwater waist deep at Charles Street – June 2007

The location of this photograph is uncertain. The modelled flood depth on Charles Street is up to 0.8 m, which is approximately waist deep as observed. Flood marks on Charles Street are also matched well.

Photo 51: Approximate flood height at 28 Martin Street – June 2007

The modelled peak flood depth at 28 Martin Street is approximately 0.4 m. Given the height of the verandah and floor above ground (approximately 6 bricks), this matches well with the photograph. The flood mark at this location (25) was matched to within 0.04 m.

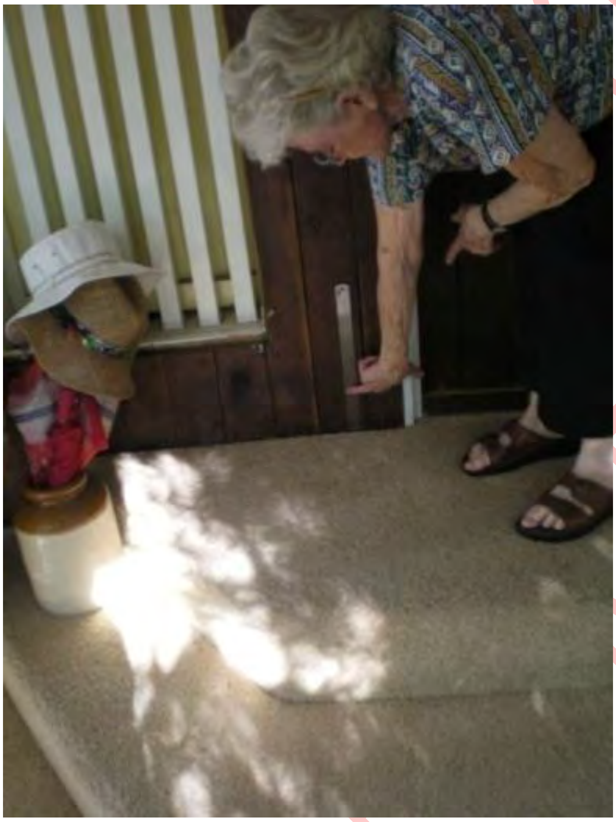


Photo 52: Approximate flood height at 26 Martin Street – June 2007

The modelled peak flood depth at 26 Martin Street is approximately 0.6 m. The flood mark at this location (26) was matched to within 0.01 m.

Photo 53: Flood damage at the Warners Bay netball courts – June 2007

The modelled peak flood depth at the Warners Bay netball courts is approximately 0.6 m – 0.8 m. This depth of flooding could cause the damage observed. The nearby flood marks (13, 15 and 16) were matched to within 0.08 m.

Photo 54: Warners Bay foreshore following the flood – June 2007

The model is not intended to simulate flooding of the foreshore from Warners Bay. This was simulated as part of Reference 6.

From the community questionnaire, one respondent stated that water “lapped over the door step at 3 Margaret St” for the 2007 event. This would align with the modelled flood level of approximately 2.6 mAHD at the front of the building and the floor level of approximately 2.5 mAHD (surveyed). Comparisons at 3 Margaret Street were also undertaken using a surveyed flood mark (ID 22) and Photo 45, which indicated a good match.

Two further community responses were also analysed (Figure 6). One was from Peachwood Close, in the west of the catchment. The respondent indicated that flooding occurs in the backyard every time there is heavy rain, and indicated that they had been affected in the February 1990, April 2001, June 2007 and April 2015 floods, with the “June” flood requiring damage to the pool to be rectified. For the 1990 and 2007 events, water is modelled to flow across this property, from the rear to the front, with peak flood depths up to 0.2 m through the property. The other respondent was from Nott Street near the centre of the catchment. The respondent indicated that flooding occurs at the rear of the property from upstream runoff, flowing across the backyard and down the sides of the building. This property was not modelled to have any significant runoff as the catchment draining to the property was considered too small to model. The runoff experienced at this location is considered to be ‘stormwater’ runoff rather than ‘flooding’ and may be exacerbated by local features such as inter-allotment drainage, garden beds, fences, etc. that would not be explicitly included in the flood model.

6.5.3. July 2022 Calibration Results

The flood behaviour for the July 2022 event can be seen in Figure 22. The only available data for this event was the Noth Creek water level gauge located at Walker Street. A comparison of the water level recorded by the gauge and the simulated water level for this event is shown in Diagram 5. The gauge water level rises to approximately 1.8 mAHD around the 25 hour mark and then fluctuates as it slowly falls over the next 12 hours. The modelled water level displays a more sensitive response to water levels, rising and falling in a quicker manner than the gauge. The simulated peak water level is up to approximately 0.3 m higher than the recorded level.

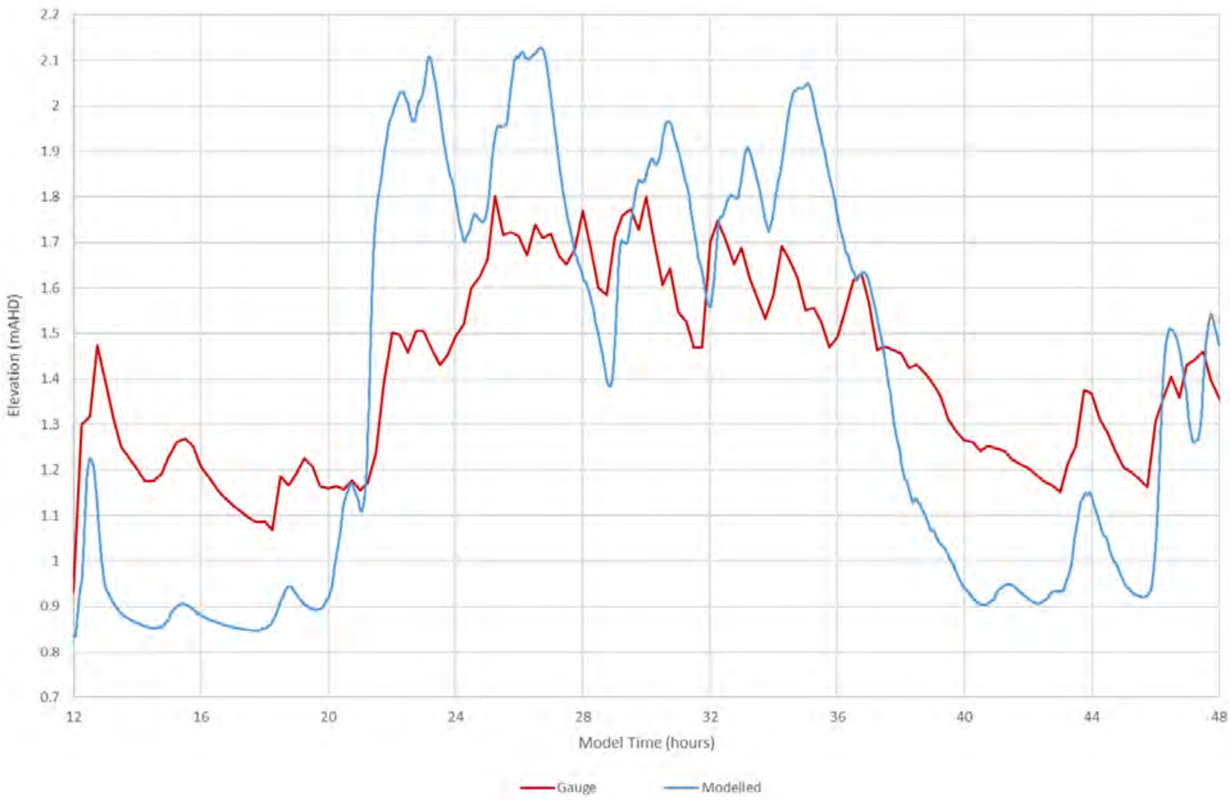


Diagram 5: Comparison of gauge and modelled water levels upstream of Walker Street

It is more difficult to match in-bank flood behaviour than out-of-bank flood behaviour (such as the 1990 and 2007 events) as there are features within the channel that can affect conveyance and capacity that may not be represented in the model. This includes changes and variations in channel morphology (such as invert levels and shape) and channel roughness (such as changes in vegetation). These features are dynamic and can also change during a flood event (such as scour and deposition of sediment or flattening of vegetation). Compounding to these issues is that the rainfall for small events can be more spatially variable than say an east coast low event (such as the June 2007 event). Thus, there is greater uncertainty about the actual rainfall over the catchment in small events. Considering these challenges of modelling the 2022 event, the calibration results are considered reasonable.

6.6. Calibration Summary

Overall, the comparisons between the modelled flood behaviour and the observed flood behaviour indicate a reasonable match was achieved. In particular, the calibration to the available flood marks for the 1990 and 2007 events was generally good (within 0.15 m). Where flood levels were not matched, it was considered more likely that the flood mark was not reliable. The model generally replicates the observed flood behaviour in flood photographs. The match with the water level gauge upstream of Walker Street was considered reasonable given the difficulties of simulating a relatively small in-bank event.

Overall, the model was considered to represent flood behaviour across the North Creek catchment, and in particular the flooding due to North Creek during large out-of-bank events. The models (WBNM and TUFLOW) are therefore considered appropriate for design flood event simulations.

7. DESIGN FLOOD MODELLING

7.1. Overview

ARR 2019 guidelines (Reference 7) for design flood modelling were adopted for this study. The new guidelines were first published in 2016, finalised in 2019 and present a significant update on the previous version published in 1987 (Reference 5). Since 1987, there have been numerous advances in the understanding of rainfall-runoff processes, technological advances and a larger set of recorded rainfall data available. This additional 30 years of data (from approximately 1985 to 2015), particularly for continuously recorded rainfall (pluviometers), allows for Australia-specific techniques and regionalised information to be used across the country. Specifically related to design flood modelling there is updated IFD information, design temporal patterns, areal reduction factors and rainfall losses to consider.

ARR 2019 guidelines were used to estimate the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP events. The PMF flows were derived using the BoM's Generalised Short Duration Method (GSDM, Reference 13) to estimate the probable maximum precipitation (PMP). A detailed critical duration analysis was undertaken to determine the most representative duration and temporal pattern across the catchment. The selected storm for each AEP event was then used to simulate the design flood behaviour.

This section outlines the design flood inputs and parameters that were used and the critical duration analysis.

7.1.1. Design Rainfall Depths

IFD information was obtained from the BoM using the 2016 design rainfall data. IFDs were obtained at the centroid of each sub-catchment to represent spatially varying rainfall. The design rainfall depths at the centroid of the catchment are shown in Table 14 and it is noted that the design rainfall depths do not vary significantly across the catchment.

Table 14: Design rainfall depths (mm) at the centroid of the North Creek catchment

Duration (min)	AEP							
	50%	20%	10%	5%	2%	1%	0.5%	0.2%
5	8.66	12.4	15.2	18.1	22.2	25.5	28.8	34.2
10	13.7	19.7	24.2	28.9	35.6	41.2	46.6	55.3
15	17.2	24.7	30.3	36.2	44.7	51.8	58.5	69.6
20	19.8	28.5	34.9	41.7	51.5	59.6	67.4	80.1
25	21.9	31.5	38.6	46.1	56.8	65.7	74.3	88.3
30	23.6	34	41.7	49.7	61.3	70.8	80.1	95.1
45	27.7	39.8	48.8	58.1	71.4	82.2	93.1	111
60	30.7	44.2	54.1	64.4	78.9	90.8	103	122
90	35.3	50.8	62.2	73.9	90.5	104	118	140
120	38.9	56.1	68.6	81.6	99.8	115	130	154
180	44.8	64.6	79.1	94	115	132	150	177
270	51.8	74.8	91.7	109	134	155	174	207
360	57.7	83.3	102	122	151	174	196	233

Note: Taken at centroid of the catchment, referencing IFD grid cell at 32.9625°S, 151.6625°E

7.1.2. PMP Rainfall Depths

The design rainfalls for the PMP were derived using the BoM's GSDM (Reference 13). The catchment terrain was estimated to be 'rough' with an elevation adjustment factor of 1 and a moisture adjustment factor of 0.72. The GSDM requires rainfall to be distributed spatially using ellipses. The ellipses were centred over the North Creek catchment, with approximately half of the catchment being within Ellipse 'A' and half being within Ellipse 'B'.

7.2. Temporal Patterns

Temporal patterns are a hydrologic tool that describe how rain falls over time and are used in hydrograph estimation. Previously, with ARR 1987 guidelines (Reference 5), a single temporal pattern was adopted for each rainfall event duration. However, ARR 2019 (Reference 7) discusses the potential inaccuracies with adopting a single temporal pattern and recommends an approach where an ensemble of different temporal patterns is investigated.

Temporal patterns for this study were obtained from the ARR 2019 data hub (Reference 14, http://data.arr-software.org/). A summary of the data hub information at the catchment centroid is presented in Attachment A. The revised ARR 2019 temporal patterns were introduced to address the key limitations of the ARR 1987 temporal pattern approach.

It is widely accepted that there are a large variety of temporal patterns possible for rainfall events of similar magnitude. This variation in temporal pattern can result in significant effects on the estimated peak flow. As such, the revised temporal patterns have adopted an ensemble of ten different temporal patterns for a particular design rainfall event and duration. Given the rainfall-runoff response can be quite catchment specific, using an ensemble of temporal patterns attempts to produce the median catchment response.

As hydrologic modelling has advanced, it is becoming increasingly important to use realistic temporal patterns. The ARR 1987 temporal patterns only provided a pattern of the most intense burst within a storm, whereas the ARR 2019 temporal patterns look at the entirety of the storm including pre-burst rainfall, the burst and post-burst rainfall. There can be significant variability in the burst loading distribution (i.e. depending on where 50% of the burst rainfall occurs an event can be defined as front, middle or back loaded). The ARR 2019 method divides Australia into 12 temporal pattern regions, with the North Creek catchment falling within the East Coast South region.

ARR 2019 provides 30 temporal patterns for each duration which are sub-divided into three temporal pattern bins based on the frequency of the events. Diagram 6 shows the three categories of bins (frequent, intermediate and rare) and corresponding AEP groups. The “very rare” bin is in the experimental stage and was not used in this flood study. There are ten temporal patterns for each AEP/duration in ARR 2019 that have been utilised in this study for the 50% AEP to 0.2% AEP events.

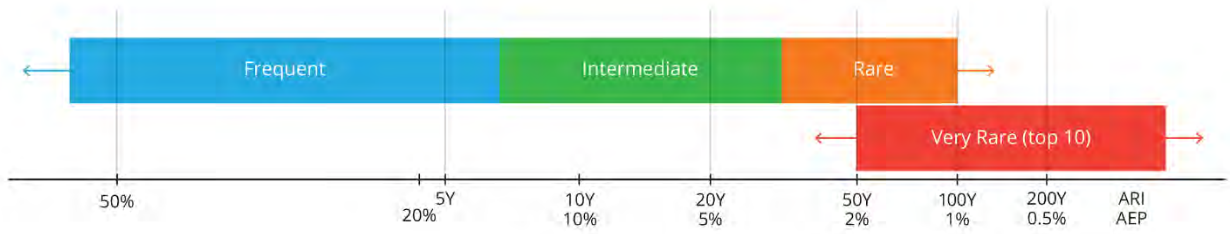


Diagram 6: Temporal Pattern Bins

The method employed to estimate the PMP utilises a single temporal pattern (Reference 13).

7.3. Design Rainfall Losses

The initial losses adopted for the calibration events were between 4 mm and 18 mm, with continuing losses fixed at 1.08 mm/hour. Sensitivity testing of the initial and continuing losses for the calibration events indicated low sensitivity to the adopted values, and as such, these are not considered to be ‘calibrated’ values. As such, for design flood modelling, the probability neutral burst initial losses from the ARR datahub (Reference 14) were adopted, in line with recent advice from the NSW Government (Reference 15). These initial losses were sourced from the ARR datahub at the centroid of each sub-catchment. The losses vary with storm duration and AEP, however, are generally in the range of 5 mm to 15 mm across the full range of AEPs and durations. The probability neutral burst initial losses at the catchment centroid are shown in Table 15 (see Attachment A for full matrix of losses).

Table 15: ARR datahub probability neutral burst initial losses (mm) at the catchment centroid

Duration (min)	AEP					
	50%	20%	10%	5%	2%	1%
60	10.6	8.5	9.4	9.2	10.2	6.0
90	11.2	9.6	10.5	10.1	10.5	5.9
120	11.0	9.5	9.5	9.9	9.7	6.5
180	12.2	10.3	10.6	9.9	9.3	3.5
360	11.3	7.4	7.9	7.5	9.3	2.9

Note 1: For AEPs rarer than 1% (i.e. 0.5% and 0.2%), the 1% AEP losses have been adopted

Note 2: For durations not listed in this table, losses were interpolated, or for durations less than 60 minutes, the 60 minute losses were applied

For continuing losses, the North Creek catchment is covered by two ARR data hub loss grid cells (eastern half and western half), with continuing losses ranging from 2.2 mm/h to 2.7 mm/h. Recent advice provided by the NSW Government (Reference 15) indicates that these losses should be factored by 0.4 for NSW catchments. This results in continuing losses of 0.88 mm/h to 1.08 mm/h across the catchment. Again, continuing losses were sourced from the ARR datahub at the centroid of each individual sub-catchment.

The PMP event adopted an initial loss of 1 mm and continuing loss of 0 mm/h.

7.4. Areal Reduction Factors

The design rainfall estimates are based on point rainfalls and in reality, the catchment-average rainfall depth will be less. Areal reduction factors (ARFs) allow for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms simultaneously over the whole catchment area. Given the nature of the study area and the focus on overland flow paths through urban areas, ARFs were not applied in the WBNM model. In accordance with ARR 2019 (Reference 7), catchments with an area up to 1 km^2 should not apply ARFs, and there is limited research on the applicability of ARFs to catchments that are less than 10 km^2. Given the size of the North Creek catchment, and in particular the tributaries of interest, no ARF was adopted. The total North Creek catchment to its outlet would have an ARF of approximately 0.95.

7.5. Downstream Boundary and Initial Conditions

As outlined in Section 5.4.2, a static water level was adopted for the downstream boundary at Lake Macquarie waterway where flows from the North Creek catchment can discharge into for all design flood events. Peak water levels in the Lake Macquarie waterway results from a combination of rainfall (ARR 1987 adopted in Reference 6) over the catchment and elevated ocean levels. Thus, the design flood levels in the Lake Macquarie waterway in Reference 6 were determined using an envelope approach (i.e. the peak level from a range of scenarios) of:

- The 48 hour duration design storm inflow in combination with an elevated tide, taken as a synthetic tide oscillating between 0 mAHD and 1 mAHD in 12.5 hour cycles representing a normal tide with a 0.4 m anomaly added uniformly, and
- The design ocean level/tide in combination with a low inflow (20% AEP inflow).

These are termed the rainfall-induced (design inflow event) and ocean-induced (design ocean level) flooding mechanisms. Comparison of these two flooding scenarios indicates that the rainfall-induced scenario produces the greater flood level in the Lake Macquarie waterway.

For the North Creek catchment, local flooding is due to a localised rainfall event over the catchment and may or may not be associated with a similar rainfall event over the entire Lake Macquarie waterway catchment or an ocean induced event. As an example, the June 2007 event produced elevated lake levels (lake flooding) as well as local flooding in the North Creek catchment. However, the timing of the local catchment peak flood levels and the peak levels in Lake Macquarie waterway were not coincident. Typically peak local catchment flood levels occur within 1 to 2 hours of the most intense rainfall burst while the peak level in Lake Macquarie waterway occurs approximately 12 hours or even longer after the most intense rainfall burst. The latter occurs because it takes several hours for runoff to pass through the larger tributary catchments that provide most of the runoff volume (such as Dora Creek or Cockle Creek) and then fill Lake Macquarie waterway.

Typically in flood studies of larger tributary catchments surrounding Lake Macquarie waterway the 1% AEP runoff from the local catchment is coincided with the 5% AEP flood level in Lake Macquarie waterway. For smaller more frequent local runoff events a smaller coincident flood level in Lake Macquarie waterway is appropriate.

For this study, which is principally investigating local catchment flooding, all design events have assumed a 2 year ARI lake level of 0.65 mAHD in the Lake Macquarie waterway. The average lake level is approximately 0.1 mAHD and hence the adopted level represents an elevated level. By way of comparison, the 1990 event level in Lake Macquarie at the time of the North Creek peak discharge was approximately 0.17 mAHD, with the peak lake level of 1.02 mAHD occurring some 48 hours later. In the 2007 event, the level in Lake Macquarie at the time of the North Creek peak discharge was approximately 0.30 mAHD, with the peak lake level of 1.14 mAHD occurring some 8 hours later. As such, the level of 0.65 mAHD was considered appropriate. Sensitivity analysis was undertaken to assess the effects of this tailwater level in Section 9.6.

The catchment was assumed to be dry at the start of the storm, with the exception of Lake Macquarie waterway and the Lakelands Pond (assumed to be at the grated pit outlet level).

7.6. Blockage

ARR 2019 (Reference 7) recommends applying blockage to hydraulic structures and outlines a methodology to determine inlet blockage factors by considering debris availability, debris mobility, debris transportability and waterway opening of the structure. This assessment was undertaken considering the typical culvert structures found in the study area. These structures fell into two categories, for which AEP dependent blockages were estimated in accordance with ARR 2019 procedures. The blockage factors from this analysis are shown in Table 16.

Table 16: Blockage assessment results of key hydraulic structures

AEP	Blockage of small[1] structures	Blockage of large[2] structures
More frequent than 5%	25%	10%
5% to 0.5%	50%	20%
Rarer than 0.5%	100%	20%

[1] Generally smaller structures with a diameter or width less than 1.2 m (the assumed L_{10}), with typically a medium 1% AEP debris potential

[2] Generally larger structures with a diameter or width greater than or equal to 1.2 m (the assumed L_{10}), with typically a high 1% AEP debris potential

A single blockage factor was considered appropriate across the range of design flood events, with the adopted blockage factors outlined in Table 17. This includes the culverts discussed above, in addition to pit inlets. A nominal blockage factor was also adopted for bridge structures.

Table 17: Adopted Blockage Factors

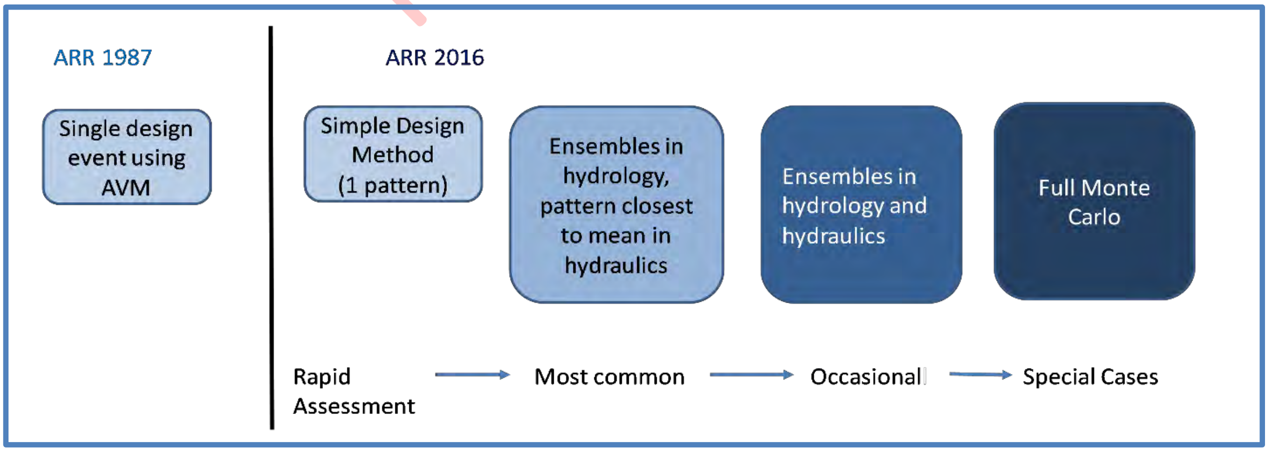
Structure	Design Blockage (%)
Small Culverts (Dia/Width < 1.2 m)	50%
Large Culverts (Dia/Width ≥ 1.2 m)	20%
Sag Pit	50%
On-grade Pit	20%
Bridge	5%

7.7. Critical Duration Assessment

7.7.1. Approach

ARR 2019 requires an ensemble of temporal patterns to be run for each AEP and duration combination, and the 'occasional' approach was adopted for the North Creek catchment, as shown in Diagram 7.

Diagram 7: Design modelling techniques for an ensemble of temporal patterns (Reference 7)



This approach requires the ensemble of temporal patterns to be run in both the hydrologic and hydraulic models. This approach was adopted due to the complex nature of the shallow overland flow paths through the urban areas, which is of interest to the study.

7.7.2. Critical Duration

The critical duration is the storm duration that best represents the flood behaviour (e.g. flow or level) for a specific design magnitude at a particular location. It is generally related to the catchment size, as flow takes longer to concentrate at the outlet from a larger catchment, as well as other considerations such as land use, shape, stream characteristics, etc.

With ARR 2019 methodology, the mean flow (or level) is computed from the ensemble of temporal patterns for each duration. The critical storm duration for a location of interest is then the design storm duration that produces the highest mean flow (or level). Where there are multiple locations of interest with different contributing catchment sizes, there can be multiple critical durations that need to be considered.

7.7.3. Representative Storm Burst

Once the critical duration is established, it is usually desirable to select a representative design storm temporal pattern that reproduces this behaviour for all points of interest. This representative storm can then be used for determining design flood behaviour and for future modelling to inform floodplain management decisions. This is typically the storm that produces the next highest flow (or level) above the average (from the ensemble of temporal patterns) for the critical duration. In most cases, however, a representative storm does not necessarily need to be of the same duration as the critical duration, and there may be a number of storms that can represent the critical duration behaviour, potentially at multiple locations and even where the critical duration varies.

Adopting a range of critical durations across a catchment can complicate future analysis and the use of modelling tools if multiple storms need to be simulated to obtain the design flood behaviour for a particular event. Thus, it is preferable to adopt a single representative storm that is similar to the critical duration behaviour across the entire catchment for each event where possible.

7.7.4. Representative Storm Selection

To select the representative storm for each AEP for the North Creek catchment, the WBNM hydrologic model was run for durations from 10 minutes to 6 hours, with the ensemble of temporal patterns for the 20% AEP, 5% AEP and 1% AEP events (representative of each temporal pattern bin). Each of these storms was then simulated in the TUFLOW model. For each duration, a grid of the mean peak level at each grid cell was calculated. A maximum envelope grid was then calculated taking the highest mean peak level for each grid cell. This shows the critical duration mean peak level at all flooded cells across the study area. The source of the peak mean level for each grid cell was mapped to show the variation in critical duration across the catchment. The critical duration figures are shown in Figure 23, Figure 24 and Figure 25 for the 20% AEP, 5% AEP and 1% AEP events, respectively. The majority of overland flow areas in the upper catchment had a critical duration of 20 minutes, which transitions to approximately 45 to 60 minutes when creeks begin to form, such as on the Western Tributary and the King Street Branch. Downstream of King Street, North Creek has a critical duration of 120 minutes (2 hours) in the 20% AEP event, 180 minutes (3 hours) in the 5% AEP event and 60 minutes (1 hour) to 90 minutes (1.5 hours) in the 1% AEP event. There are several small flood storage areas within detention basins that have a longer critical duration although typically still less than 3 hours.

A histogram of the number of cells (frequency) for each duration is shown in Diagram 8 for each of the events simulated. This indicates that the majority of the study area is dominated by the 45 to 90 minute storms, with the 20 minute also being prevalent across large areas of overland flow in the upper catchment.

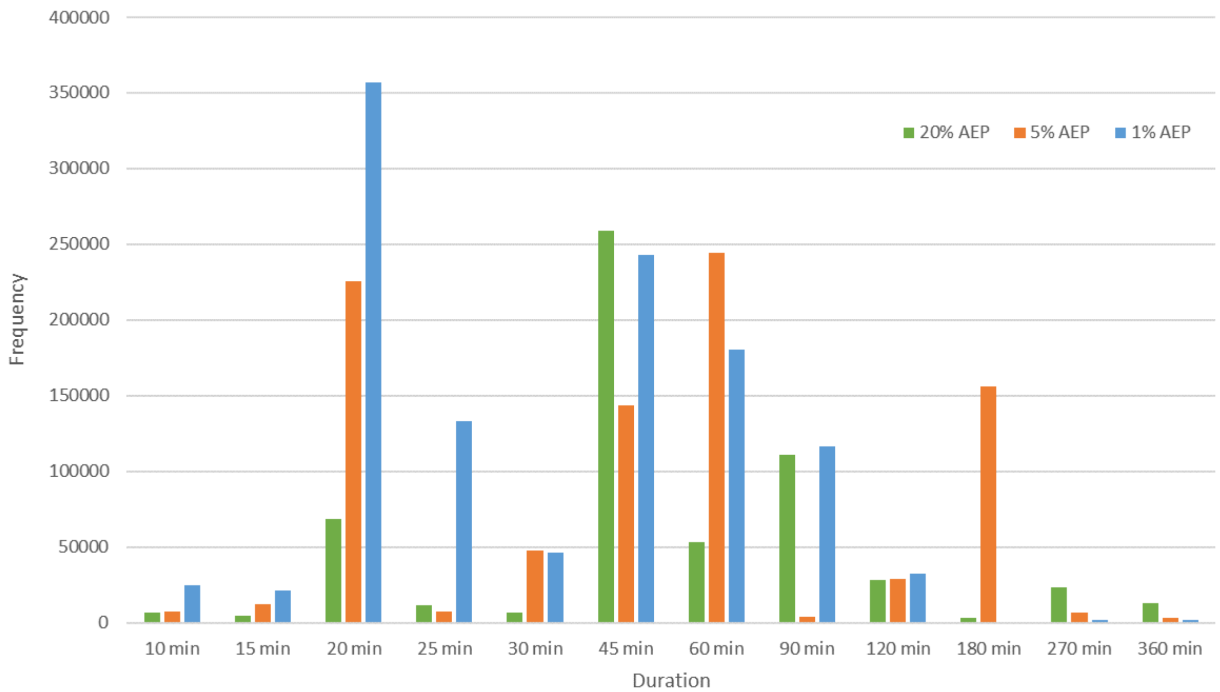


Diagram 8: Histogram of critical durations across the study area

Through a comparison of the peak flood level grid for each storm with the critical duration mean peak level across the entire study area, a representative storm was selected for each AEP event simulated. This was a temporal pattern from the 60 minute or 90 storms durations which were able to represent the critical duration in North Creek, while also being representative of flooding in upstream areas. Although the critical duration in the upstream areas was around 20 minutes, the shallow overland nature of flooding in these areas means that there is very little change in peak flood levels across different durations or even different AEPs. The selected storms result in minimal variation in peak water level from the critical duration mean peak level. The selected storm typically results in flood levels within ±0.03 m, with a tendency to be slightly over. In some of the basins, due to the nature of the short duration event, peak water levels are up to 0.2 m lower than the critical duration for the more frequent events. In the 1% AEP event, however, the levels are within 0.05 m and the downstream behaviour remains largely representative. This difference in level is shown in Figure 26, Figure 27 and Figure 28 for the 20% AEP, 5% AEP and 1% AEP events, respectively.

A similar, but simplified approach was undertaken for the PMF event, whereby a single storm was run for durations from 15 minutes to 150 minutes (2.5 hours). The results indicated that the 15 minute storm is critical for upstream overland flow paths, while the 30 minute is typically critical for tributaries and the 60 minute (1 hour) and 90 minute (1.5 hour) storms are critical for North Creek, as shown in Figure 29. For the purpose of this study, the 30 minute and 90 minute storms were selected as representative of PMF inundation across the North Creek catchment. The maximum envelope of these two durations was taken to produce the PMF results.

The selected storms were considered representative for all design events within that temporal pattern bin (Diagram 6). The selected storms were adopted for modelling of the design flood events and processing of flood results (as described in Section 8). The adopted representative design storms are summarised in Table 18.

Table 18: Adopted Representative Design Storms

Temporal Pattern Bin	Events	Duration (mins)	Temporal Pattern ID (Ensemble No.)
Frequent	50% AEP 20% AEP	90	4602 (2)
Intermediate	10% AEP 5% AEP	60	4573 (10)
Rare (2% AEP to 0.2% AEP)	2% AEP 1% AEP 0.5% AEP 0.2% AEP	60	4463 (3)
N/A	PMP	30, 90	GSDM

8. DESIGN FLOOD EVENT RESULTS

8.1. Overview

The 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP events were simulated using the adopted representative design storms. The PMF event was also simulated using the 30 minute and 90 minute PMP storms. The storms were run in the WBNM model and the resulting flows were input into the TUFLOW model to simulate flood behaviour across the study area. The results for the design flood events are presented in the following figures (found in Appendix C):

- Peak flood depths and levels in Figure C1 to Figure C9;
- Peak flood levels in Figure C10 to Figure C18;
- Peak flood velocities in Figure C19 to Figure C27;
- Hydraulic hazard in Figure C28 to Figure C31;
- Flood Function in Figure C32 to Figure C35;
- Flood emergency response classification of communities in Figure C36; and
- Preliminary flood planning area in Figure C37.

These results are available in electronic GIS and tabular format. The digital data should be used in preference to the figures in this report as they provide more detail. The figures are intended to provide an overview of the results and should not be relied upon for detailed information at individual properties.

Additional results are presented in the following tables and graphs (found in Appendix D, with locations shown in Figure 30):

- Peak water level profiles in Figure D1 to Figure D6;
- Stage hydrographs at road crossings in Figure D7 to Figure D37;
- Peak flows on North Creek and tributaries in Table D1;
- Detention basin performance (peak water level and flows) in Table D2; and
- Peak flood levels, depths and flows at road crossings and key locations in Table D3, Table D4 and Table D5, respectively.

A discussion of these results is provided in the following sections.

8.2. Summary of Results

The flood behaviour across the North Creek catchment can be seen in the peak flood depth figures (Figure C1 to Figure C9), peak flood level figures (Figure C10 to Figure C18) and peak velocity figures (Figure C19 to Figure C27) in Appendix C. These results are presented for the range of design flood events modelled from the 50% AEP to the PMF event. A tabulated summary of peak flood levels, depths and flows at selected locations, as shown in Figure 30, are detailed in Table D1 to Table D5 of Appendix D.

In frequent events, the flow is generally contained within the creeks and channels, with shallow overland flows (< 0.15 m deep) evident on streets as water moves toward the creeks. This flow is typically contained within the gutters and dedicated drainage reserves across the catchment. In the 50% AEP event there are areas of ponding within the industrial area to the north of Hillsborough Road, between East Street and Chartley Street, to the west of New Road. Medcalf Street, Queen Street, Walker Street and Wilton Close have flood depths between 0.2 m and 0.4 m. Low-lying areas near North Creek (such as John Street, Charles Street, Martin Street and the Warners Bay netball courts) experience inundation. In the 20% AEP event, inundation of roads is more extensive, although still fairly shallow. Tributaries such as the King Street Branch upstream of Queen Street become continuous through properties. Inundation from North Creek spreads to the east (between Margaret Street and John Street) and west (between Albert Street and Charles Street).

There is increased inundation with rarer events. In the 1% AEP event flood depths exceed 0.5 m across a large area adjacent to North Creek downstream of Walker Street on both sides of the channel. There is inundation of properties downstream of the Lakelands Pond, in the industrial area north of Hillsborough Road and along the King Street Branch (downstream of Wilton Close). Properties on the western side of the catchment (Seaman Avenue Branch and Western Tributary) are subject to comparatively shallow inundation as flow paths are more dispersed than the western side of the catchment. Depths reach between 0.5 m and 1 m on several roads within the catchment.

In the PMF event there are extensive inundation of areas adjacent to North Creek, typically between 1 m and 2 m deep. Along tributaries a large number of properties are impacts and water ponds on roads at low points to substantial depths (typically more than 0.5 m).

8.3. Tributary and North Creek Flows

Peak flows at several locations along North Creek in addition to flows contributed by the tributary branches are provided in Table D1 in Appendix D. A summary of the 1% AEP peak flows is provided in Table 19 below. The reported locations are shown in Figure 30.

Table 19: Summary of North Creek and Tributary 1% AEP Peak Flows

ID[1]	Location	1% AEP Peak Flow (m^3/s)
T01	Seaman Avenue Branch	5.8
T02	Western Tributary	8.3
T03	Lakelands Branch	17.1
T04	King Street Branch	13.6
T05	Vermont Place Branch	16.3
N01	North Creek at Hillsborough Road service road	4.3
N02	North Creek at King Street	31.6
N03	North Creek at Walker Street	37.6
N04	North Creek at Albert Street	56.7
N05	North Creek at the Esplanade	67.8

1. Locations shown on Figure 30

8.4. North Creek Gauge at Walker Street Design Results

The design flood results at the North Creek gauge at Walker Street are presented in Table 20. Walker Street is at a level of approximately 3 mAHD and as such water starts spilling onto the road in the 20% AEP event, with full overtopping occurring in the 10% AEP event.

Table 20: Summary of Design Flood Results at the North Creek Gauge

Event	Peak Water Level (mAHD)	Peak Flow (m^3/s)
50% AEP	2.37	9.9
20% AEP	2.97	16.0
10% AEP	3.23	20.0
5% AEP	3.39	24.4
2% AEP	3.58	30.6
1% AEP	3.72	37.6
0.5% AEP	3.85	44.8
0.2% AEP	4.04	56.9
PMF	5.46	156.3

An indicative rating curve for the gauge is also provided. Note that this rating curve is derived from design flood results on the rising limb of the hydrograph up to the PMF event, assuming a tailwater level of 0.65 mAHD. Higher tailwater levels may influence the rating curve at the gauge and the rating curve displays some hysteresis on the falling limb of the hydrograph that is not shown in this graph.

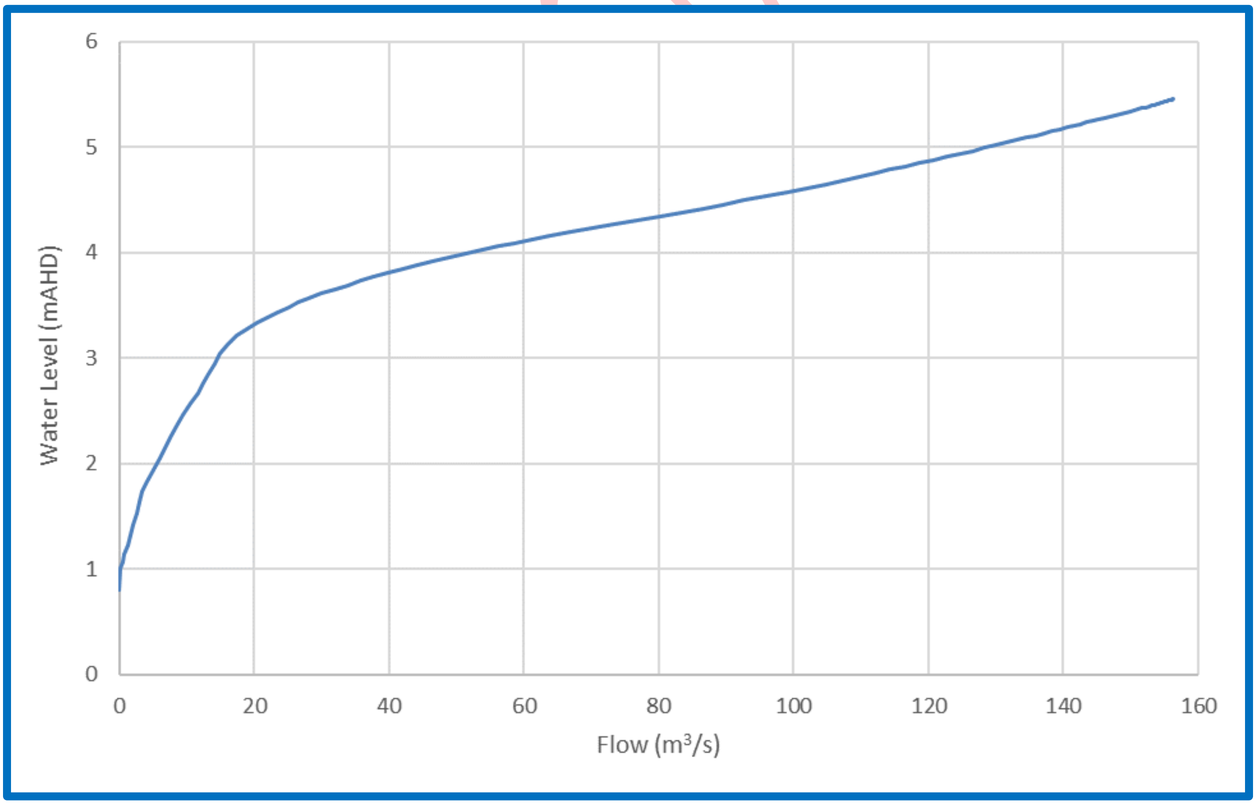


Diagram 9: Indicative rating curve for the North Creek Walker Street gauge

8.5. Detention Basins

There are several detention basins located throughout the North Creek catchment including the Biddibah Creek wetlands and Lakelands Pond. A summary of peak flood levels and outflows from each basin is provided in Table D2 in Appendix D. The locations of these basins are shown in Figure 30. A summary of when the basin overtops (via a high flow weir structure or the crest overtopping) is provided in Table 21. Note that detailed designs for these basins were not available and hence some assumptions about outlet structures (such as elevated pit outflow structures) were made. As such the actual overtopping characteristics may be different to that simulated.

Table 21: Summary of Basin Performance

ID[1]	Location	Event in which overtopping first occurs
B01	Inala Street	50% AEP
B02	Biddibah Wetland	1% AEP
B03	Burgin Way	50% AEP
B04	Lakelands Pond	10% AEP
B05	Whitehaven Drive	20% AEP
B06	Vermont Place	2% AEP
B07	Forrester Close	10% AEP
B08	Aurora Circuit	5% AEP
B09	New York Avenue	50% AEP
B10	Biddibah Avenue	5% AEP
B11	Wellham Close	50% AEP
B12	Wilton Close	20% AEP

1. Locations shown on Figure 30

8.6. Hydraulic Hazard Categorisation

Hydraulic hazard is a measure of potential risk to life and property damage from flooding. Hydraulic hazard is typically determined by considering the depth and velocity of floodwaters. In recent years, there have been a number of developments in the classification of hazards. Research has been undertaken to assess the hazard to people, vehicles and buildings based on flood depth, velocity and velocity-depth product. ARR 2019 (Reference 7) contains updated recommendations regarding the categorisation of flood hazard. A summary of this categorisation is provided in Diagram 10. This categorisation is based on an extensive literature review and laboratory testing. It considers hazard to people, vehicles and buildings to develop six categories of flood hazard based on flood depth, velocity and velocity-depth product.

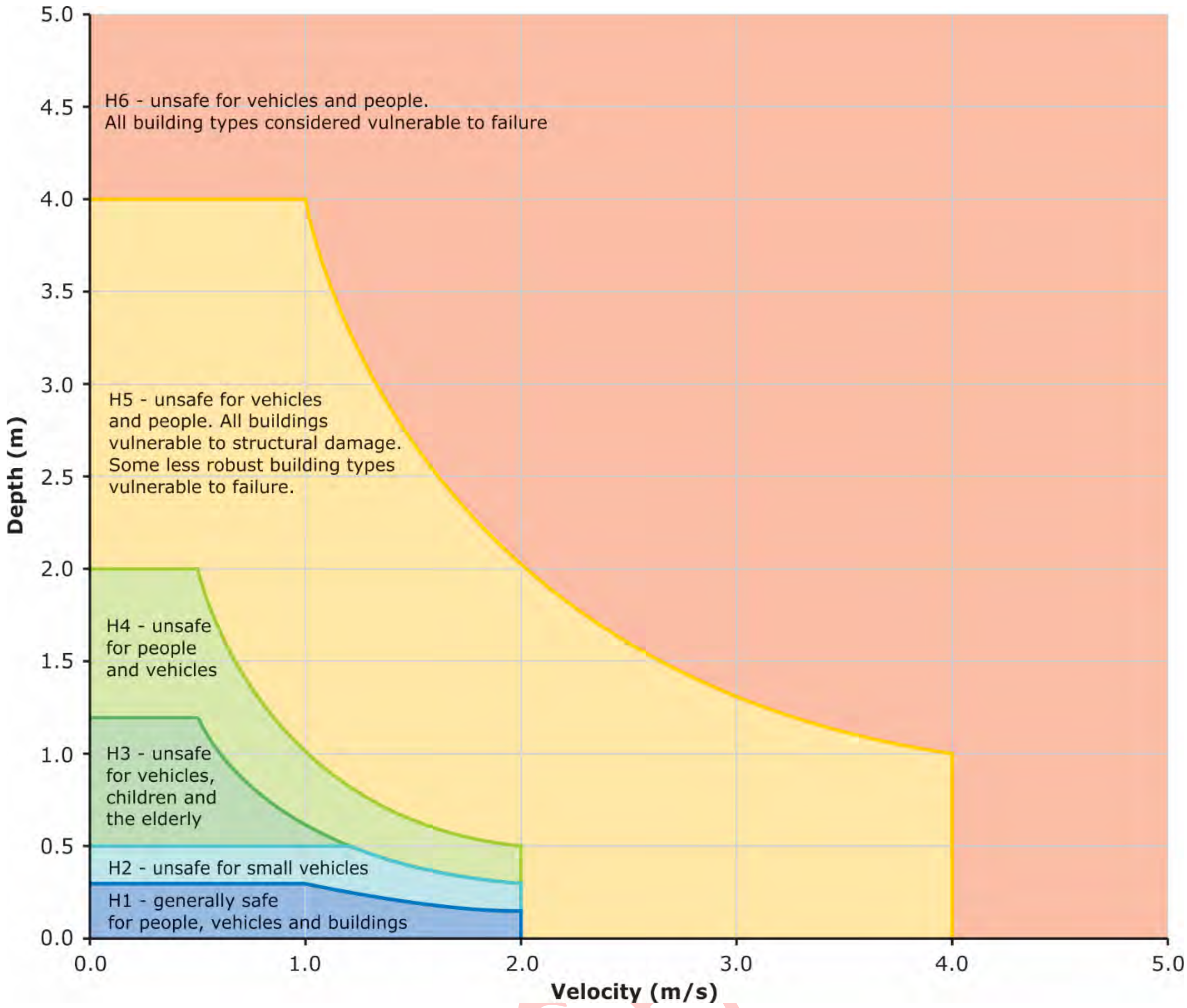


Diagram 10: General flood hazard vulnerability curves (Source: Reference 7)

The following 6 classes of hazard are defined:

- H1 – Generally safe for vehicles, people and buildings;
- H2 – Unsafe for small vehicles;
- H3 – Unsafe for vehicles, children and the elderly;
- H4 – Unsafe for vehicles and people;
- H5 – Unsafe for vehicles and people. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure; and
- H6 – Unsafe for vehicles and people. All building types considered vulnerable to failure.

The hazard categories using the ARR 2019 classification are mapped in Figure C28 to Figure C31 for the 5% AEP, 1% AEP, 0.2% and PMF events. In the 5% AEP event, much of the urban area affected by flooding is only H1 hazard, with areas of higher hazard (H3 and above) generally restricted to basins and creek channels (reaching H5). There are some areas of higher hazard outside these areas including the around the Warners Bay netball courts, between Charles Street and John Street, the end of Martin Street, between East Street and Chartley Street and small areas of the industrial area north of Hillsborough Road.

In the 1% and 0.2% AEP events the higher hazard areas increase, covering a large portion of the North Creek floodplain downstream of Walker Street. Roads along the upstream portions of the King Street Tributary convey high hazard flows. In the PMF event, the North Creek channel becomes H6 hazard and the surrounding downstream floodplain is H5 hazard. H5 hazard is also evident on other tributary branches and roads that convey flows.

8.7. Flood Function

Flood function (or hydraulic categorisation) involves mapping the floodplain to indicate which areas are most important for the conveyance of floodwaters and the temporary storage of floodwaters. This can help in planning decisions about which parts of the floodplain are suitable for development, and which areas are not restricted to ensure that flooding impacts are not worsened compared to existing conditions.

The Flood Risk Management Manual (NSW Department of Planning and Environment 2023) defines three hydraulic categories which can be applied to different areas of the floodplain depending on the flood function:

- Floodways;
- Flood Storage; and
- Flood Fringe

Floodways are generally areas which convey a significant portion of water during floods and are particularly sensitive to changes that impact flow conveyance. They often align with naturally defined channels. Flood storage areas are located outside of floodways and generally store a significant proportion of the volume of water. Flood behaviour in these areas is sensitive to changes that impact on the storage of water during a flood. Flood fringe areas are within the extent of flooding for a particular event but are outside floodway and flood storage areas. The flood fringe is less sensitive to changes in either flow conveyance or storage.

There is no quantitative definition of these three categories or accepted approach to differentiate between the various classifications. The delineation of these areas is somewhat subjective based on knowledge of an area and flood behaviour, hydraulic modelling and previous experience in categorising flood function. A number of approaches, such as that of Howells et al (2003), rely on combinations of velocity and depth criteria to define the floodway.

For this study, hydraulic categories were defined by the following criteria and is considered to be a reasonable representation of the flood function of this catchment:

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

  The remainder of the floodplain is either Flood Storage or Flood Fringe;
- Flood Storage comprises areas outside the floodway where peak depth > 0.2 m; and
- Flood Fringe comprises areas outside the Floodway where peak depth < 0.2 m.

The adopted parameters are consistent with those derived by WMAwater for similar catchments draining to the Lake Macquarie waterway. The flood function defined in this study are considered to be 'preliminary' and subject to review and refinement in a subsequent FRMS.

The hydraulic categories have been mapped in Figure C32 to Figure C35 for the 5% AEP, 1% AEP, 0.2% AEP and PMF events. As expected, the creeks and major flow paths are classified as floodways in the 5% AEP, 1% AEP and 0.2% AEP events, with flood storage areas where there are basins, around the Hillsborough Road roundabout and in the downstream low-lying floodplain of North Creek. In the PMF event, the floodway is quite extensive for North Creek, encompassing much of the floodplain. As tributaries spill out of channels the floodway also follows the wider flow paths. Flood storage areas are primarily found adjacent to the North Creek floodways with only shallow overland flow remaining as flood fringe.

8.8. Flood Emergency Response Planning

8.8.1. Road Inundation

There are numerous local roads throughout the study area that are subject to inundation. The inundation is typically shallow as overland flows are conveyed along road corridors. In some areas, where major flow paths cross roads or at sag points, however, the depth of inundation can be significant. At some of the major road crossings and sag points, water level hydrographs have been provided in Figure D7 to Figure D37 of Appendix D. Peak flood levels, depths and flows at key locations are also provided in Table D3, Table D4 and Table D5, respectively. The locations of these road crossings are shown in Figure 30.

In the 50% AEP event, Medcalf Street, Queen Street, Walker Street and Wilton Close have flood depths between 0.2 m and 0.4 m. in the 20% AEP event, a large number of roads have flood depths between 0.2 m and 0.3 m, with depths reaching over 0.5 m at Medcalf Street (downstream of Lakelands Pond). In the 5% AEP event, depths of approximately 0.7 m are estimated at Medcalf Street (downstream of Lakelands Pond), Hillsborough Road service road, King Street at North Creek and Martin Street, with other roads being less than this. In the 1% AEP event, flood depths reach 1 m at these locations. In the PMF all the roads at the reported locations are inundated, with depths approaching 3 m in some locations.

The rate of rise at each of the road crossings is reasonably quick, typically reaching peak levels within 1 hour of the onset of rainfall. This is driven by the quick catchment response and the adopted critical duration storms (60 to 90 minutes). It also means, however, that the duration of inundation is also short, with flooding typically lasting less than 1.5 to 2 hours.

8.8.2. Flood Emergency Response Classification

The Flood Risk Management Manual (Reference 2) requires flood studies to address the management of continuing flood risk to both existing and future development areas. As continuing flood risk varies across the floodplain, so does the type and scale of the emergency response problem and therefore the information necessary for effective Emergency Response Planning (ERP). Classification provides an indication of the vulnerability of the community in flood emergency response and identifies the type and scale of information needed by the NSW State

Emergency Service (SES) to assist in ERP.

The Flood Emergency Response Classification (FERC) for the study area was undertaken in accordance with the *Australian Disaster Resilience Handbook 7 Managing the Floodplain: A guide to best practice flood risk management in Australia* (Reference 16). FERC classifications consider flood affected communities as those in which the normal functioning of services is altered, either directly or indirectly, and results in the need for external assistance. This impact relates directly to the operational issues of evacuation, resupply and rescue, which is coordinated by the SES.

The ERP classification for urban areas within the North Creek catchment were defined using the PMF flood event as shown in Figure C36. The classification has been undertaken on a precinct basis rather than lot-by-lot and is targeted at highlighting those areas which may require evacuation or assistance during a flood event. However, these classifications may vary depending on local flood characteristics and resultant flood behaviour, i.e. in flash flooding or overland flood areas. These categories are described in Diagram 11 below.

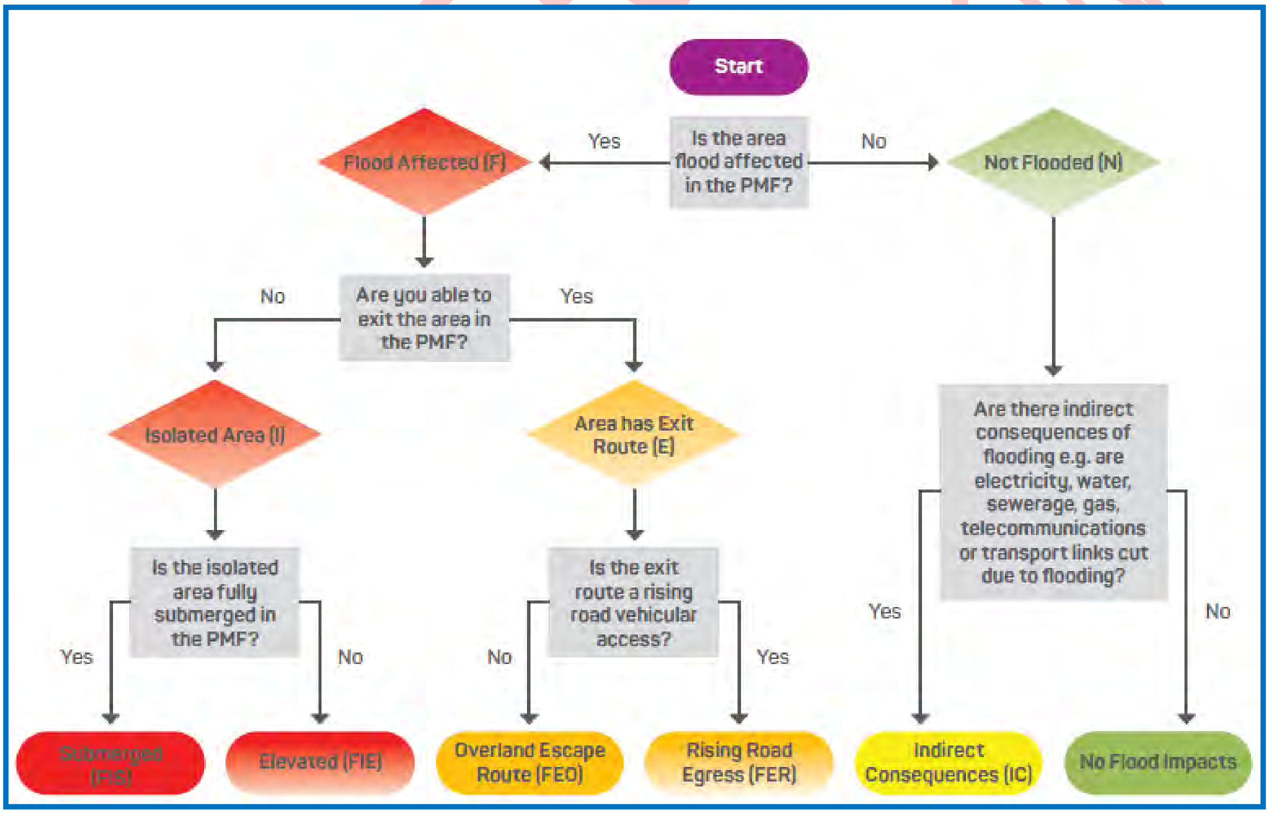


Diagram 11: Flow Chart for Determining Flood Emergency Response Classifications (Reference 16)

A summary of the FERC for North Creek is as follows:

- An area approximately 300 m wide adjacent to North Creek from the Hillsborough Road service road to the Esplanade is submerged (FIS)
- Areas that are isolated (FIE) include the industrial area north of Hillsborough Road, a residential area around Albert Street, Martin Street and Campbell Street, residential area between The Esplanade and New Road, Hughes Avenue, Milloba

Close, between Albert Street and The Esplanade (adjacent to Lake Street) and along the King Street Branch, from East Street to Walker Street.

- There are large areas of residential and commercial land that is affected by flooding, but has rising road access (FER) away from North Creek including east of Fairfax Road, the Hillsborough Road commercial area and around King Street.
- A number of open spaces and reserves may be affected by flooding and have adjacent roads that are inundated, however, typically access is available from these areas on foot (FEO).
- There are numerous other locations where roads are cut and people may be isolated, however, these areas are not directly affected by floodwaters and hence are classified as indirectly affected (IC).

8.9. Flood Planning Area

8.9.1. Background

Land use planning is an effective means of minimising flood risk and damages from flooding. Land use planning for flooding can be achieved through the use of:

- A Flood Planning Area (FPA), which identifies land that is subject to flood related development controls; and
- A Flood Planning Level (FPL), which identifies the minimum floor level applied to residential development proposals within the FPA.

Defining FPAs and FPLs in urban areas can be complicated by the variability of flow conditions between mainstream and local overland flow. Traditional approaches developed for riverine or "mainstream" flow areas often cannot be applied in steeper urban overland flow areas. Additionally, defining the area of flood affectation due to overland flow (which by its nature includes shallow flow) involves determining at which point flow is significant enough to be classified as "flooding" rather than just a drainage or local runoff issue. In some areas of overland flow, the difference in peak flood level between events of varying magnitude can be so minor that applying the typical freeboard can result in an FPL greater than the PMF level.

The FPA should include properties where development would result in impacts on flood behaviour in the surrounding area and in areas of high hazard where there is a risk to safety or life. The FPL is determined in addition to this with the purpose of decreasing the likelihood of damage such as over-floor flooding of houses.

The Flood Risk Management Manual (Reference 2) identifies that the FPL is generally based on the 1% AEP event plus an appropriate freeboard (typically 0.5 m). However, it also recognises that different freeboards may be deemed appropriate due to local conditions provided adequate justification is provided. For North Creek, the 1% AEP event with 0.9 m sea level rise was adopted.

Further consideration of flood planning areas and levels is typically undertaken as part of the Floodplain Risk Management Study to determine what should be included in the Floodplain Risk Management Plan and as such, the FPA derived as part of this study is considered to be preliminary.

8.9.2. Methodology

The methodology used for defining the FPA is consistent with that adopted in similar studies throughout the Lake Macquarie LGA. It divides the flood area between "mainstream" and "overland" flooding areas using the following criteria:

- Mainstream flooding: In these areas, the flow is sufficiently deep and there is sufficient relief that freeboard can be added to the flood surface and the extent then "stretched" to include adjacent land. Mainstream flooding consisted of North Creek and other major tributaries as follows:
    - North Creek downstream of Myles Avenue
    - Vermont Place Branch downstream of the Vermont Place basin
    - Whitehaven Drive detention basin and downstream area through the industrial area north of Hillsborough Road
    - King Street Branch downstream of King Street
    - The Lakelands Pond and channel downstream of Medcalf Street
    - Western Tributary downstream of Medcalf Street
    - Seaman Avenue Branch downstream of Seaman Avenue.

  In other areas, such as the Seaman Avenue Branch upstream of Seaman Avenue, they were not included as "mainstream" since adding freeboard to the 1% AEP peak flood level in the channel and stretching results in all areas south of the channel being encapsulated by this level. The 1% AEP flood results (with 0.9 m sea level rise) for the "mainstream" areas were filtered to remove shallow inundation, based on a hazard classification of H3 or higher (see Section 8.6) and depths exceeding 0.15 m. This filtering identifies the main creek/tributary and overland flow paths and reduces the issues associated with attempting to add freeboard and stretch in minor overland flow areas. The FPA in the mainstream area was defined as the 1% AEP peak flood level (with 0.9 m sea level rise) plus 0.5 m freeboard, with the level extended perpendicular to the flow direction either side of the flow path, to where this surface intersects with the ground level. This extent defines the "mainstream" FPA.

- Overland flooding: For overland flow areas, addition of freeboard and stretching generally produces an over-estimate of the land subject to flood risk, because the stretching extends across land in a way that would not actually occur even with significant additional flow from a much larger storm, and may even extend beyond the modelled PMF extent (as discussed for the Seaman Avenue Branch). It is therefore considered appropriate to use the 1% AEP design flood results (with 0.9 m sea level rise) without freeboard. This approach considers a true flood surface, accounting for factors such as flow momentum rather than an artificial surface generated by adding freeboard. In overland flow areas, it was considered appropriate to use filtered results to remove those areas that are affected by very shallow runoff, considered to be 'stormwater' rather than 'flooding'. The following filters were applied to the 1% AEP event with 0.9 m sea level rise:
    - **Depth Filter** – Exclude results below 0.15 m depth, and
    - **Small Pond Filter** – Remove isolated 'puddles' or 'orphans' smaller than 100 m^2.

The resulting extent was used to define the "overland" FPA.

The preliminary FPA (combined mainstream and overland) developed using the methodology above can be found in Figure C37. This figure also shows the 1% AEP peak flood level extent from Lake Macquarie flooding, in addition to considering this level with 0.4 m and 0.9 m sea level rise, in accordance with Reference 6 (see Table 2).

8.10. Advice on Land-Use Planning Considering Flooding

It is considered good practice to permit land use and development that is compatible with the nature of flooding in a particular area. For example, it is wise to limit use and development of land that is classified as floodway, since these are areas of conveyance and not only pose significant risks to occupiers, but any development in these areas can shift flood risks to other areas.

Lake Macquarie City Council implements flood-related planning controls via the LEP and DCP. The LEP specifies overarching objectives and principles to consider when granting development consent, while the DCP provides more detailed requirements and restrictions for different land uses. The LEP and DCP refer to mapping outputs that have been produced as part of this and other flood studies undertaken for Council.

This is a typical approach for consideration of flooding in land use planning, although WMAwater recommends that Council consider how 'overland' flooding sits within the framework and planning controls. Flooding in the North Creek catchment assessed in this study comprises both mainstream (relatively deep and fast flowing) and overland (generally shallow flow but can be fast flowing) flooding. This is slightly different to most of the previous flood studies undertaken by Council which have focused either on lake flooding (Lake Macquarie waterway) or major creek flooding (such as Dora Creek or Cockle Creek). Many Councils develop different or modified controls for these different flooding regimes. For example, with shallow overland flow depth inundation a 0.5 m freeboard may be considered too severe if such a level is well above the PMF level for the property. Overland flow is also more affected by local structures, such as fencing, than mainstream flooding where generally the majority of flow follows a well-defined path. Consideration might be given to having development controls that address these local issues with new development applications. These considerations will need to extend to other flood-prone areas of the LGA.

In urban areas with a short critical duration (say < 2 hours) the safe evacuation of residents either before or during a flood is not possible and can present a greater risk than remaining in the house or building. Flooding in the overland flow parts of the study area may occur rapidly with no effective and reliable warning. Thus, the flood is upon residents before they are aware of the problem (it could occur at night). All new buildings placed within flood extents should therefore be designed to be structurally sound (certified by a structural engineer) during a flood up to 0.5 m above the 1% AEP and with a floor at that level. Residents will therefore be able to shelter in place until the flood passes (typically < 1 hour). In a life threatening emergency residents should call the SES or police for rescue rather than attempt to drive or walk through floodwaters.

The Lake Macquarie LEP currently does not include the *Special Flood Considerations* clause

(5.22). Changes to the NSW Government planning framework in relation to flooding allows Council the opportunity to include a second clause within their LEPs which applies to land between the FPA and the PMF extent and considers sensitive and hazardous uses in addition to those uses which may have evacuation constraints. This inclusion empowers Council to apply controls that ensure the developers of such facilities appropriately consider and plan for the full range of flood risk at the site, so as to reduce potential property damages and minimise the risk to life in future flood events. Council should consider if this clause is appropriate for controlling development within the North Creek catchment and across the wider LGA.

A comprehensive review of Council's flood planning controls should be completed when the next Flood Risk Management Study and Plan is undertaken within the Lake Macquarie LGA.

9. SENSITIVITY ANALYSIS

9.1. Overview

A number of sensitivity analyses were undertaken to establish the variation in design flood levels and flows that may occur if different parameter assumptions were made. These sensitivity scenarios are summarised in Table 22.

Table 22: Overview of Sensitivity Analyses

Scenario	Description
Rainfall Losses	The initial and continuing rainfall losses were increased and decreased by 20%
Catchment Lag Factor, 'C'	The catchment lag factor value was increased and decreased by 20%
Mannings 'n'	The hydraulic roughness values were increased and decreased by 20%
Hydraulic Structure Blockage	Sensitivity to blockage of hydraulic structures including pits and culverts was assessed for both a low blockage and high blockage scenario.
Tailwater Level	The tailwater level in Lake Macquarie was increased and decreased by 0.25 m.
Climate Change – Rainfall Intensity	The 0.5% AEP and 0.2% AEP events were compared with the 1% AEP event as proxies for increases in rainfall intensities.
Climate Change – Sea Level Rise	Sea level rise for the year 2040 and 2090 were assessed assuming a corresponding increase in the Lake Macquarie level of 0.4 m and 0.9 m , respectively.

The sensitivity scenarios were simulated for the 5% AEP and 1% AEP events, except for climate change, which was assessed for the 1% AEP event only. The change in flood level across the study area for each scenario compared to the adopted design 5% AEP or 1% AEP flood events are provided in Appendix E.

9.2. Rainfall Losses

Rainfall losses were adopted from the ARR 2019 data hub (see Section 7.3). A sensitivity analysis was undertaken for both initial loss and continuing loss. Initial losses were taken from the data hub's probability neutral burst initial losses, which vary based on the AEP and duration of the storm (generally in the range of 5 mm to 15 mm). The continuing loss adopted was 0.88 mm/h to 1.08 mm/h, based on the factored data hub loss values. Rainfall losses were decreased and increased by 20% for the sensitivity analysis, with a summary presented in Table 23 for losses at the catchment centroid.

Table 23: Rainfall Loss Sensitivity Analysis

Scenario	Description	5% AEP Event		1% AEP Event	
		IL	CL	IL	CL
Design	Adopted design event losses	9.2	0.88	6.0	0.88
Losses + 20%	Sensitivity increasing losses	11.0	1.06	7.2	1.06
Losses - 20%	Sensitivity decreasing losses	7.4	0.70	4.8	0.70

The change in peak flood level for the increase and decrease in rainfall losses are provided in Figure E1 to Figure E4. The increase in rainfall losses has minimal impact throughout the catchment, with decreases in flood levels typically up to 0.02 m along North Creek. Similarly, the decrease in rainfall losses has minimal impact throughout the catchment, with increases in flood levels of up to 0.02 m along North Creek. The impact of losses is more pronounced in the 5% AEP event.

9.3. Catchment Lag Parameter

The catchment lag factor (termed 'C' in the WBNM model) can be used to accelerate or delay the runoff response to rainfall. The adopted C parameter of 2.4 was increased and decreased by 20% (2.88 and 1.92, respectively). The increase in the lag factor slightly decreases and delays peak catchment flows, while a decrease in the lag factor speeds up the response of the catchment and slightly increases peak flows.

The results of the catchment lag parameter sensitivity analysis are provided in Figure E5 to Figure E8. The results indicate that the 5% AEP and 1% AEP peak flood levels typically decrease by up to 0.05 m within the catchment (more prominent along North Creek) with an increase in the catchment lag. The converse applies with increasing the catchment lag factor, with typical increases of up to 0.05 m across the catchment.

9.4. Mannings Roughness Variations

The Mannings 'n' parameter in the TUFLOW model represents the surface roughness, and the adopted values are outlined in Table 10. A sensitivity analysis was conducted with both an increase and decrease in these values by 20%. The change in peak flood level for the increase and increase Mannings 'n' are provided in Figure E9 to Figure E12. Increasing the Mannings 'n' results in an increase in peak flood levels of up to approximately 0.05 m, most prominently along North Creek. Decreasing the Mannings 'n' results in a decrease in peak flood levels of a similar magnitude.

9.5. Blockage Variations

A sensitivity analysis was undertaken for the blockage of structures in the TUFLOW model. For the design events, blockage was applied in accordance with ARR 2019, as described in Section 7.6. A sensitivity analysis was undertaken by adopting a high and low blockage factor for the 20% AEP and 1% AEP events, as detailed in Table 24.

Table 24: Blockage Sensitivity Analysis

Structure	Design Blockage	Low Blockage	High Blockage
Small Culverts (Dia/Width < 1.2 m)	50%	20%	80%
Large Culverts (Dia/Width ≥ 1.2 m)	20%	0%	50%
Bridge	5%	0%	10%
Sag Pit	50%	20%	80%
On-grade Pit	20%	0%	50%

The results of ‘pipe’ blockage (including culverts and bridges) is provided in Figure E13 to Figure E16. The results indicate that blockage has only a minor impact on impacts on water levels. The low blockage scenario results in minor changes to flood levels, up to 0.02 m across the catchment. There are larger decreases up to 0.1 m within basins and localised decreases greater than 0.1 m within the Biddabah wetland, within the industrial area north of Hillsborough Road and North Creek upstream of Hillsborough Road. There is typically a decrease upstream of key structures and decrease downstream of them. The high blockage scenario results in the converse, with increases upstream of key structures and decreases downstream. Again, these changes in peak flood level are typically within 0.02 m, although larger increases are observed within basins (up to 0.2 m), with sensitive areas being the Biddabah wetlands, the industrial area upstream of Hillsborough Road, North Creek upstream of Hillsborough Road and the Vermont Place detention basin.

The results of ‘pit’ blockage (including sag and on-grade pits) is provided in Figure E17 to Figure E20. The results indicate that pit blockage generally has a negligible impact on peak flood levels. The most sensitive scenario was the high blockage scenario for the 5% AEP event, with peak flood levels increasing by up to 0.05 m in localised areas such as downstream of Lakelands Pond and on the King Street Branch upstream of Queen Street.

9.6. Tailwater Level Variations

For all the design flood events, a static tailwater level of 0.65 mAHD was adopted (see Section 7.5). This tailwater assumption affects the downstream reach of North Creek. A sensitivity analysis was undertaken by increasing the tailwater level by 0.25 m for the 5% AEP and 1% AEP events.

The results of the sensitivity analysis are provided in Figure E21 to Figure E24. Decreasing the tailwater level results in a reduction in peak flood level of approximately 0.02 m in areas adjacent to North Creek downstream of Martin Street in the 5% AEP event. In the 1% AEP event, it only affects the North Creek channel downstream of John Street. Increasing the tailwater level results in increases to peak flood levels of up to 0.03 m in areas adjacent to North Creek downstream of Albert Street in the 5% AEP event. In the 1% AEP event, only a small portion of the area adjacent to North Creek is affected, downstream area of Charles Street. The increase in peak flood level is up to 0.02 m.

9.7. Climate Change

Climate change is expected to increase sea levels and rainfall intensities. It is typical practice in catchment flood studies under the NSW flood program to model scenarios incorporating the effects of these impacts from climate change to understand the potential future changes in flood behaviour.

9.7.1. Sea Level Rise

Various projections of the likely increases to sea levels are available. Lake Macquarie Council engaged Manly Hydraulics Laboratory in 2012, 2015 and 2020 to analyse and report on Lake Macquarie Water Level Trends. The latest report concluded that the water levels at the Belmont gauge have risen by 2.74 mm/year over the last 33 years and 3.05 mm/year over the last 19 years.

The Lake Macquarie Waterway Flooding and Tidal Inundation Policy (2020) adopts sea level rise planning benchmarks established by the repealed *NSW Sea Level Rise Policy Statement* (2009). These benchmarks were a rise from 1990 levels of 0.4 m by 2050 and 0.9 m by 2100. These benchmarks have been adopted for some time as they are still reasonably close to the most recent reports and changing the planning levels too often would be unhelpful for owners, builders, developers and planners. Planning levels will be reviewed again when there is new scientific advice, or there is a change in government policy.

As a result of the information provided in the above and other documents, and to keep up-to-date with current best practice, this present study incorporates an assessment of climate change. However, it should be noted that climate change due to man-made or natural processes will still occur beyond the 2100 estimate.

The results of sea level rise, considering increases of 0.4 m and 0.9 m are contained in Figure E25 and Figure E26, respectively. The results indicate that for the 1% AEP event, a sea level rise of 0.4 m (2050 projection) will increase peak flood levels along North Creek, with levels in the overbank areas adjacent to the creek of approximately 0.02 m extending up to Albert Street. With sea level rise of 0.9 m (2100 projection), the 1% AEP peak flood levels will increase by up to 0.1 m in the overbank areas adjacent to North Creek, extending up to Walker Street.

9.7.2. Rainfall Intensity

Any increase in design flood rainfall intensities will increase the frequency, depth and extent of inundation across the catchment. The primary driver for this change is under a warmer climate, the atmosphere can hold more water, and hence more rainfall can occur in any given storm event. The design rainfall information currently provided by the BoM is based on historical climate data and does not currently include any allowance for likely increases to rainfall intensity in the future. ARR 2019 (Reference 7) provides some guidance about consideration of the impacts of climate change on design rainfall intensities. It suggests that rainfall intensities can be assumed to scale up by 5% per degree of average surface warming.

Projected increases to evaporation under a warmer climate are also an important consideration because increased evaporation would lead to generally drier catchment conditions, resulting in lower runoff from rainfall. Mean annual rainfall is projected to decrease, which will also result in generally dryer catchment conditions.

The current NSW State Government's advice recommends sensitivity analysis on flood modelling should be undertaken to develop an understanding of the effect of various levels of change in the hydrologic regime on the study area (Reference 17). To understand potential changes to flood behaviour due to increased intensity of rainfall, the 0.5% AEP and 0.2% AEP events were compared with the 1% AEP event, as suggested in the NSW Flood Risk Management Manual (Reference 17). These events provide an indication of how 1% AEP flood levels would change if the rainfall intensity increased to the point that it matches either the current 0.5% AEP (a 13% increase in intensity for the adopted critical duration) or 0.2% AEP (a 34% increase in intensity for the adopted critical duration). The change in peak flood levels, comparing the 0.5% AEP event and 0.2% AEP with the 1% AEP event can be seen in Figure E27 and Figure E28, respectively.

In comparison to the 1% AEP event, the 0.5% AEP flood levels are typically 0.02 m higher on overland flow paths and even on tributaries. Along North Creek, the increase in peak flood level is up to approximately 0.15 m. Similarly, the 0.2% AEP flood levels are higher than the 1% AEP levels by approximately 0.05 m in upstream overland flow areas and approximately 0.1 m to 0.2 m on the tributaries. On North Creek itself, peak flood levels are up to 0.4 m higher, with the most sensitive area being in the vicinity of King Street.

10. ECONOMIC IMPACTS OF FLOODING

10.1. Background

The impact of flooding can be quantified through the calculation of flood damages. Flood damage calculations do not include all impacts associated with flooding. They do, however, provide a basis for assessing the economic loss of flooding and also provide a non-subjective means of assessing the merit of flood mitigation works such as detention basins, levees, drainage enhancement etc. The quantification of flood damages is an important part of the floodplain risk management process. By quantifying flood damage for a range of design events, appropriate cost-effective management measures can be analysed in terms of their benefits (reduction in damages) versus the cost of implementation. The cost of damage and the degree of disruption to the community caused by flooding depends upon many factors including:

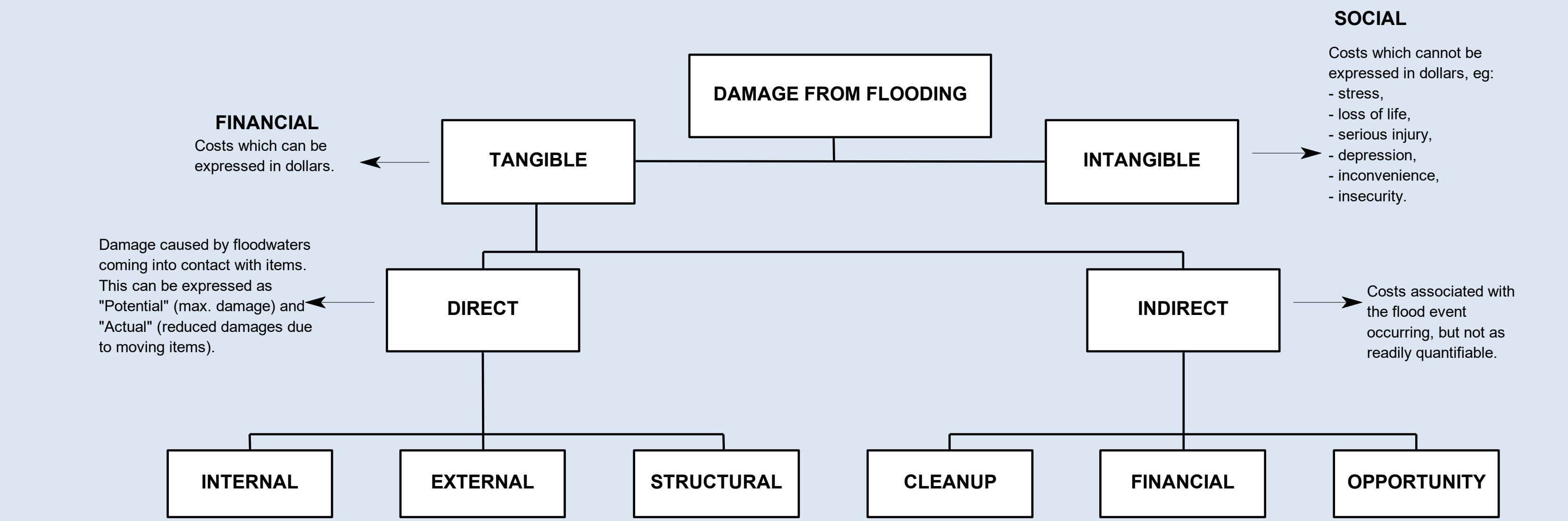
- The magnitude (depth, velocity and duration) of the flood,
- Land use and susceptibility to damages,
- Awareness of the community to flooding,
- Effective warning time,
- The availability of an evacuation plan or damage minimisation program,
- Physical factors such as failure of services (sewerage), flood borne debris, sedimentation, and
- The types of assets and infrastructure affected.

The estimation of flood damages tends to focus on the physical impact of damages on the human environment, but there is also a need to consider the ecological cost and benefits associated with flooding. Flood damages can be defined as being tangible or intangible. Tangible damages are those for which a monetary value can be easily assigned (for example damage to buildings, infrastructure, furnishings, goods or stock), while intangible damages are those to which a monetary value cannot easily be attributed (for example social costs such as increased levels of mental stress, loss of sentimental items, inconvenience to people, injury or loss of life). Types of flood damages are shown in Diagram 12.

The assessment of flood damages not only quantifies potential costs due to flooding but also identifies when properties are likely to become flood affected by either flooding on the property or by over floor flooding.

The total likely damages in any given flood event are difficult to quantify precisely, given the variable nature of flooding and the property and content values of houses affected. Design flood damages are estimated to obtain an indication of the magnitude of the flood problem and compare the economic effectiveness of proposed mitigation options. Understanding the total damages prevented over the life of a mitigation option in relation to current damages, or to an alternative option, can assist in the decision-making process.

Diagram 12: Flood Damages Categories (including damage and losses from permanent inundation)



	INTERNAL	EXTERNAL	STRUCTURAL	CLEANUP	FINANCIAL	OPPORTUNITY
RESIDENTIAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Laundries, Caravans, Sheds, Tools, Gardens, Fences	Physical Damage to Buildings: Gyprock, Cupboards, Scour of Footings, Houses becoming buoyant (floating off footings)	Clean Carpets, Walls, Clothes; Re-instate Furniture; Remove Mud and Debris	Loss of wages, Living costs (temporary accomodation and food), Time to repair/replace damaged items	Not Applicable
RURAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Sheds (stables/barns), Machinery, Tools, Fences, Feed storage, Saddles, Crops &/or Stock, Irrigation Systems	Physical Damage to Structures: Damage to Homestead, Sheds, Access tracks, Protection levees	Clean Homestead and Out-buildings; Remove Debris; Dispose of affected crops &/or stock	Loss of Farm Production and Income, Re-instatement of Pastures, Supplementary feeding of stock (by hand or outside agistment), Stock movement/ transport, Living costs (temporary accomodation and food)	Sowing or harvesting of Crops, Sale of Stock (at depreciated value or dependent on market influences)
COMMERCIAL	Contents of Buildings: Products, Stock, Fittings, Tools, Machinery, Raw Materials	External Items: Vehicles, Machinery, Display, Raw Materials/Stockpiles, Fences	Physical Damage to Buildings	Dispose of damaged products, stock, materials; Cleaning and Re-instatement	Loss of Productivity and Income, Bank Interest Charges	Loss of existing &/or Potential Trade
PUBLIC AUTHORITIES	Contents of Public Buildings and Facilities	Public Property and Facilities: Parks, Signs, Machinery, Equipment	Physical Damage to Infrastructure: Electricity, Water, Telephone, Gas, Road & Rail Transport Links	Remove Mud & Debris from Facilities, Public & Private Property Repairs (temporary & permanent)	Disruption of Services, Community Service Relief Grants	Provision of Public Service

10.2. Approach

Estimation of flood damage has focussed on residential and community buildings in the study area using guidelines issued by the NSW Government (Reference 17) and recognised damage assessment methodologies. The most common approach to present flood damage data is in the form of flood-damage curves for a range of property types, i.e. residential, commercial, public property, public utilities etc. These relate flood damage to depth of flooding above a threshold level (usually floor level). The estimation of damage is based upon a flood level relative to the floor level of a property. These damage curves are then factored by 6.26% (according to the consumer price index) to adjust the damages from its initial estimates (in 2022) to current day dollars. Additionally, these damages are varied for different regions in the state. The study area is located within the Eastern Land Division and no further regional cost adjustment factor is required south of Newcastle.

The assumed parameters and flood damage curve assumptions are outlined in the following sections.

10.2.1. Property Database

A property database was assembled using the available data, since it is not cost-effective to undertake detailed topographic survey of all or even a portion of flood prone properties across the study area. Floor levels of properties were estimated based on the following approach:

1. Obtained surveyed floor levels from the North Creek Flood Study (Reference 3), consisting of 170 properties. Surveyed floor levels were also obtained from the Lake Macquarie Waterway Floodplain Risk Management Study and Plan (Reference 6), consisting of 21 properties. These points were reviewed and GIS points placed appropriately for this study.
2. Determine properties affected by the 1% AEP flood extent (including those previously surveyed in point 1. above) for inclusion in the property database and estimate the height of the floor level above the ground level for these properties by undertaking a ‘windscreen survey’, utilising Google Street View where available. This involved looking at features such as number of steps into the building, number of bricks to the floor level or other visible features which can be used to provide an estimate of the difference between the floor level and adjacent ground level. For properties where it was difficult to estimate the floor height above ground due to obstructions, the lower level of confidence in the estimate was noted in the database.
3. Based on the above analysis, an indicative average floor level height above adjacent ground levels was determined. It was found that the average height above ground was 0.33 m.
4. Determine additional properties flood affected up to the PMF and add these to the property database.
5. Use GIS analysis to determine the ground level adjacent to each building within the property database using LiDAR data (see 2.2.1).
6. Estimate the floor level using, in order of preference:
    - Utilise the surveyed floor level, where it was considered to still be valid. This was assessed based on aerial imagery and the ‘windscreen survey’, where any properties that appeared to be developed within the last 10 years were assumed to have different floor levels to those surveyed.

- Estimated floor level from ground level and task 2 (typically those within the 1% AEP extent where floor levels were visible from Google Street View). This included surveyed properties that were considered to be redeveloped since the previous survey was undertaken.
- Estimated floor level from ground level and task 3 (typically those properties outside the 1% AEP extent but within the PMF extent).

The level of accuracy for the estimated floor heights is considered suitable for two reasons. Firstly, the estimation of property damage due to flooding is inherently difficult to estimate, given the large variation in building types, their contents, the duration of flooding and other factors, and so the accuracy of floor heights should be in line with the accuracy and applicability of the flood damage curves. Secondly, the economic damages assessment is only intended to be used as an estimate of the entire study area flood affectation and not on a per-property basis.

The property points can be seen in Figure 31. The total number of properties within the database was 1,843.

10.2.2. Residential Flood Damages

Tangible flood damages are comprised of two basic categories; direct and indirect damages (refer Diagram 12). Direct damages are caused by floodwaters wetting goods and possessions thereby damaging them and resulting in either costs to replace or repair or in a reduction to their value. Direct damages are further classified as either internal (damage to the contents of a building including carpets, furniture), structural (referring to the structural fabric of a building such as foundations, walls, floors, windows) or external (damage to all items outside the building such as cars, garages). Indirect damages are the additional financial losses caused by the flood for example the cost of temporary accommodation, loss of wages by employees etc.

Given the variability of flooding, property and content values, the total likely damages figure in any given flood event is useful to get a feel for the magnitude of the flood problem, however it is of little value for absolute economic evaluation. Flood damages estimates are also useful when studying the economic effectiveness of proposed mitigation options. Understanding the total damages prevented over the life of the option in relation to current damages, or to an alternative option, can assist in the decision-making process.

The standard way of expressing flood damages is in terms of average annual damages (AAD). AAD represents the equivalent average damages that would be experienced by the community on an annual basis, by taking into account the probability of a flood occurrence. This means the smaller floods, which occur more frequently, are given a greater weighting than the rare catastrophic floods.

In order to quantify the damages caused by inundation for existing development, the floor level database was used (see Section 10.2.1) in conjunction with modelled flood level information to calculate damages. The flood damages assessment was undertaken for existing development in accordance with current NSW Government guidelines (Reference 17). The damages were calculated using a number of height-damage curves which relate the depth of water above the floor with tangible damages. Each component of tangible damages is allocated a maximum value and a maximum depth at which this value occurs. Any flood depths greater than this allocated value do not incur additional damages as it is assumed that, by this level, all potential damages have already occurred.

10.2.2.1. Direct Internal Damages

Internal damages were assumed to follow the default damages of $550 per square metre (in 2022 dollars) adopted in the guideline (Reference 17) for residential properties. The actual damage to contents in an event can be reduced by actions taken during the warning time available in response to a flood threat. These actions may include raising goods and furniture, moving valuable items to the kitchen benchtop, onto tables, or up to the second storey, and taking some valuables as part of evacuation, if possible. The default value of 0.9 for the actual to potential damage ratio in the guideline (Reference 17) was adopted for this study area.

10.2.2.2. Direct Structural Damages

Structural damages were assumed to follow the default damages relationships to the dwelling size and number of storeys adopted in the guideline (Reference 17). Damage per m^2 is assumed to be $2,280 for single storey houses and $2,620 for double storey houses and $2,730 for units and $2,620 for townhouses. As the dwelling size has not been obtained, all houses were assumed to have the default size of 220 m^2 and units and townhouses were assumed to be 100 m^2 and 160 m^2, respectively. In floods larger than the 1% AEP event there is the possibility that some buildings may collapse or have to be demolished. The cost of these damages have not been included in the analysis.

10.2.2.3. Direct External Damages

The default external damages of $17,000 (in 2022 dollars) in the guideline (Reference 17) were adopted. This fixed external damage value was applied when the flood depth above ground level exceeded 300 mm or was above the habitable floor level.

10.2.2.4. Indirect Damages

Indirect damages were assumed to follow the default damage relationship in the guideline (Reference 17). That is, for residential clean-up costs of $4,500 (in 2022 dollars) and relocation costs of $609 per week (in 2022 dollars, median price for renting a 3 bedroom house in the area) will apply if over floor inundation exists. Non-residential indirect costs, which cover clean-up costs and loss of trading are 30% of the direct damages.

10.2.3. Non-residential Buildings

10.2.3.1. Commercial Properties and Public Buildings

Damage curves for commercial, industrial, and public buildings were adopted from the guideline (Reference 17). Direct damages (accounting for structural and contents damage) to these buildings are based on the value classification of the building as well as the floor area.

Commercial and industrial buildings are classified as low to medium, medium/default, and medium to high. The low to medium damage curves are factored by 0.6 of the default and medium to high damage curves are factored by 1.5. Commercial and industrial buildings used the medium/default damage curve as no further information on these buildings had been provided. As no information on floor area of each commercial and industrial building was provided, the default area of 418 m^2 was adopted. Actual to potential damage ratio was assumed to be 0.9.

Public buildings were classified as low/default and medium to high categories. The low/default damage curves for public buildings were assumed to be 40% of the medium/default commercial damage curve, whereas medium to high public buildings damage curve were assumed to be the same as the medium/default commercial damage curve.

10.2.4. Intangible Damages

Intangible damages were assumed to follow the default damage relationship in the guideline (Reference 17). These intangible damages cover social and wellbeing impacts of flooding to the community. These intangible damages have been incorporated in this assessment and were found to contribute only a small portion of the total flood damages (<5%).

10.3. Estimated Flood Damages

An estimation of the number of properties impacted (flooding occurring at the building), number of properties with above floor flooding and total damage costs for each modelled flood event was undertaken for each of the model areas. Properties estimated to be flooded above floor can be seen in Figure 31, with the event first flooded above floor indicated.

A typical measure used to estimate flood damages over a range of flood events is the Annual Average Damage (AAD). AAD represents the equivalent average damages that would be experienced by the community on an annual basis, by taking into account the probability of a flood occurrence over the long term. The AAD value is determined by multiplying the damages that can occur in a given flood by the probability of that flood actually occurring in a given year, and then summing across a range of floods. This method allows smaller floods, which occur more frequently to be given a greater weighting than the larger catastrophic floods that only occur rarely. The AAD for the existing case then provides a benchmark by which to assess the merit of flood management options.

A summary of the flood damages is provided in Table 25. Residential damages and the total damages (which include residential, commercial and public buildings, along with infrastructure damages) are provided separately. The total number of properties affected is also presented in these tables. The number of lots affected indicates that the flood level was higher than the ground level near the building on the property and the number of lots affected above floor indicates that the flood level was higher than the floor level.

Table 25: Summary of Estimated Flood Damages for the North Creek Catchment

	Flood Event	No. Lots Affected	No. Lots Flooded Above Floor Level	Total Damages for Event	Average Damage Per Flood Affected Property	% of AAD
Residential	50% AEP	28	0	$47,877	$1,710	1%
	20% AEP	125	1	$336,872	$2,695	7%
	10% AEP	234	8	$1,354,955	$5,790	10%
	5% AEP	329	30	$3,885,465	$11,810	15%
	2% AEP	446	67	$9,733,120	$21,823	23%
	1% AEP	539	89	$15,450,858	$28,666	14%
	0.5% AEP	618	122	$19,466,344	$31,499	10%
	0.2% AEP	700	150	$26,304,888	$37,578	8%
	PMF	1,198	450	$85,612,260	$71,463	13%
	Average Annual Damages			**$883,344**	**$737**	
Total	50% AEP	52	5	$242,778	$4,669	4%
	20% AEP	164	11	$849,144	$5,178	10%
	10% AEP	283	20	$2,736,091	$9,668	11%
	5% AEP	389	48	$6,520,565	$16,762	15%
	2% AEP	523	89	$14,736,580	$28,177	21%
	1% AEP	624	121	$22,759,315	$36,473	12%
	0.5% AEP	717	161	$29,624,034	$41,317	8%
	0.2% AEP	801	201	$40,798,719	$50,935	7%
	PMF	1,326	560	$148,063,336	$111,662	12%
	Average Annual Damages			**$1,606,102**	**$1,211**	

While there are no residential properties flooded above floor, there are 5 commercial/industrial properties flooded above floor in the 50% AEP event. Above floor inundation of residential properties commences in the 20% AEP event and steadily increases to 89 properties in the 1% AEP event, with residential damages exceeding $15M. There are over 500 residential properties estimated to be affected in the 1% AEP event. In the PMF event, there are over 1,300 properties affected, with 450 residential properties and 110 commercial/industrial properties flooded above floor. Residential flood damages reach $85M and total flood damages reach approximately $150M in the PMF event.

Average annual residential damages are approximately $880,000, with the total AAD reaching $1.6M. The AAD per flood affected property in the PMF is approximately $700 considering residential properties and $1,200 considering all flood affected properties in the PMF. It is the 2% AEP event that contributes the most to the AAD. This indicates that flood mitigation measures should target events of this magnitude.

The estimation of flood damages is a high-level exercise, intended to capture flood damages at the catchment scale, providing a good indication of average damages across a catchment. The accuracy of the results (flood depths) at individual properties can be affected by vagaries such as the variability in the flood level across the property, the location of the sampled flood level for the property, whether the floor level varies through the building, etc. The estimation of damages (flood damage curves) is subject to similar accuracy limitations at the property level. These variabilities tend to average out across the catchment, particularly if many properties are considered.

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ATTACHMENT A: ARR 2019 Datahub Metadata

Attachment A

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	151.653
Latitude	-32.967
Selected Regions	
River Region	
ARF Parameters	
Storm Losses	
Temporal Patterns	
Areal Temporal Patterns	
Interim Climate Change Factors	

Region Information

Data Category	Region
River Region	Macquarie-Tuggerah Lakes
ARF Parameters	SE Coast
Temporal Patterns	East Coast South

Data

River Region

division	South East Coast (NSW)
rivregnum	11
River Region	Macquarie-Tuggerah Lakes

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2016_v1

ARF Parameters

Long Duration ARF

$$Areal\ reduction\ factor = Min\left\{1, \left[1 - a\left(Area^{b} - c\log_{10}Duration\right)Duration^{-d} + eArea^{f}\,Duration^{g}\left(0.3 + \log_{10}AEP\right) + h10^{iArea\frac{Duration}{1440}}\left(0.3 + \log_{10}AEP\right)\right]\right\}$$

Zone	SE Coast
a	0.06
b	0.361
c	0.0
d	0.317
e	8.11e-05
f	0.651
g	0.0
h	0.0
i	0.0

Short Duration ARF

$$ARF = Min\left[1, 1 - 0.287\left(Area^{0.265} - 0.439\log_{10}(Duration)\right).Duration^{-0.36} + 2.26 \text{ x } 10^{-3} \text{ x } Area^{0.226}.Duration^{0.125}\left(0.3 + \log_{10}(AEP)\right) + 0.0141 \text{ x } Area^{0.213} \text{ x } 10^{-0.021\frac{(Duration-180)^2}{1440}}\left(0.3 + \log_{10}(AEP)\right)\right]$$

Layer Info

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Storm Losses

Note: Burst Loss = Storm Loss - Preburst

Note: These losses are only for rural use and are NOT FOR USE in urban areas

id	21396.0
Storm Initial Losses (mm)	21.0
Storm Continuing Losses (mm/h)	2.2

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2016_v1

Temporal Patterns

code	ECsouth
Label	East Coast South

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2016_v2

Areal Temporal Patterns

code	ECsouth
arealabel	East Coast South

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2016_v2

BOM IFD Depths

Click here to obtain the IFD depths for catchment centroid from the BoM website

No data	No data found at this location!

Layer Info

Time Accessed	26 August 2024 08:25PM

Median Preburst Depths and Ratios

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	9.0 (0.294)	8.4 (0.191)	8.0 (0.149)	7.7 (0.119)	3.3 (0.042)	0.0 (0.000)
90 (1.5)	8.0 (0.226)	6.2 (0.121)	5.0 (0.080)	3.8 (0.052)	2.5 (0.027)	1.4 (0.014)
120 (2.0)	7.3 (0.187)	7.2 (0.128)	7.1 (0.103)	7.0 (0.086)	4.5 (0.045)	2.6 (0.023)
180 (3.0)	5.4 (0.120)	6.1 (0.094)	6.6 (0.083)	7.0 (0.075)	7.6 (0.066)	8.0 (0.060)
360 (6.0)	10.5 (0.182)	23.8 (0.285)	32.5 (0.318)	41.0 (0.335)	30.3 (0.201)	22.2 (0.128)
720 (12.0)	5.9 (0.079)	12.1 (0.110)	16.2 (0.120)	20.1 (0.123)	27.7 (0.138)	33.5 (0.143)
1080 (18.0)	3.7 (0.042)	10.7 (0.083)	15.3 (0.096)	19.7 (0.102)	28.7 (0.120)	35.5 (0.127)
1440 (24.0)	0.5 (0.005)	5.3 (0.037)	8.5 (0.047)	11.5 (0.053)	20.9 (0.077)	27.9 (0.088)
2160 (36.0)	0.0 (0.000)	3.5 (0.021)	5.9 (0.028)	8.1 (0.032)	9.5 (0.030)	10.5 (0.028)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	1.3 (0.004)	2.2 (0.006)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

10% Preburst Depths

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
90 (1.5)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

25% Preburst Depths

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.4 (0.012)	0.2 (0.005)	0.1 (0.002)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
90 (1.5)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.9 (0.011)	1.6 (0.015)	2.1 (0.018)	0.9 (0.006)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	2.6 (0.011)	4.6 (0.016)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.2 (0.001)	0.3 (0.001)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

75% Preburst Depths

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	35.6 (1.162)	35.9 (0.813)	36.1 (0.667)	36.2 (0.563)	26.2 (0.332)	18.7 (0.206)
90 (1.5)	37.0 (1.050)	34.3 (0.675)	32.5 (0.523)	30.8 (0.416)	35.3 (0.390)	38.6 (0.372)
120 (2.0)	43.6 (1.121)	46.2 (0.824)	48.0 (0.699)	49.6 (0.609)	46.6 (0.467)	44.3 (0.386)
180 (3.0)	39.1 (0.872)	42.4 (0.656)	44.6 (0.564)	46.7 (0.496)	66.9 (0.581)	82.1 (0.620)
360 (6.0)	39.9 (0.691)	61.9 (0.743)	76.6 (0.748)	90.6 (0.742)	108.4 (0.720)	121.7 (0.700)
720 (12.0)	29.7 (0.393)	41.6 (0.381)	49.6 (0.367)	57.1 (0.351)	77.9 (0.386)	93.5 (0.398)
1080 (18.0)	24.8 (0.280)	40.1 (0.312)	50.3 (0.315)	60.0 (0.311)	75.0 (0.312)	86.2 (0.308)
1440 (24.0)	17.8 (0.181)	32.5 (0.226)	42.2 (0.235)	51.5 (0.236)	64.5 (0.238)	74.2 (0.235)
2160 (36.0)	11.5 (0.100)	25.2 (0.151)	34.3 (0.164)	43.1 (0.169)	51.4 (0.162)	57.6 (0.156)
2880 (48.0)	9.1 (0.072)	11.3 (0.061)	12.7 (0.055)	14.0 (0.050)	23.2 (0.066)	30.2 (0.074)
4320 (72.0)	0.0 (0.000)	4.4 (0.021)	7.4 (0.028)	10.2 (0.032)	19.5 (0.050)	26.5 (0.059)

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

90% Preburst Depths

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	87.6 (2.858)	114.7 (2.598)	132.6 (2.453)	149.8 (2.328)	128.5 (1.628)	112.4 (1.239)
90 (1.5)	89.6 (2.540)	88.0 (1.732)	87.0 (1.399)	86.0 (1.163)	117.9 (1.303)	141.8 (1.364)
120 (2.0)	84.1 (2.162)	101.0 (1.801)	112.2 (1.635)	123.0 (1.508)	127.8 (1.280)	131.4 (1.146)
180 (3.0)	68.3 (1.526)	97.1 (1.503)	116.1 (1.469)	134.4 (1.429)	171.6 (1.489)	199.4 (1.506)
360 (6.0)	78.8 (1.366)	109.5 (1.314)	129.9 (1.269)	149.4 (1.223)	178.1 (1.183)	199.6 (1.148)
720 (12.0)	58.5 (0.776)	83.8 (0.766)	100.4 (0.743)	116.5 (0.716)	156.8 (0.777)	186.9 (0.797)
1080 (18.0)	49.8 (0.563)	78.1 (0.607)	96.8 (0.606)	114.7 (0.594)	142.6 (0.594)	163.5 (0.584)
1440 (24.0)	56.2 (0.569)	77.2 (0.536)	91.0 (0.508)	104.3 (0.479)	125.8 (0.464)	142.0 (0.449)
2160 (36.0)	49.5 (0.432)	69.9 (0.418)	83.3 (0.399)	96.2 (0.377)	106.5 (0.336)	114.1 (0.309)
2880 (48.0)	20.3 (0.161)	43.8 (0.237)	59.3 (0.257)	74.2 (0.262)	85.7 (0.245)	94.3 (0.233)
4320 (72.0)	6.6 (0.046)	19.2 (0.092)	27.6 (0.106)	35.6 (0.112)	62.7 (0.160)	83.1 (0.184)

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

Interim Climate Change Factors

Values are of the format temperature increase in degrees Celcius (% increase in rainfall)

	RCP 4.5	RCP6	**RCP 8.5**
2030	**0.892 (4.5%)**	0.775 (3.9%)	**0.979 (4.9%)**
2040	**1.121 (5.6%)**	1.002 (5.0%)	**1.351 (6.8%)**
2050	**1.334 (6.7%)**	1.28 (6.4%)	**1.765 (8.8%)**
2060	**1.522 (7.6%)**	1.527 (7.6%)	**2.23 (11.2%)**
2070	**1.659 (8.3%)**	1.745 (8.7%)	**2.741 (13.7%)**
2080	**1.78 (8.9%)**	1.999 (10.0%)	**3.249 (16.2%)**
2090	**1.825 (9.1%)**	2.271 (11.4%)	**3.727 (18.6%)**

Layer Info

Time Accessed	26 August 2024 08:25PM
Version	2016_v1
Note	ARR recommends the use of RCP4.5 and RCP 8.5 values

FIGURES

Figures

FIGURE 1
NORTH CREEK WARNERS BAY FLOOD STUDY
NORTH CREEK CATCHMENT STUDY AREA

FIGURE 2
NORTH CREEK WARNERS BAY FLOOD STUDY
DEVELOPMENT AREAS SINCE 2007
Seaman Avenue Branch
Western Tributary
North Creek
Lakelands Branch
King Street Branch
Vermont Place Branch
Areas of development
North Creek Catchment
Creeks
Cadastre
0
0.125
0.25
0.5
0.75
1
Kilometres
N
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\Figures\Fig02_Development.mxd

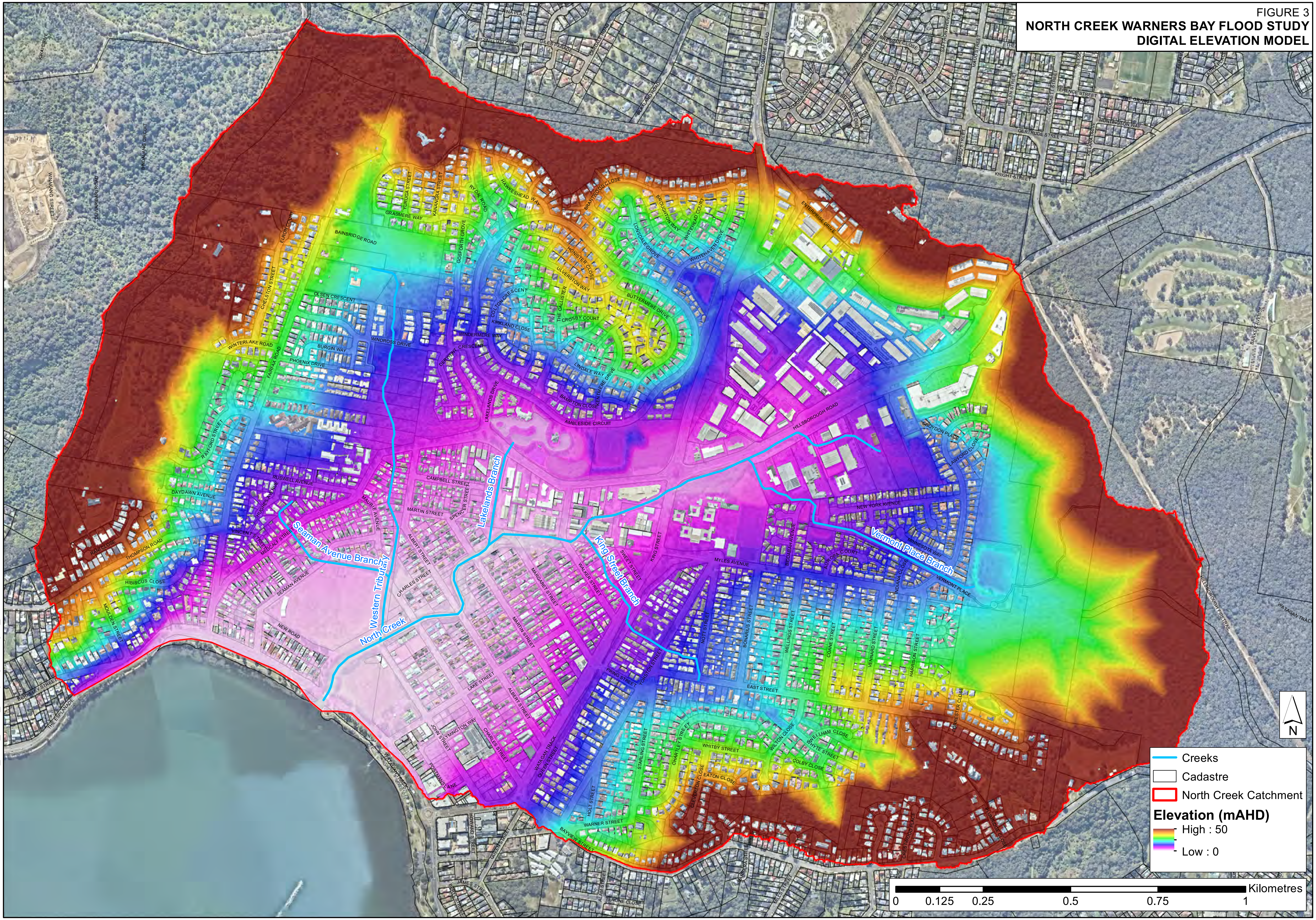

FIGURE 3
NORTH CREEK WARNERS BAY FLOOD STUDY
DIGITAL ELEVATION MODEL

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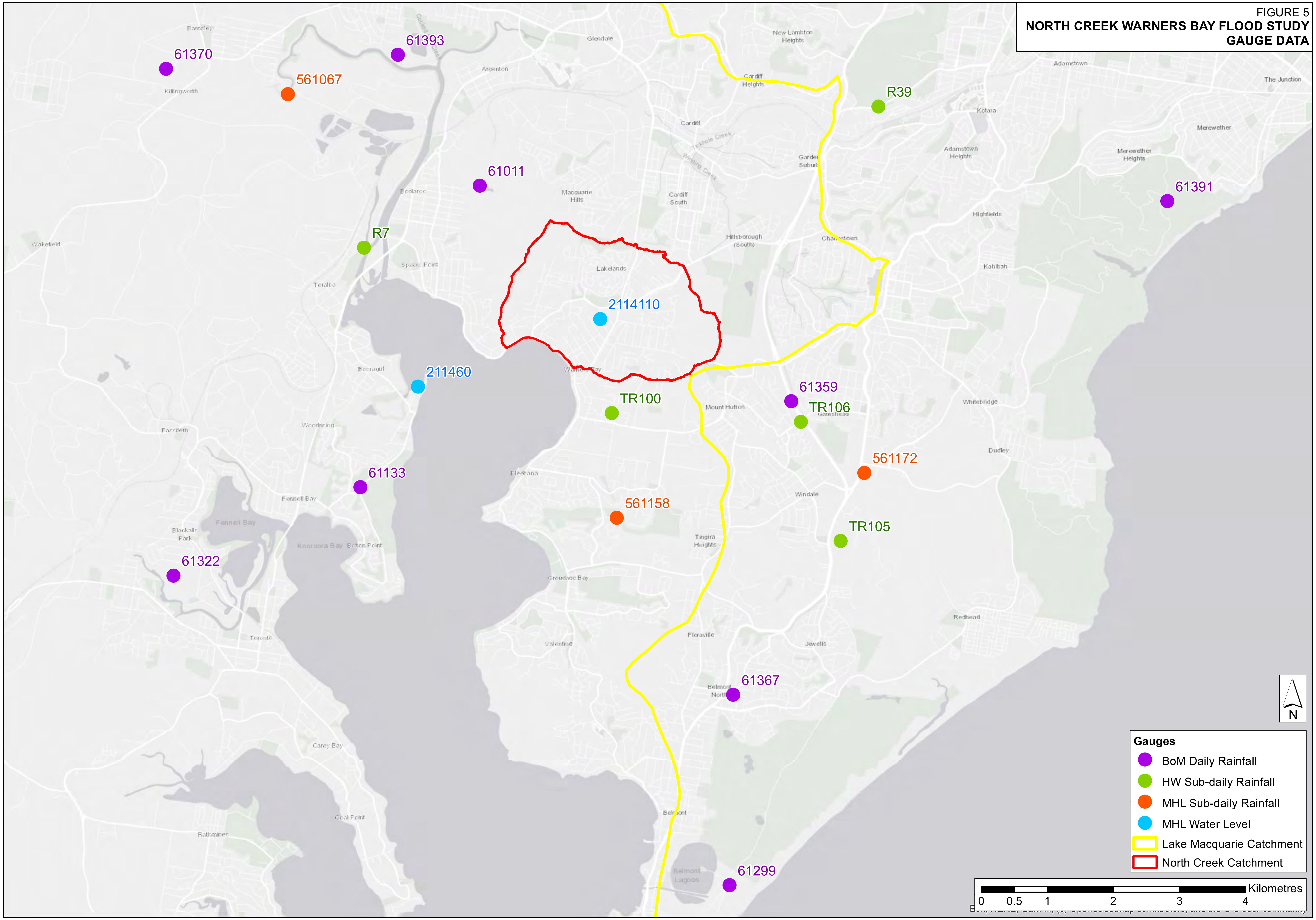

FIGURE 5
NORTH CREEK WARNERS BAY FLOOD STUDY
GAUGE DATA
61370
61393
561067
61011
R7
R39
61391
2114110
211460
61359
TR100
TR106
561172
61133
561158
TR105
61322
61367
61299
Gauges
BoM Daily Rainfall
HW Sub-daily Rainfall
MHL Sub-daily Rainfall
MHL Water Level
Lake Macquarie Catchment
North Creek Catchment
0 0.5 1 2 3 4 Kilometres
N
J:\Jobs\123045\GIS\ArcGIS\240510_Stage2_Report\Fig05_Gauges.mxd

FIGURE 6
NORTH CREEK WARNERS BAY FLOOD STUDY
COMMUNITY QUESTIONNAIRE RESPONSES

FIGURE 7
NORTH CREEK WARNERS BAY FLOOD STUDY
WBNM SUB-CATCHMENTS

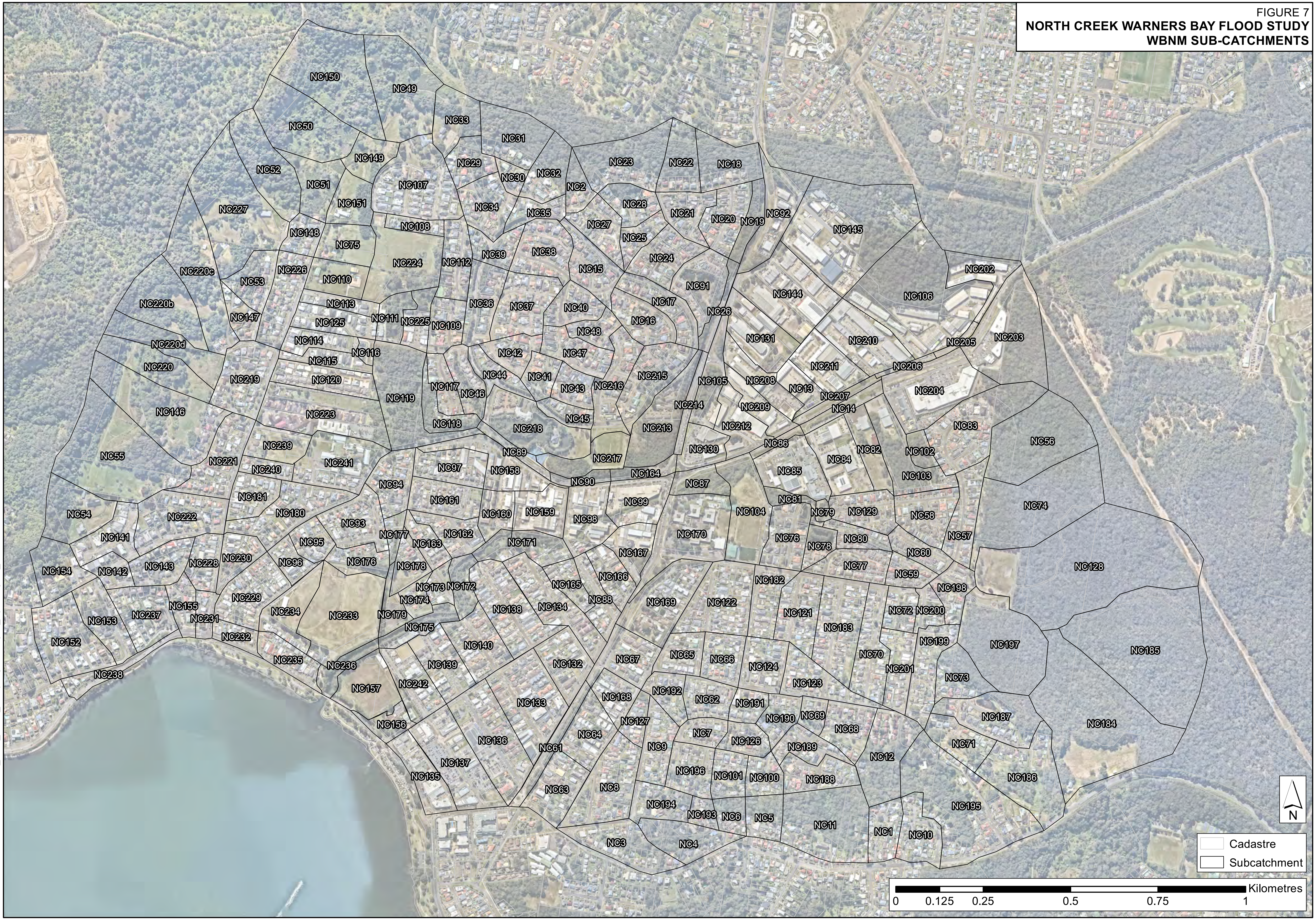

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FIGURE 8
NORTH CREEK WARNERS BAY FLOOD STUDY
TUFLOW MODEL LAYOUT

J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\Figures\Fig08_Hydraulic_model.mxd

FIGURE 9
NORTH CREEK WARNERS BAY FLOOD STUDY
TUFLOW STORMWATER NETWORK

J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\Figures\Fig09_Stormwater_network.mxd

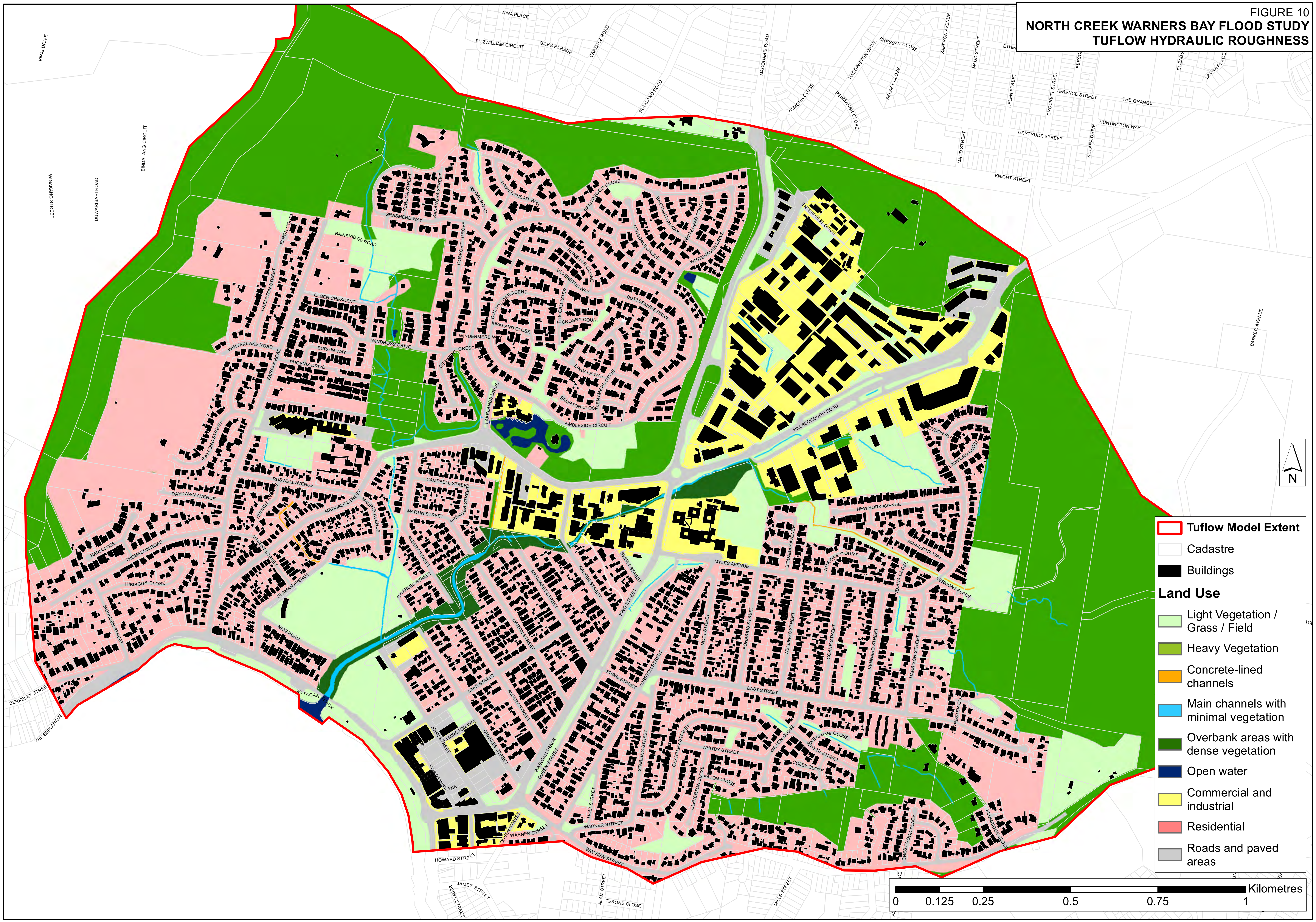

FIGURE 10
NORTH CREEK WARNERS BAY FLOOD STUDY
TUFLOW HYDRAULIC ROUGHNESS

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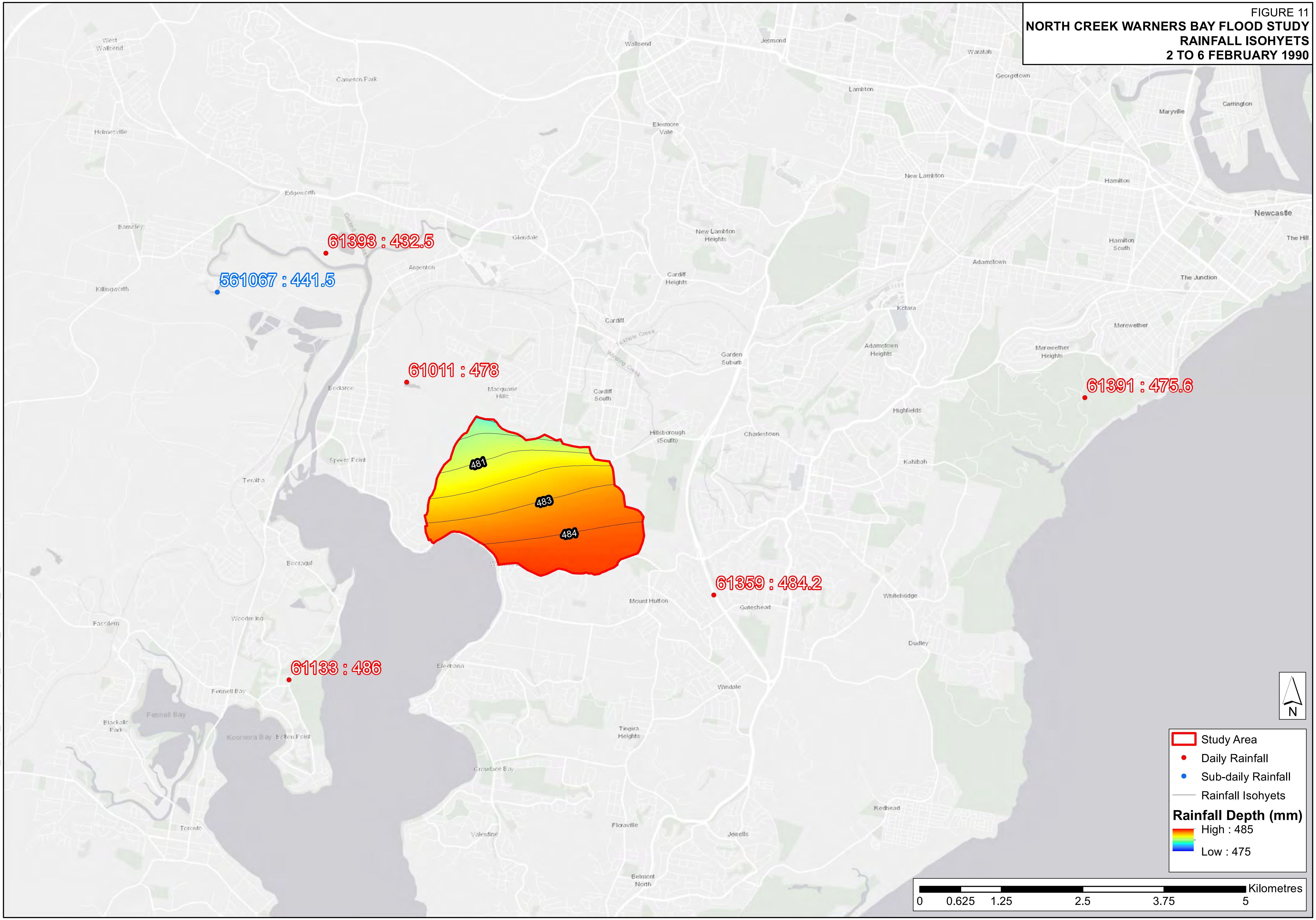

FIGURE 11
NORTH CREEK WARNERS BAY FLOOD STUDY
RAINFALL ISOHYETS
2 TO 6 FEBRUARY 1990
61393 : 432.5
561067 : 441.5
61011 : 478
61391 : 475.6
61359 : 484.2
61133 : 486
481
483
484
N
Study Area
Daily Rainfall
Sub-daily Rainfall
Rainfall Isohyets
Rainfall Depth (mm)
High : 485
Low : 475
Kilometres
0
0.625
1.25
2.5
3.75
5
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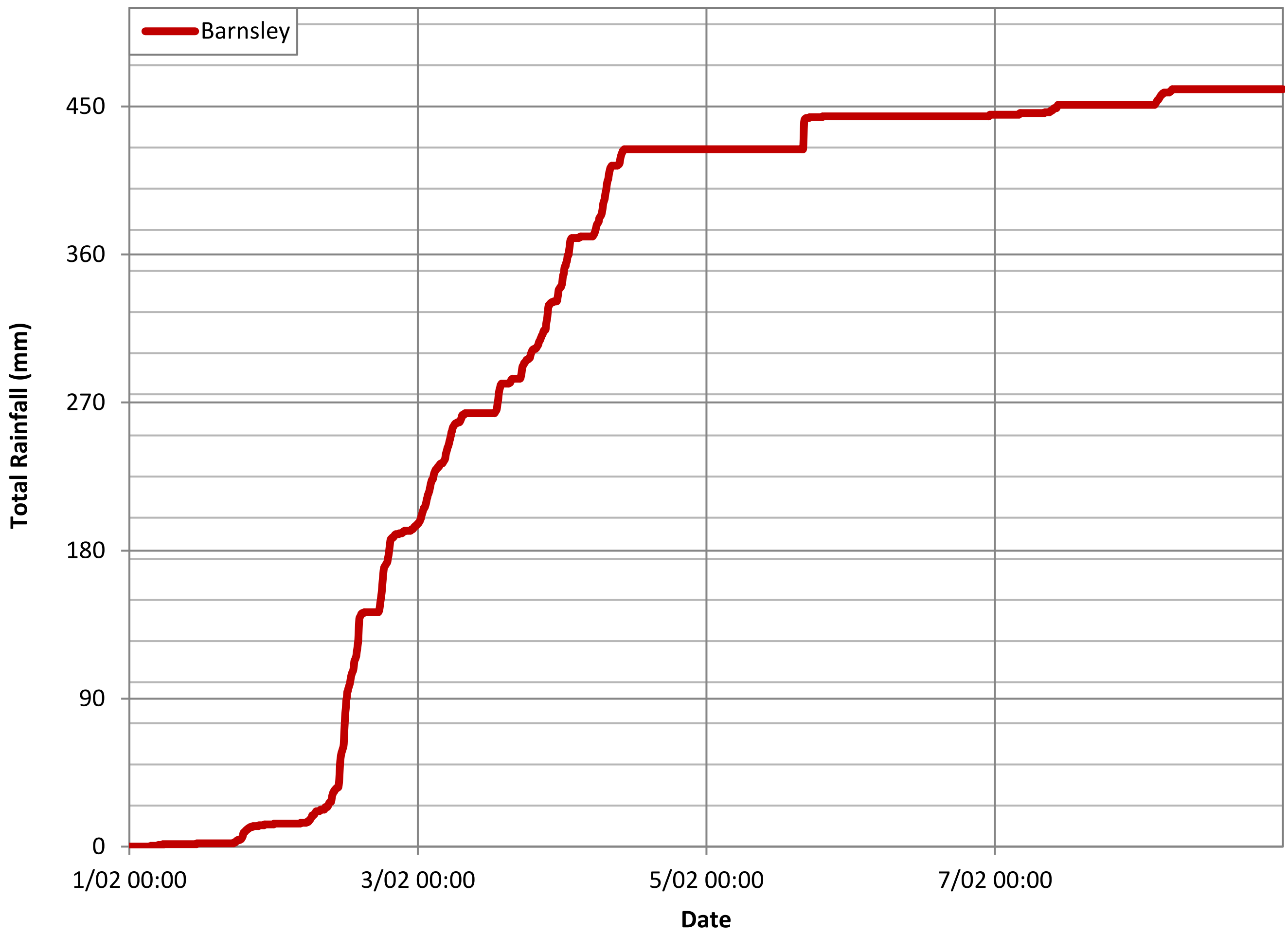
CUMULATIVE RAINFALL DATA
February 1990 EVENT



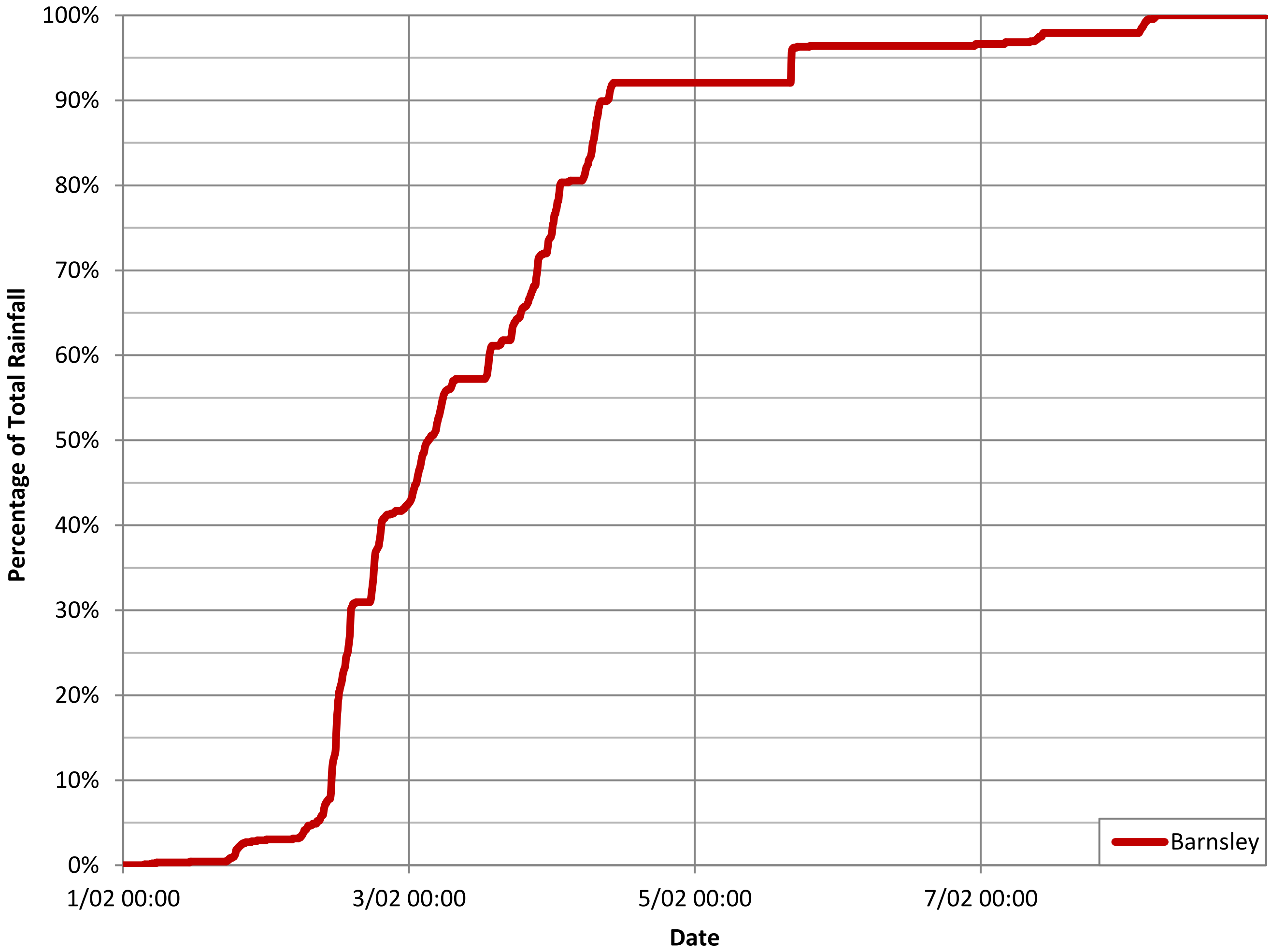


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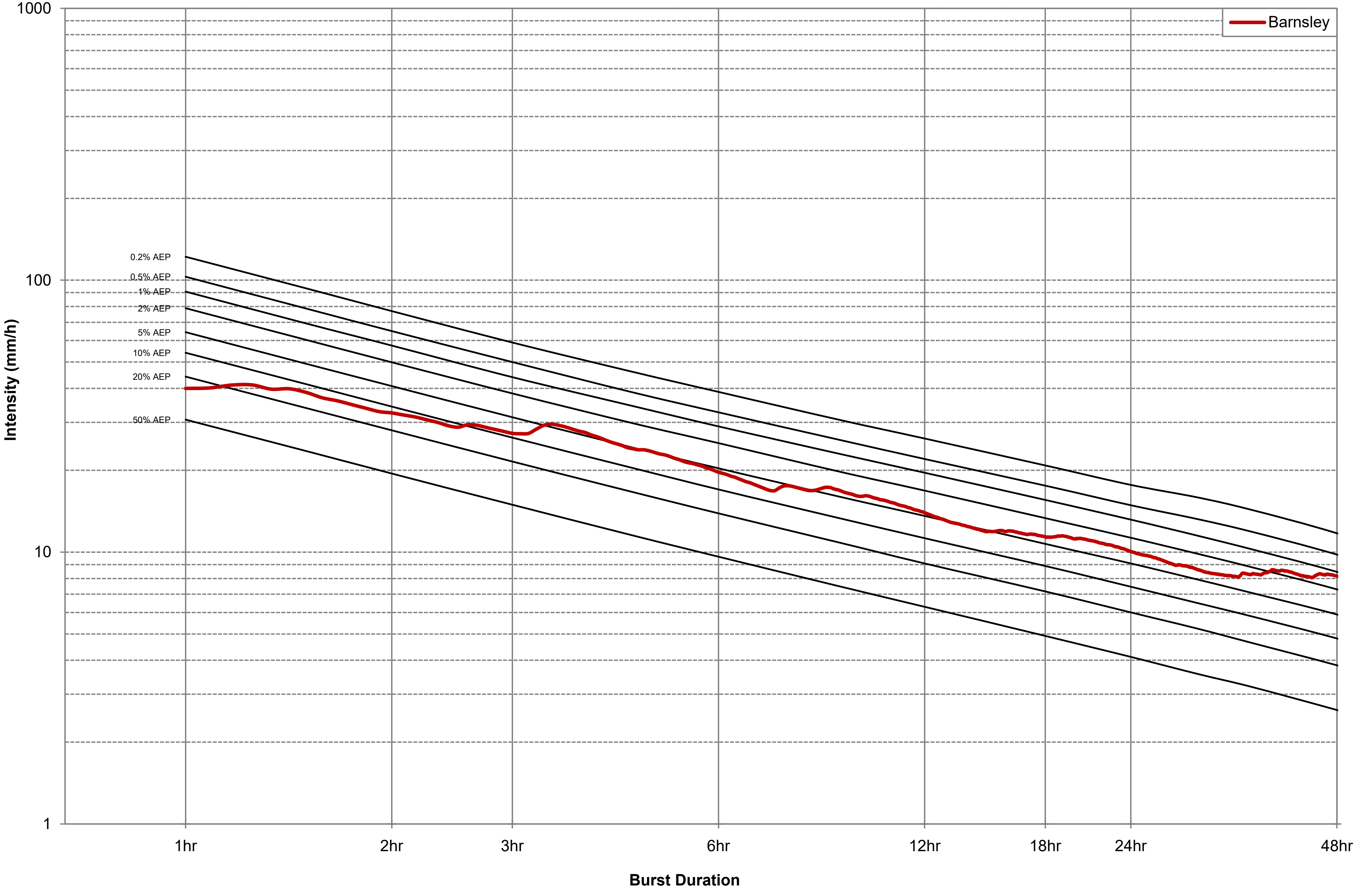
FIGURE 13
BURST INTENSITIES AND FREQUENCIES
February 1990 EVENT



J:\Jobs\123045\Hydrology\Calibration\pluviometer\123045_Historical_Burst_IFD_Comparison_v1.4.xlsm

FIGURE 14
NORTH CREEK WARNERS BAY FLOOD STUDY
RAINFALL ISOHYETS
9 JUNE 2007

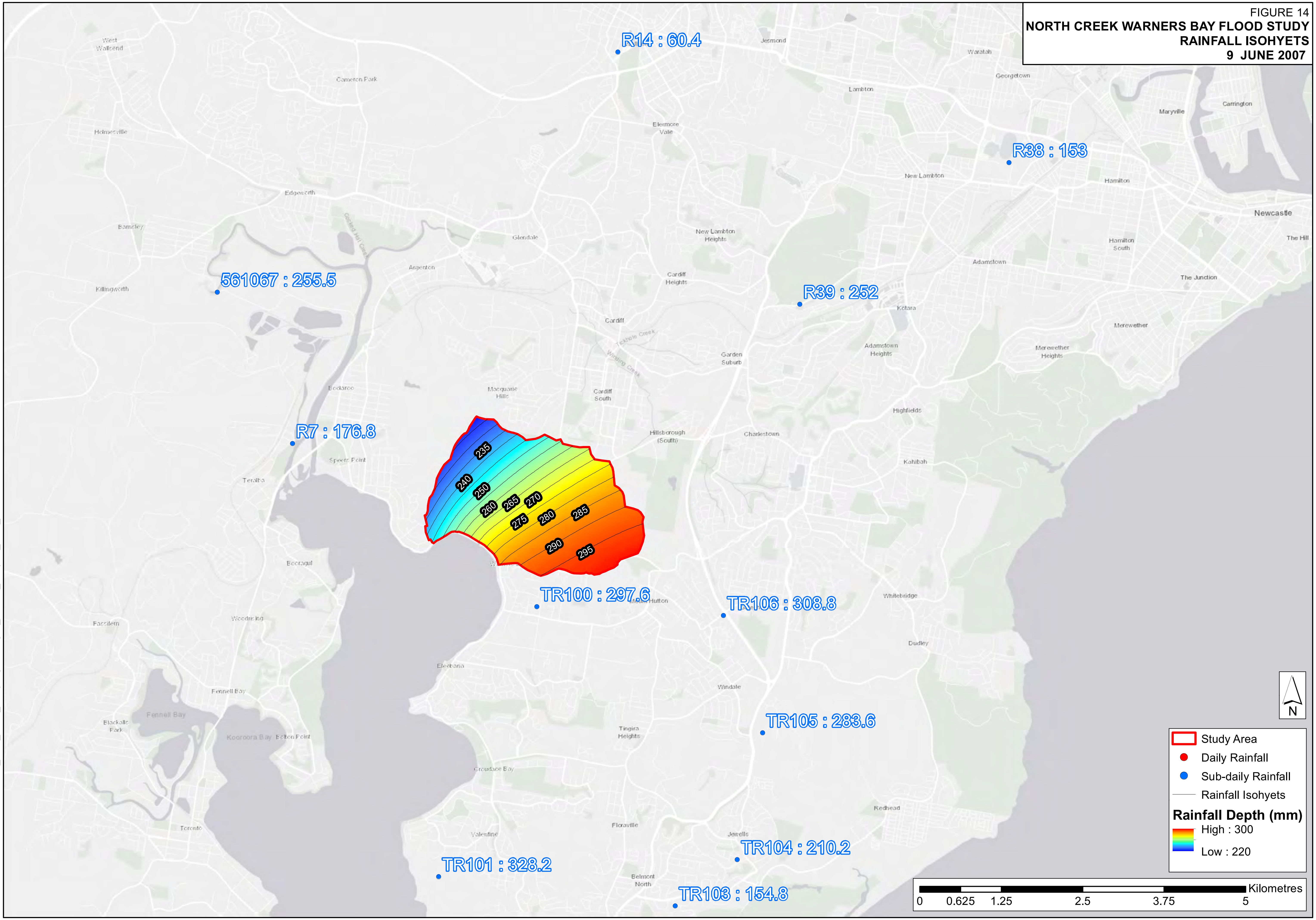


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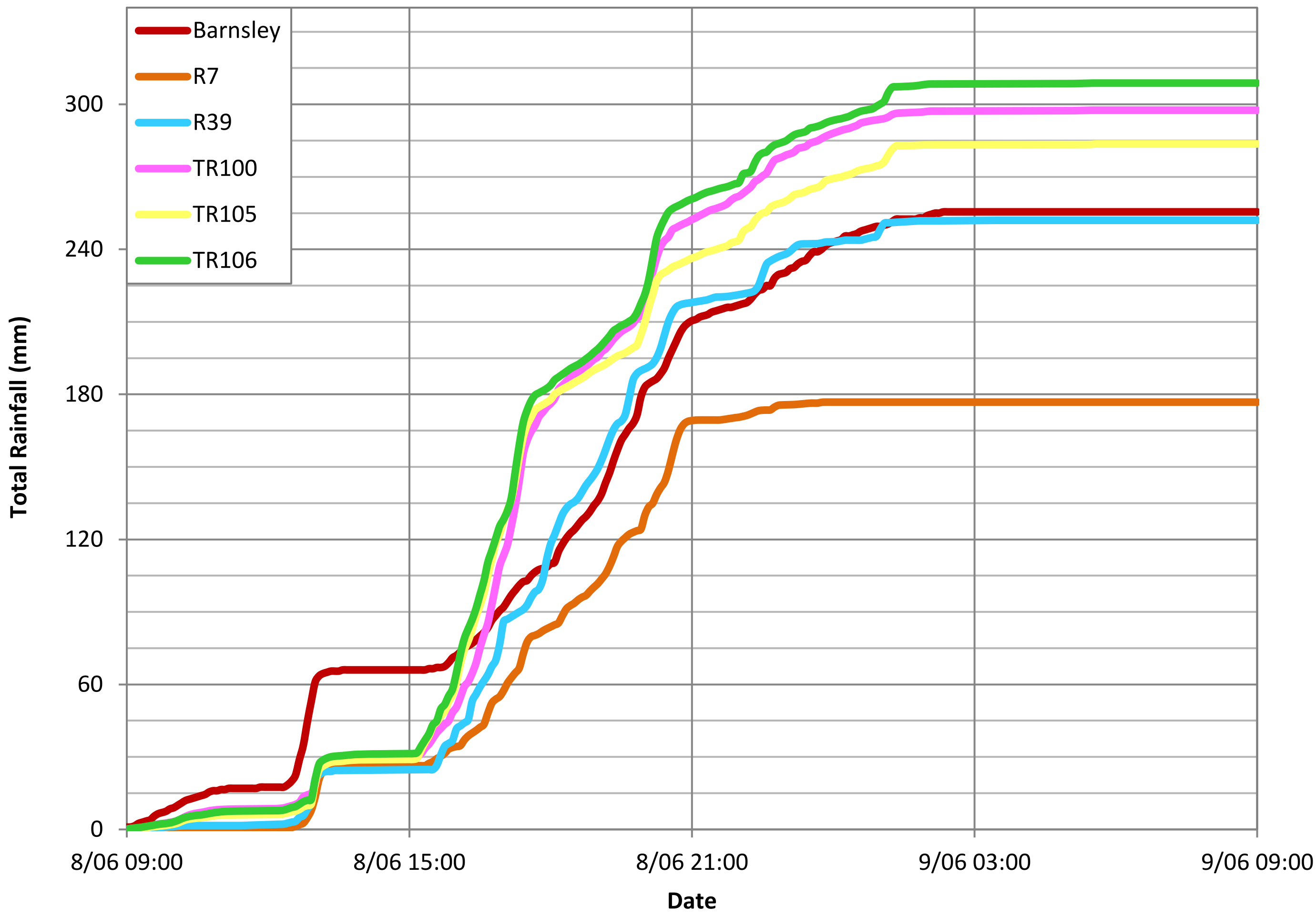
FIGURE 15
CUMULATIVE RAINFALL DATA
June 2007 EVENT



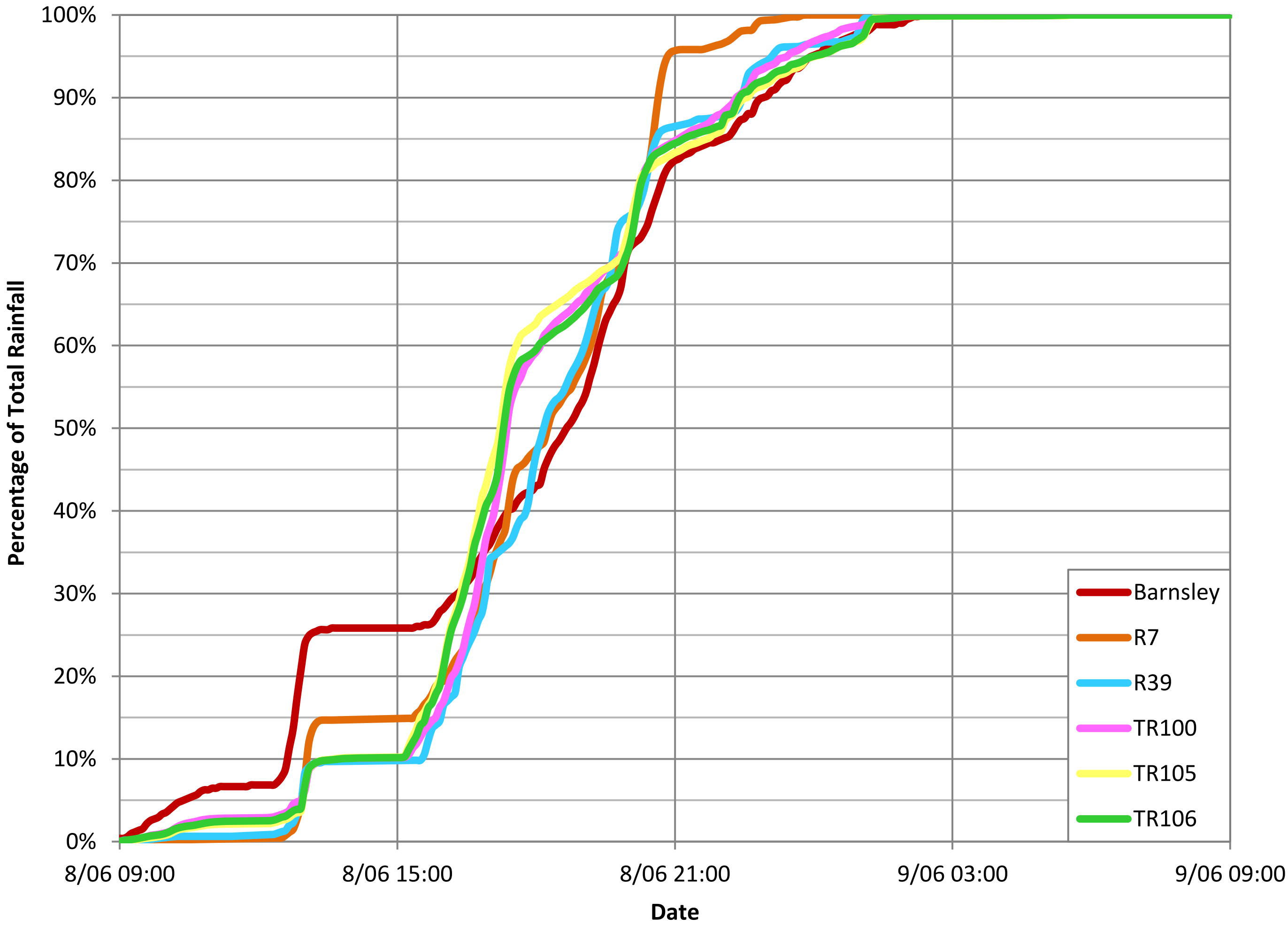


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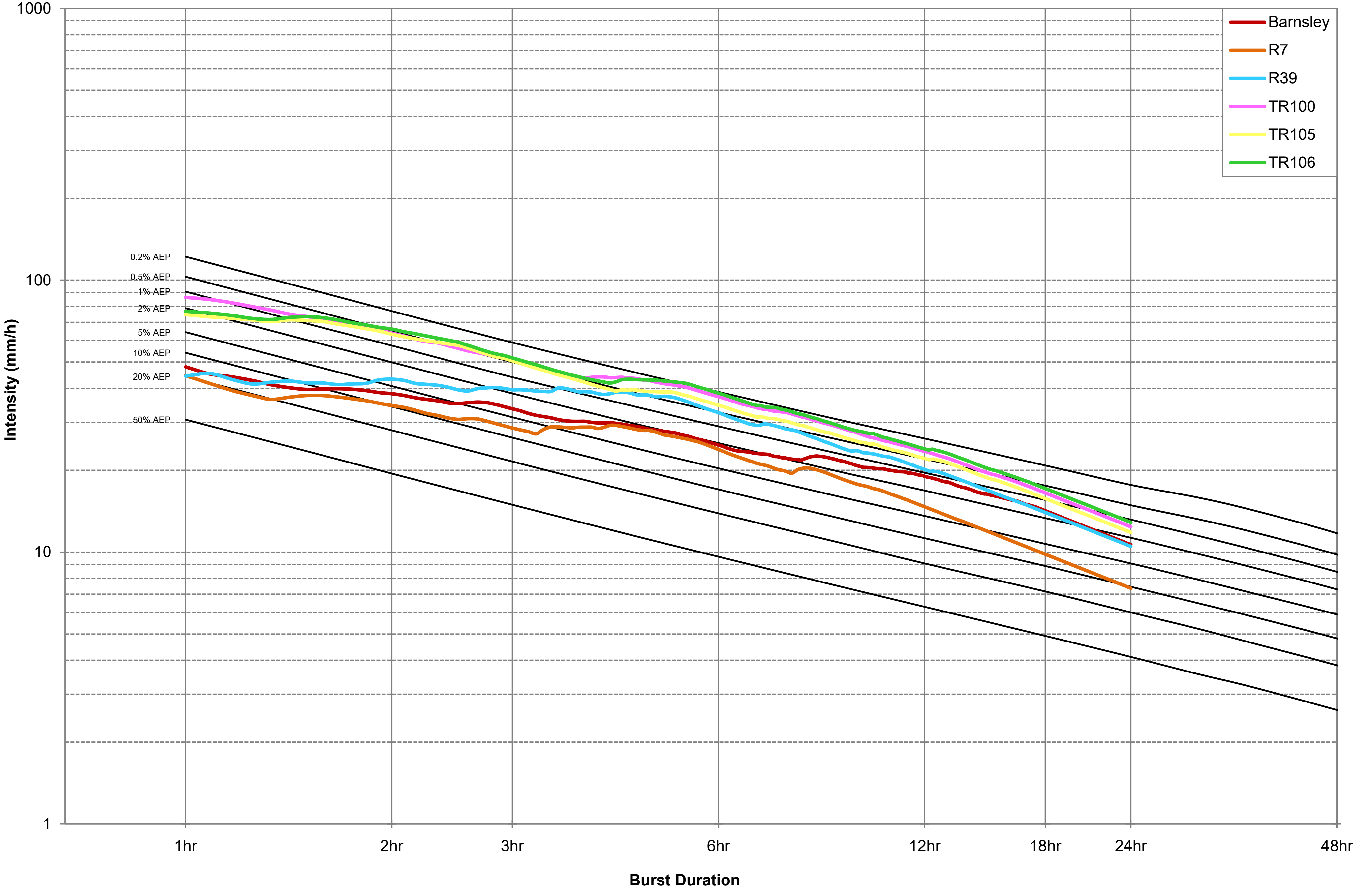
FIGURE 16
BURST INTENSITIES AND FREQUENCIES
June 2007 EVENT



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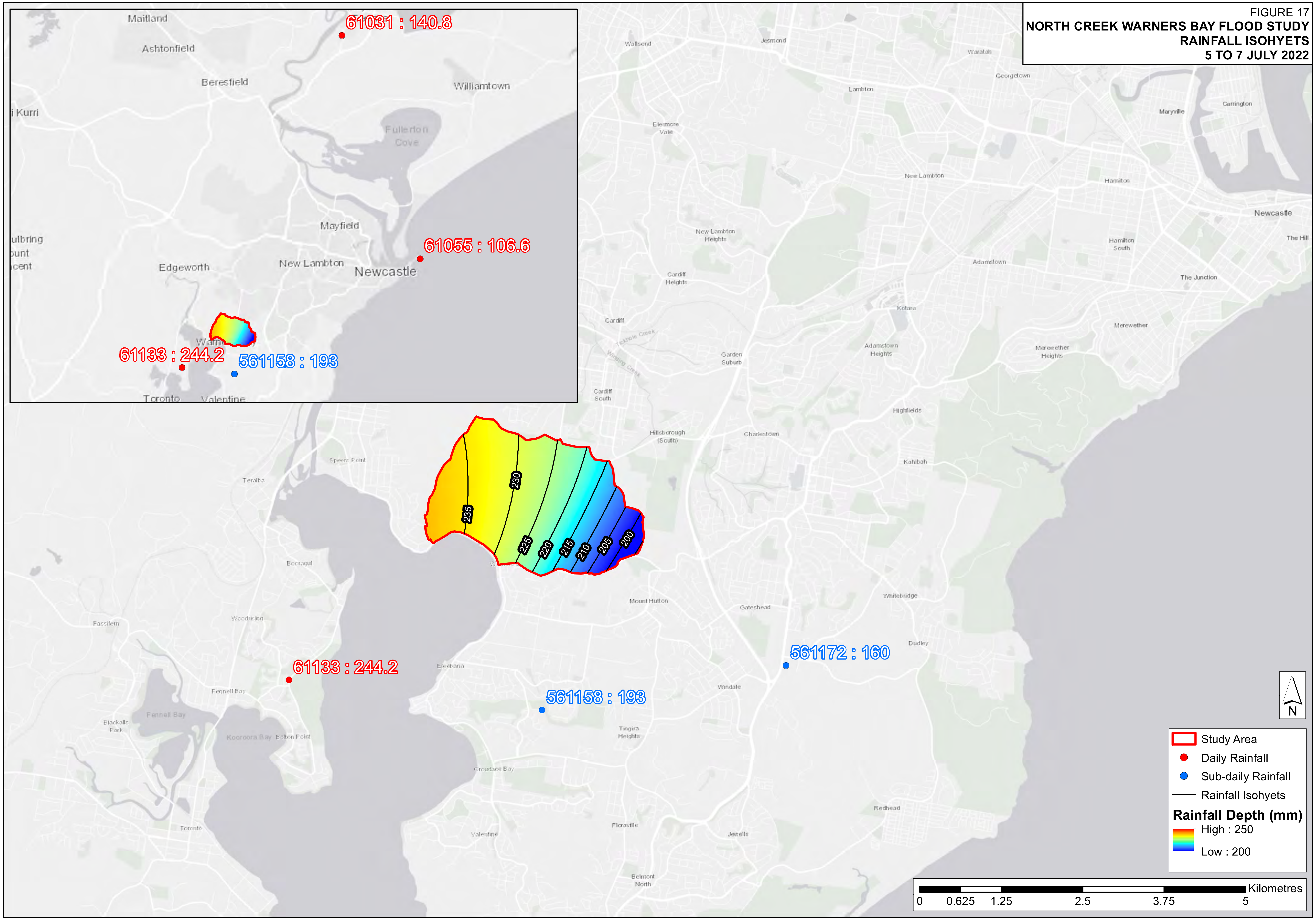
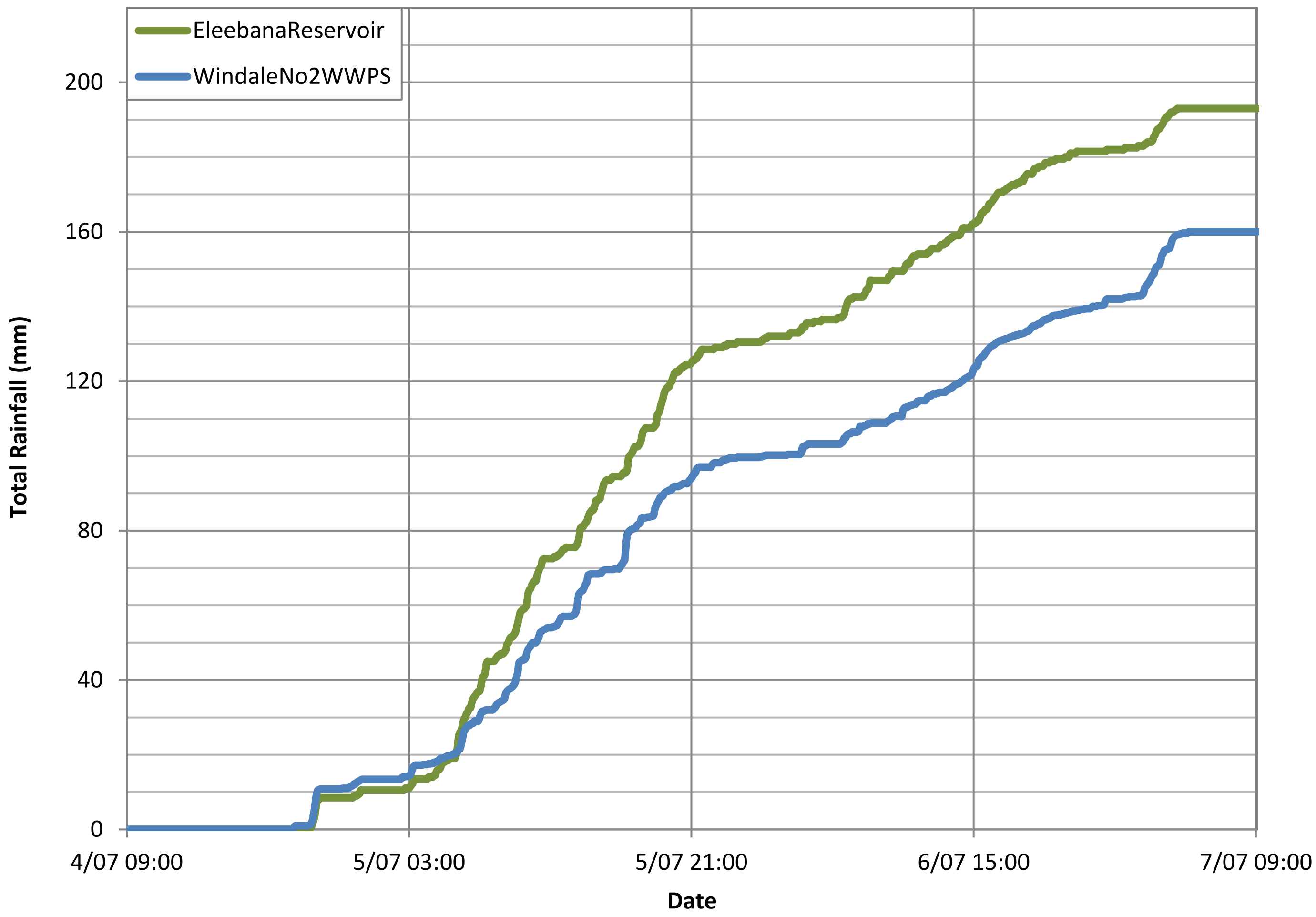
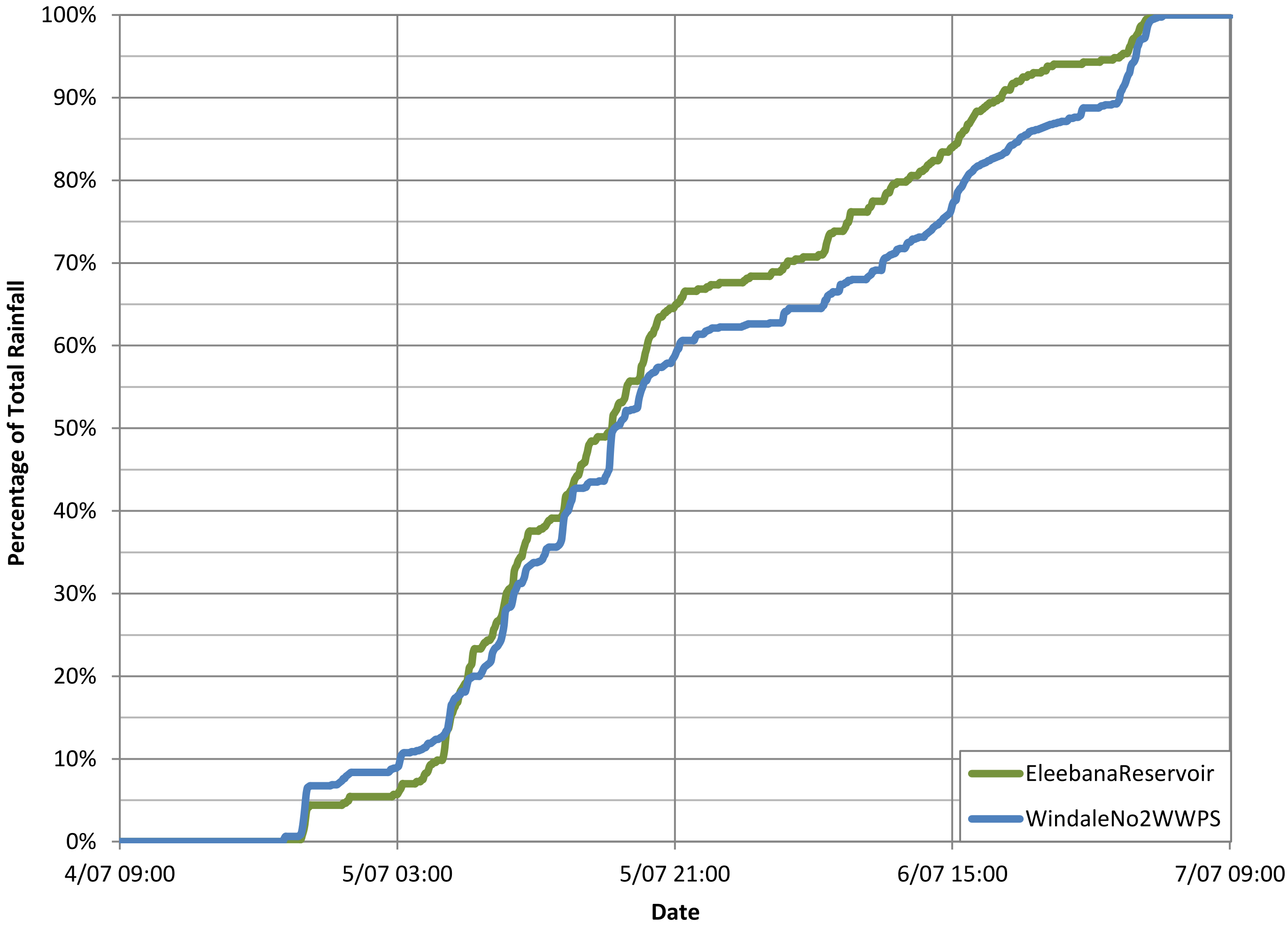


FIGURE 17
NORTH CREEK WARNERS BAY FLOOD STUDY
RAINFALL ISOHYETS
5 TO 7 JULY 2022
61031 : 140.8
61055 : 106.6
61133 : 244.2
561158 : 193
561172 : 160
235
230
225
220
215
210
205
200
Maitland
Ashtonfield
Beresfield
Williamtown
Fullerton Cove
Mayfield
Edgeworth
New Lambton
Newcastle
Toronto
Valentine
Study Area
Daily Rainfall
Sub-daily Rainfall
Rainfall Isohyets
Rainfall Depth (mm)
High : 250
Low : 200
0
0.625
1.25
2.5
3.75
5
Kilometres
N
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\Figures\Fig17_Rainfall_Isohyets_2022_03.mxd

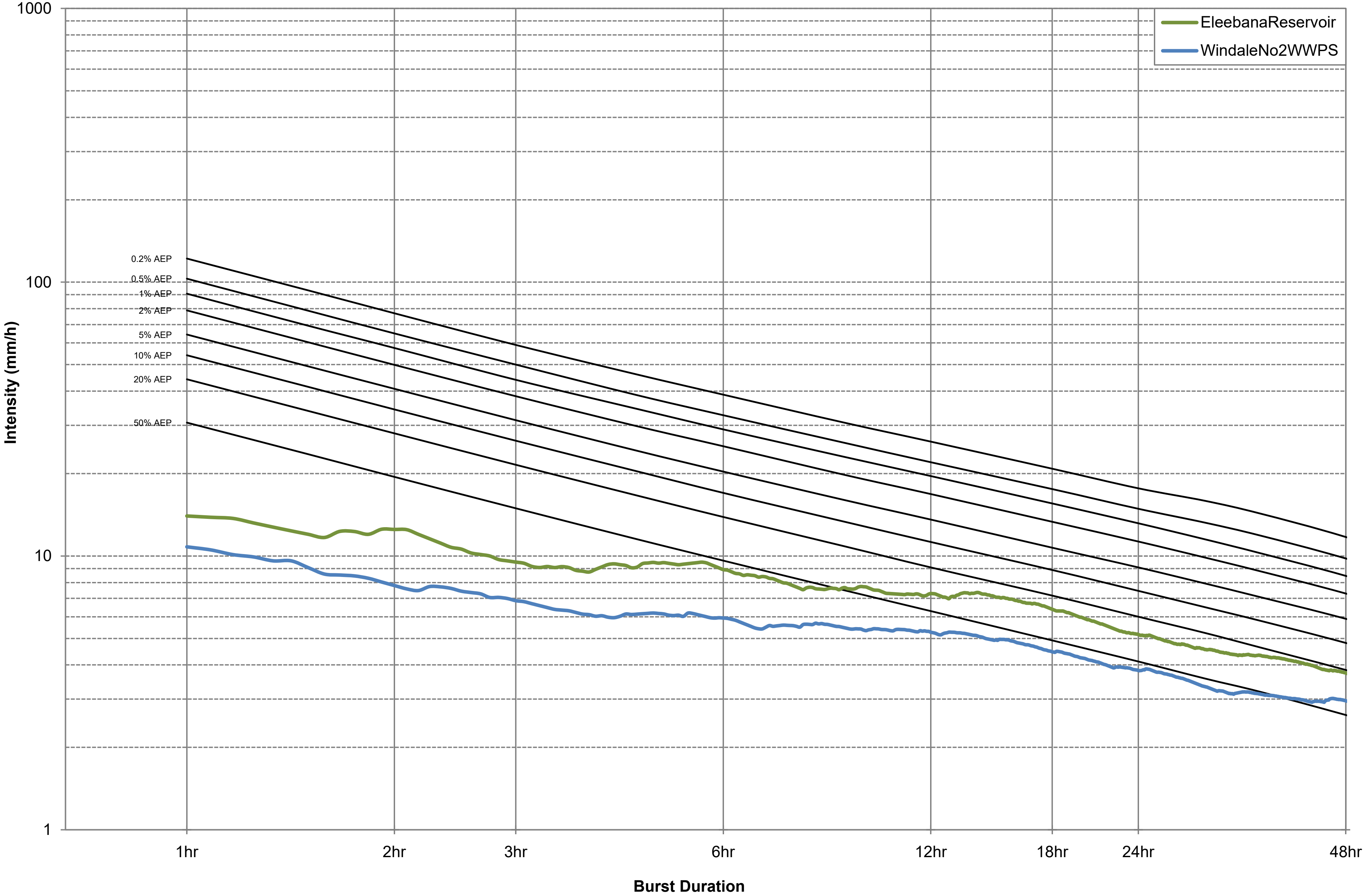
FIGURE 18
CUMULATIVE RAINFALL DATA
July 2022 EVENT





J:\Jobs\123045\Hydrology\Calibration\pluviometer\123045_Historical_Burst_IFD_Comparison_v1.4.xlsm

FIGURE 19
BURST INTENSITIES AND FREQUENCIES
July 2022 EVENT



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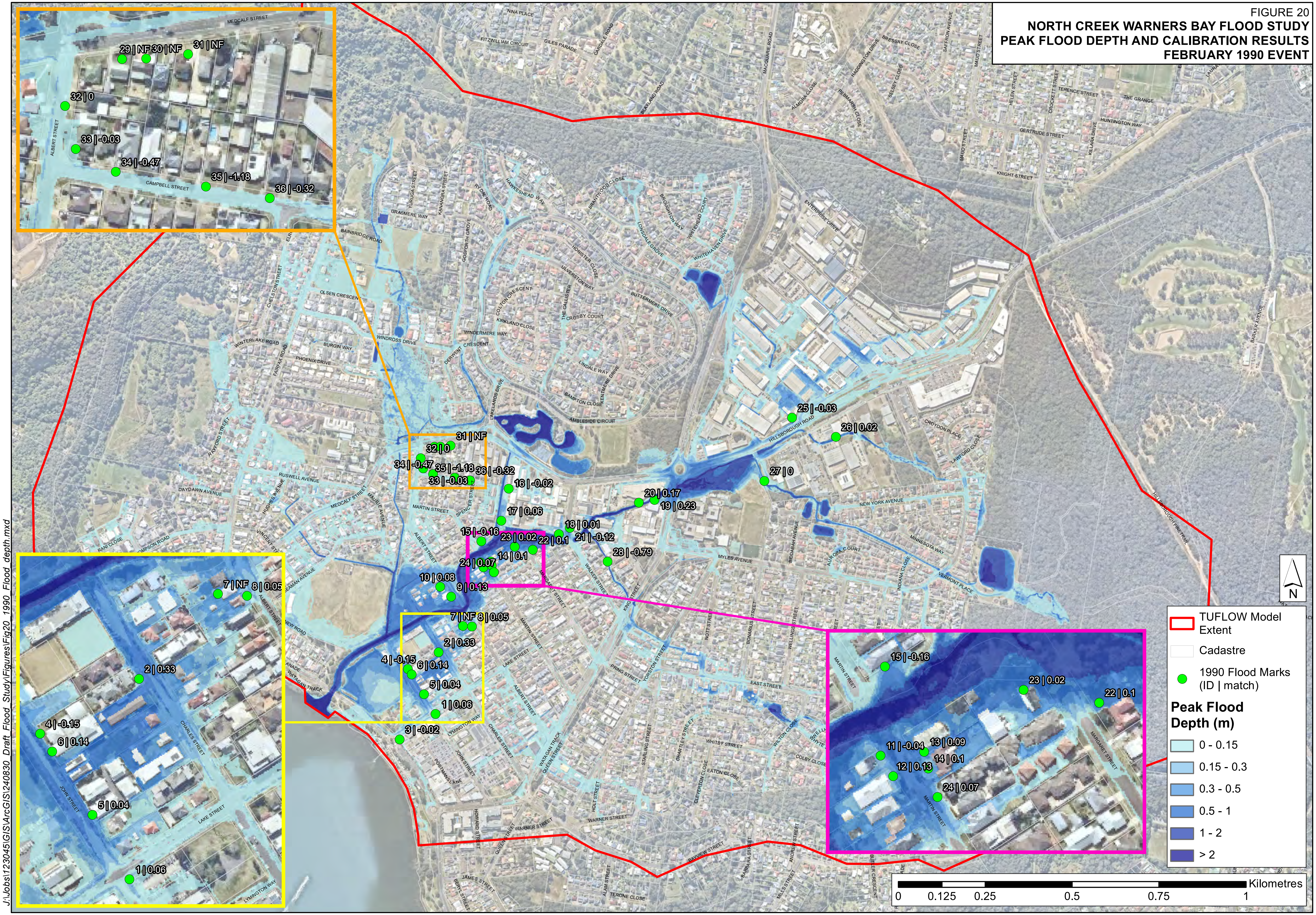

FIGURE 20
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH AND CALIBRATION RESULTS
FEBRUARY 1990 EVENT

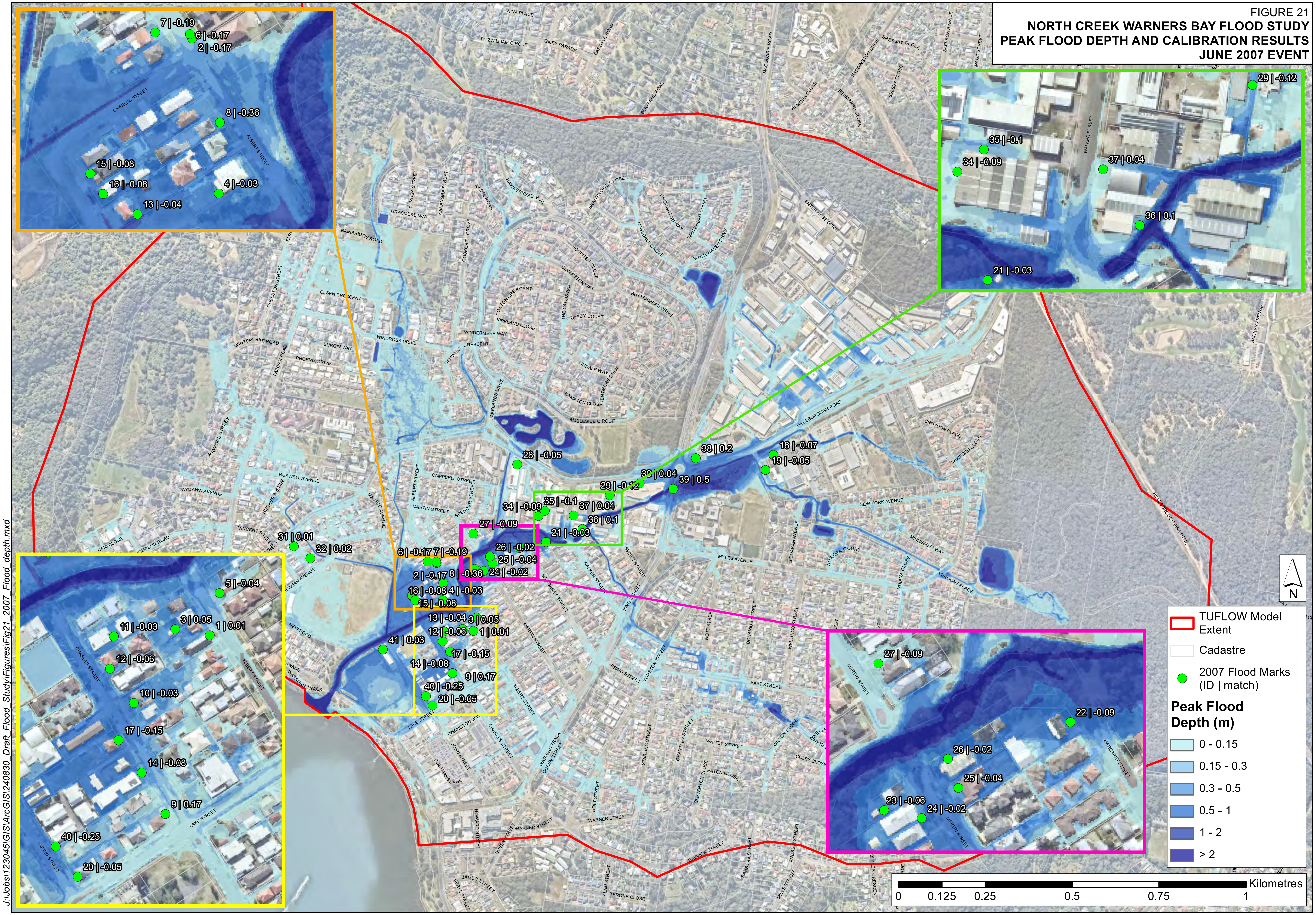
FIGURE 21
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH AND CALIBRATION RESULTS
JUNE 2007 EVENT
TUFLOW Model Extent
Cadastre
2007 Flood Marks (ID | match)
Peak Flood Depth (m)
0 - 0.15
0.15 - 0.3
0.3 - 0.5
0.5 - 1
1 - 2
> 2
N
Kilometres
0 0.125 0.25 0.5 0.75 1
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FIGURE 24
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH
JULY 2022 EVENT

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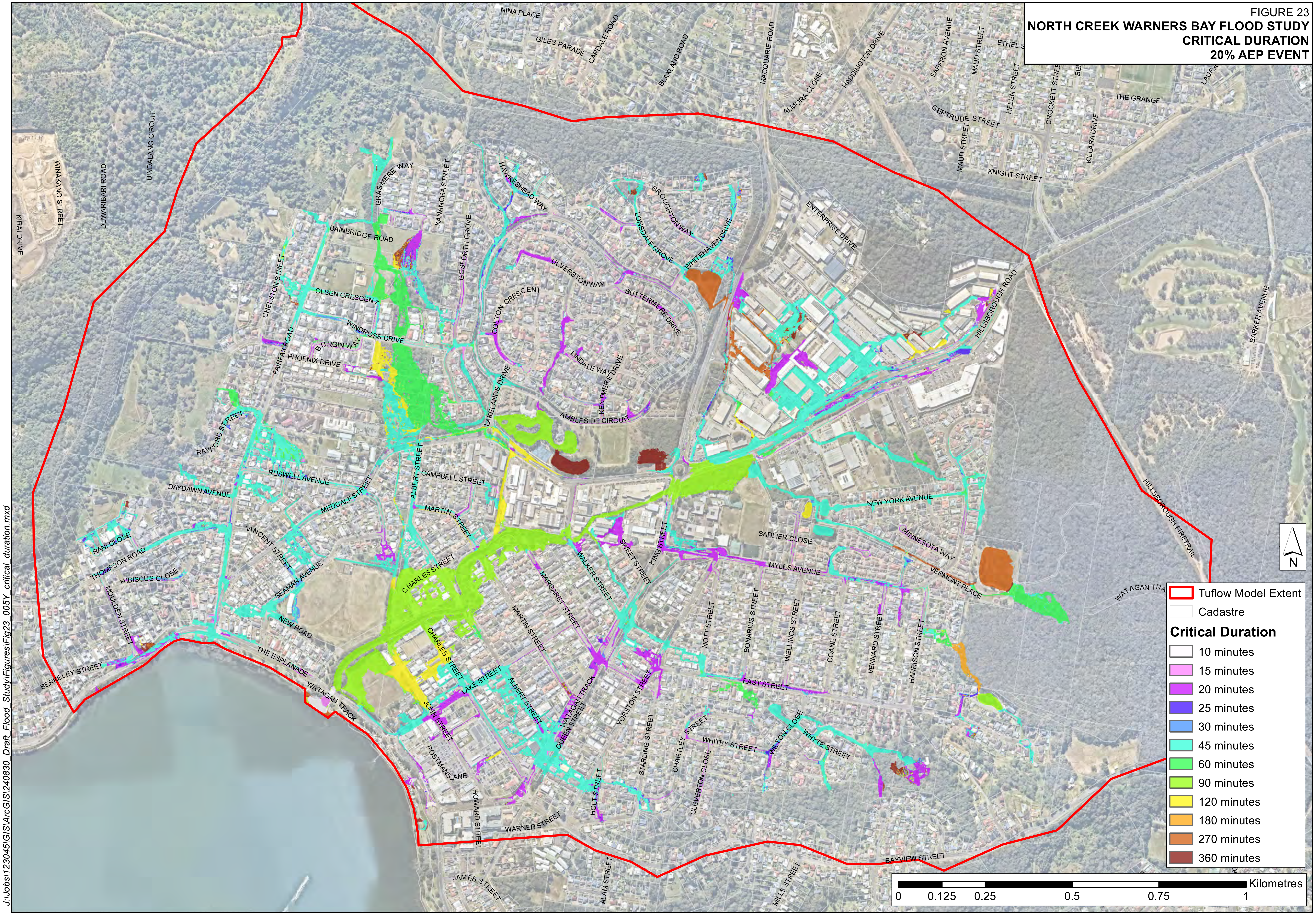
FIGURE 23
NORTH CREEK WARNERS BAY FLOOD STUDY
CRITICAL DURATION
20% AEP EVENT



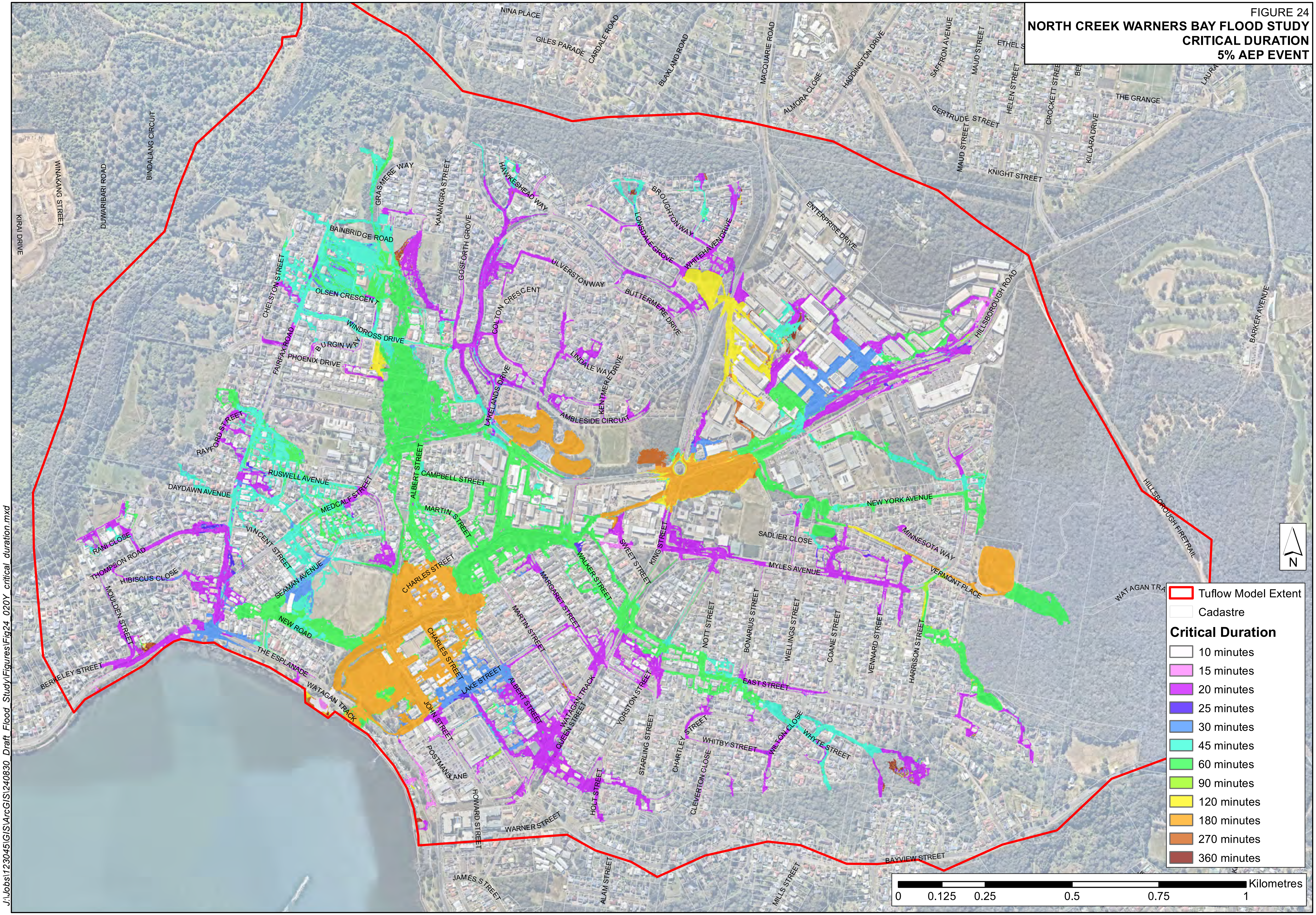


FIGURE 24
NORTH CREEK WARNERS BAY FLOOD STUDY
CRITICAL DURATION
5% AEP EVENT
Tuflow Model Extent
Cadastre
Critical Duration
10 minutes
15 minutes
20 minutes
25 minutes
30 minutes
45 minutes
60 minutes
90 minutes
120 minutes
180 minutes
270 minutes
360 minutes
0
0.125
0.25
0.5
0.75
1
Kilometres
N
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\Figures\Fig24_020Y_critical_duration.mxd
NINA PLACE
GILES PARADE
CARDALE ROAD
BLAXLAND ROAD
MACQUARIE ROAD
ALMORA CLOSE
HADDINGTON DRIVE
SAFFRON AVENUE
MAUD STREET
HELEN STREET
CROCKETT STREET
THE GRANGE
GERTRUDE STREET
KILLARA DRIVE
KNIGHT STREET
WINAKANG STREET
DUWARIBARI ROAD
BINDALANG CIRCUIT
KIRAI DRIVE
GRASMERE WAY
KANANGRA STREET
HAWKESHEAD WAY
BROUGHTON WAY
LONSDALE GROVE
WHITEHAVEN DRIVE
ENTERPRISE DRIVE
BAINBRIDGE ROAD
GOSFORTH GROVE
ULVERSTON WAY
BUTTERMERE DRIVE
COLTON CRESCENT
CHELSTON STREET
OLSEN CRESCENT
WINDROSS DRIVE
BURGIN WAY
PHOENIX DRIVE
FAIRFAX ROAD
LINDALE WAY
KENTMERE DRIVE
AMBLESIDE CIRCUIT
LAKELANDS DRIVE
HILLSBOROUGH ROAD
BARKER AVENUE
RAYFORD STREET
RUSWELL AVENUE
DAYDAWN AVENUE
MEDCALF STREET
ALBERT STREET
CAMPBELL STREET
MARTIN STREET
NEW YORK AVENUE
HILLSBOROUGH FIRETRAIL
RANI CLOSE
THOMPSON ROAD
HIBISCUS CLOSE
VINCENT STREET
SEAMAN AVENUE
CHARLES STREET
SWEET STREET
KING STREET
WALKER STREET
SADLIER CLOSE
MINNESOTA WAY
MYLES AVENUE
VERMONT PLACE
WATAGAN TRACK
MOULDEN STREET
NEW ROAD
THE ESPLANADE
BERKELEY STREET
MARGARET STREET
NOTT STREET
BONARIUS STREET
WELLINGS STREET
COANE STREET
VENNARD STREET
HARRISON STREET
LAKE STREET
JOHN STREET
QUEEN STREET
YORSTON STREET
EAST STREET
WILTON CLOSE
WHYTE STREET
WHITBY STREET
CHARTLEY STREET
STARLING STREET
CLEVERTON CLOSE
HOLT STREET
POSTMANS LANE
HOWARD STREET
WARNER STREET
JAMES STREET
ALAM STREET
MILLS STREET
BAYVIEW STREET

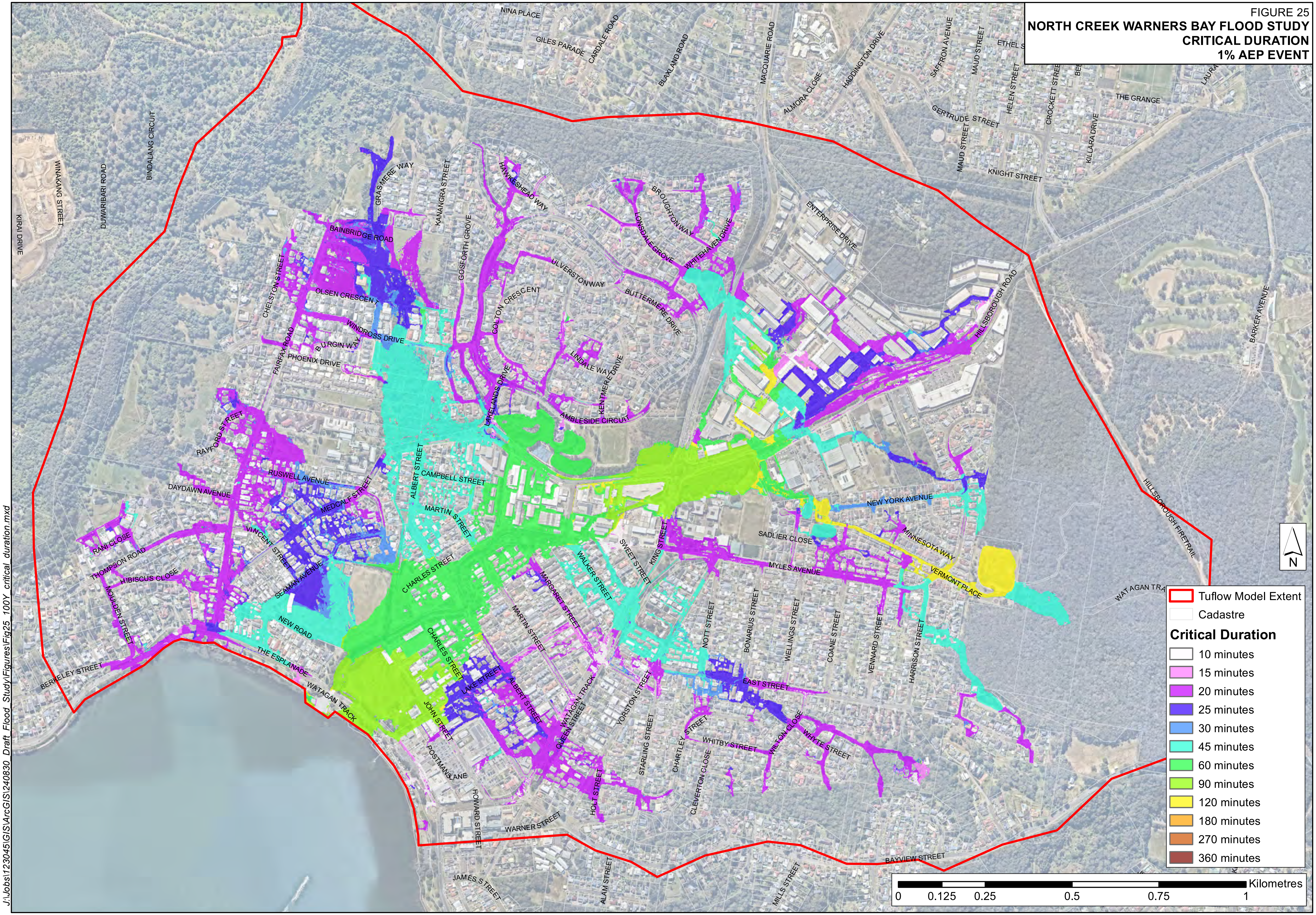


FIGURE 25
NORTH CREEK WARNERS BAY FLOOD STUDY
CRITICAL DURATION
1% AEP EVENT
Tuflow Model Extent
Cadastre
Critical Duration
10 minutes
15 minutes
20 minutes
25 minutes
30 minutes
45 minutes
60 minutes
90 minutes
120 minutes
180 minutes
270 minutes
360 minutes
0
0.125
0.25
0.5
0.75
1
Kilometres
N
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\Figures\Fig25_100Y_critical_duration.mxd
NINA PLACE
GILES PARADE
CARDALE ROAD
BLAXLAND ROAD
MACQUARIE ROAD
ALMORA CLOSE
HADDINGTON DRIVE
SAFFRON AVENUE
MAUD STREET
HELEN STREET
CROCKETT STREET
THE GRANGE
GERTRUDE STREET
KILLARA DRIVE
KNIGHT STREET
ENTERPRISE DRIVE
HILLSBOROUGH ROAD
BARKER AVENUE
HILLSBOROUGH FIRETRAIL
WATAGAN TRACK
KIRAI DRIVE
WINAKANG STREET
DUWARIBARI ROAD
BINDALANG CIRCUIT
GRASMERE WAY
KANANGRA STREET
HAWKSHEAD WAY
BROUGHTON WAY
WHITEHAVEN DRIVE
LONSDALE GROVE
BAINBRIDGE ROAD
GOSFORTH GROVE
ULVERSTON WAY
BUTTERMERE DRIVE
COLTON CRESCENT
CHELSTON STREET
OLSEN CRESCENT
WINDROSS DRIVE
BURGIN WAY
FAIRFAX ROAD
PHOENIX DRIVE
LINDALE WAY
KENTMERE DRIVE
LAKELANDS DRIVE
AMBLESIDE CIRCUIT
RAYFORD STREET
RUSWELL AVENUE
DAYDAWN AVENUE
MEDCALF STREET
ALBERT STREET
CAMPBELL STREET
MARTIN STREET
NEW YORK AVENUE
SADLIER CLOSE
MINNESOTA WAY
VERMONT PLACE
MYLES AVENUE
KING STREET
SWEET STREET
WALKER STREET
MARGARET STREET
RANI CLOSE
THOMPSON ROAD
HIBISCUS CLOSE
VINCENT STREET
SEAMAN AVENUE
CHARLES STREET
MOULDEN STREET
NEW ROAD
THE ESPLANADE
BERKELEY STREET
WATAGAN TRACK
JOHN STREET
LAKE STREET
NOTT STREET
BONARIUS STREET
WELLINGS STREET
COANE STREET
VENNARD STREET
HARRISON STREET
EAST STREET
YORSTON STREET
STARLING STREET
CHARTLEY STREET
WHITBY STREET
WILSON CLOSE
WHYTE STREET
CLEVERTON CLOSE
QUEEN STREET
HOLT STREET
POSTMANS LANE
HOWARD STREET
WARNER STREET
ALAM STREET
MILLS STREET
JAMES STREET
BAYVIEW STREET

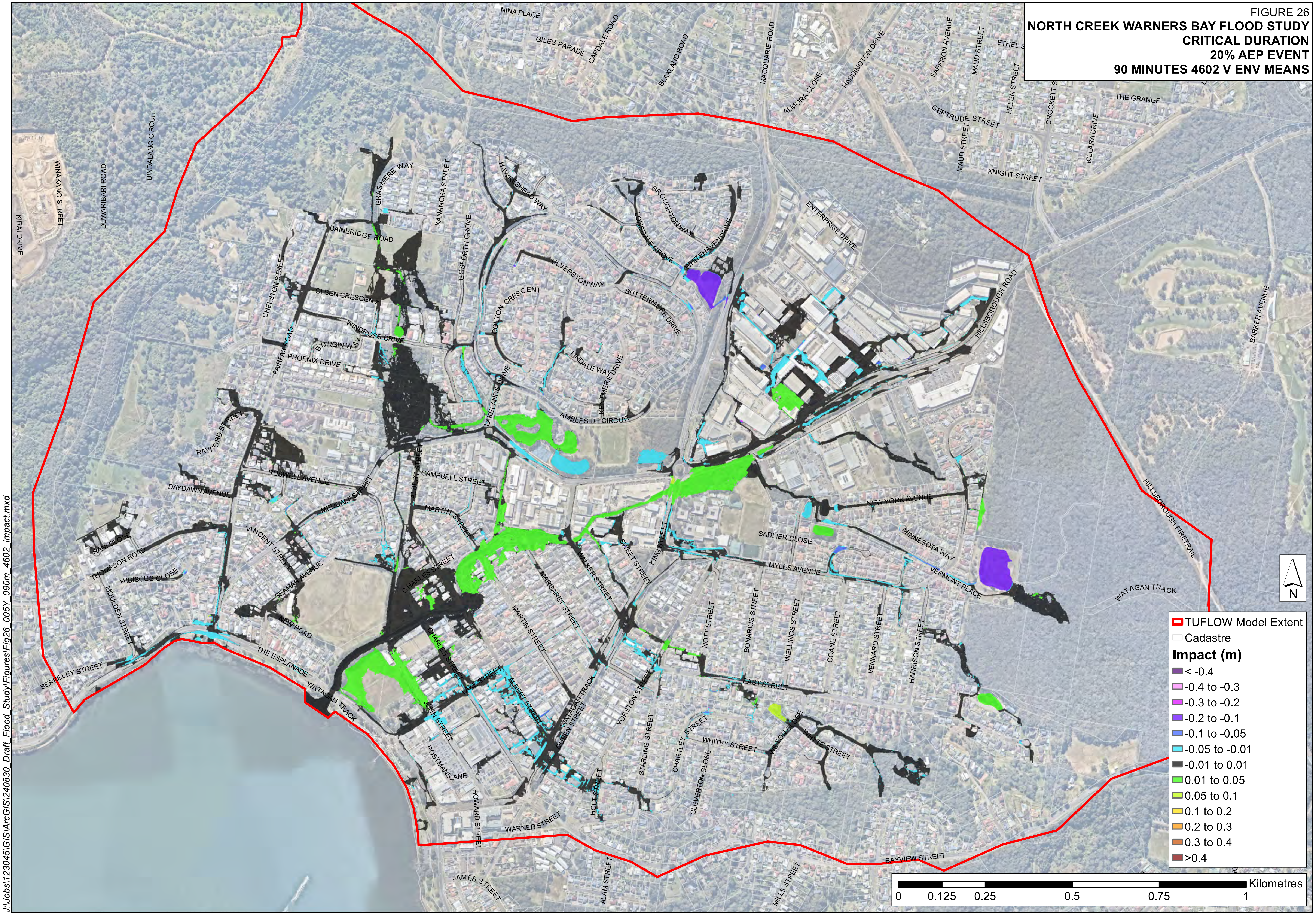


FIGURE 26
NORTH CREEK WARNERS BAY FLOOD STUDY
CRITICAL DURATION
20% AEP EVENT
90 MINUTES 4602 V ENV MEANS
TUFLOW Model Extent
Cadastre
Impact (m)
< -0.4
-0.4 to -0.3
-0.3 to -0.2
-0.2 to -0.1
-0.1 to -0.05
-0.05 to -0.01
-0.01 to 0.01
0.01 to 0.05
0.05 to 0.1
0.1 to 0.2
0.2 to 0.3
0.3 to 0.4
>0.4
0 0.125 0.25 0.5 0.75 1
Kilometres
N
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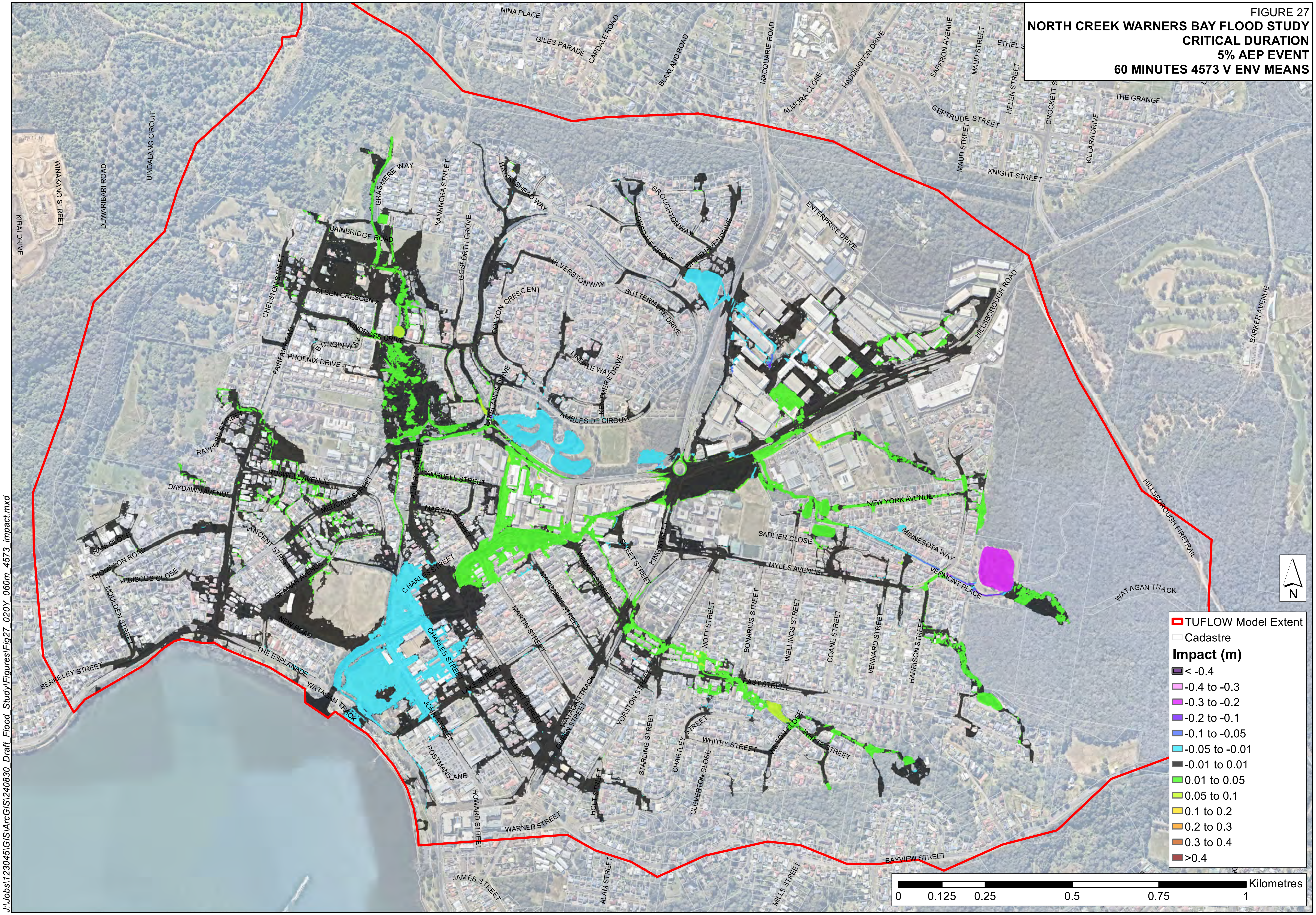

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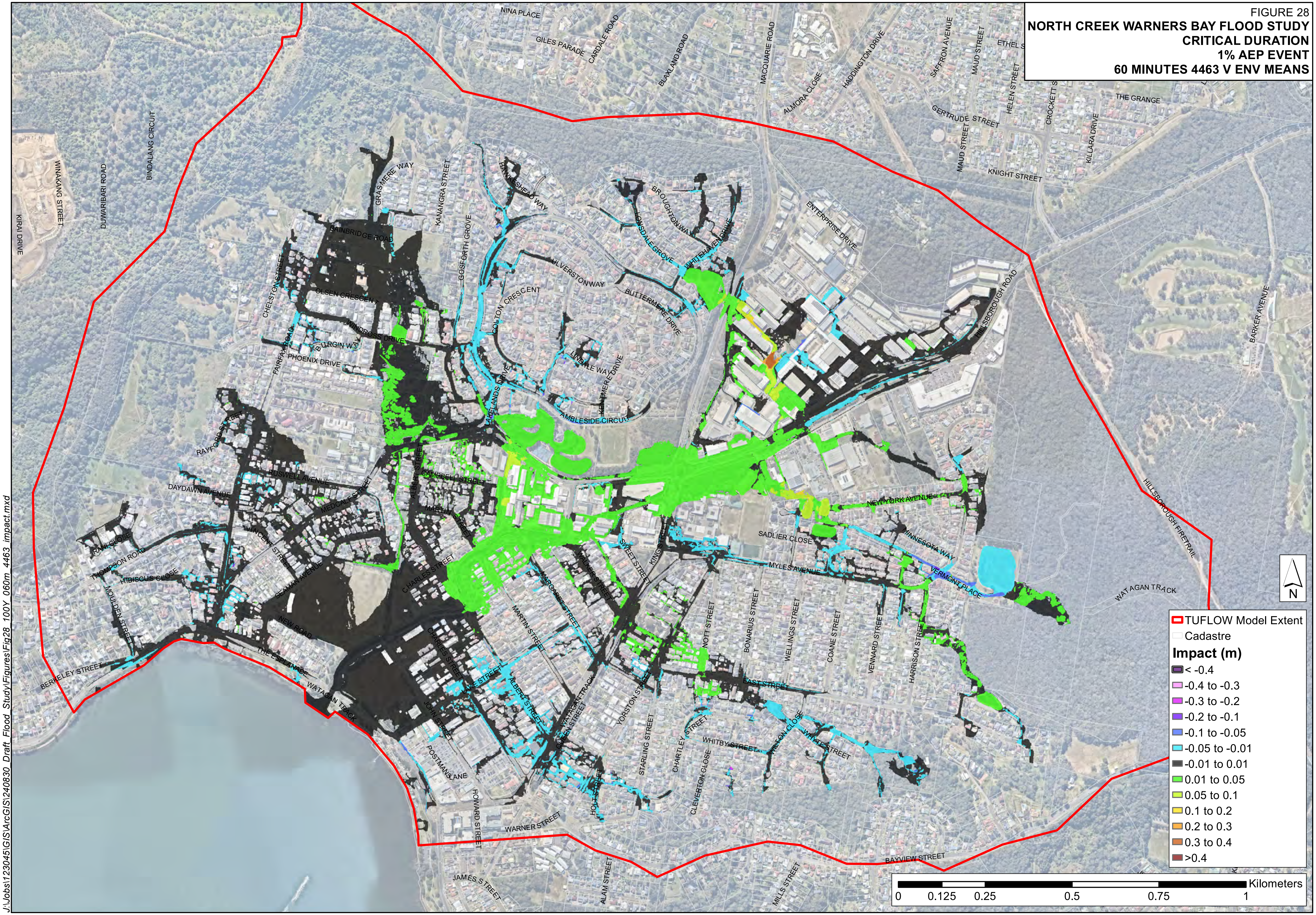

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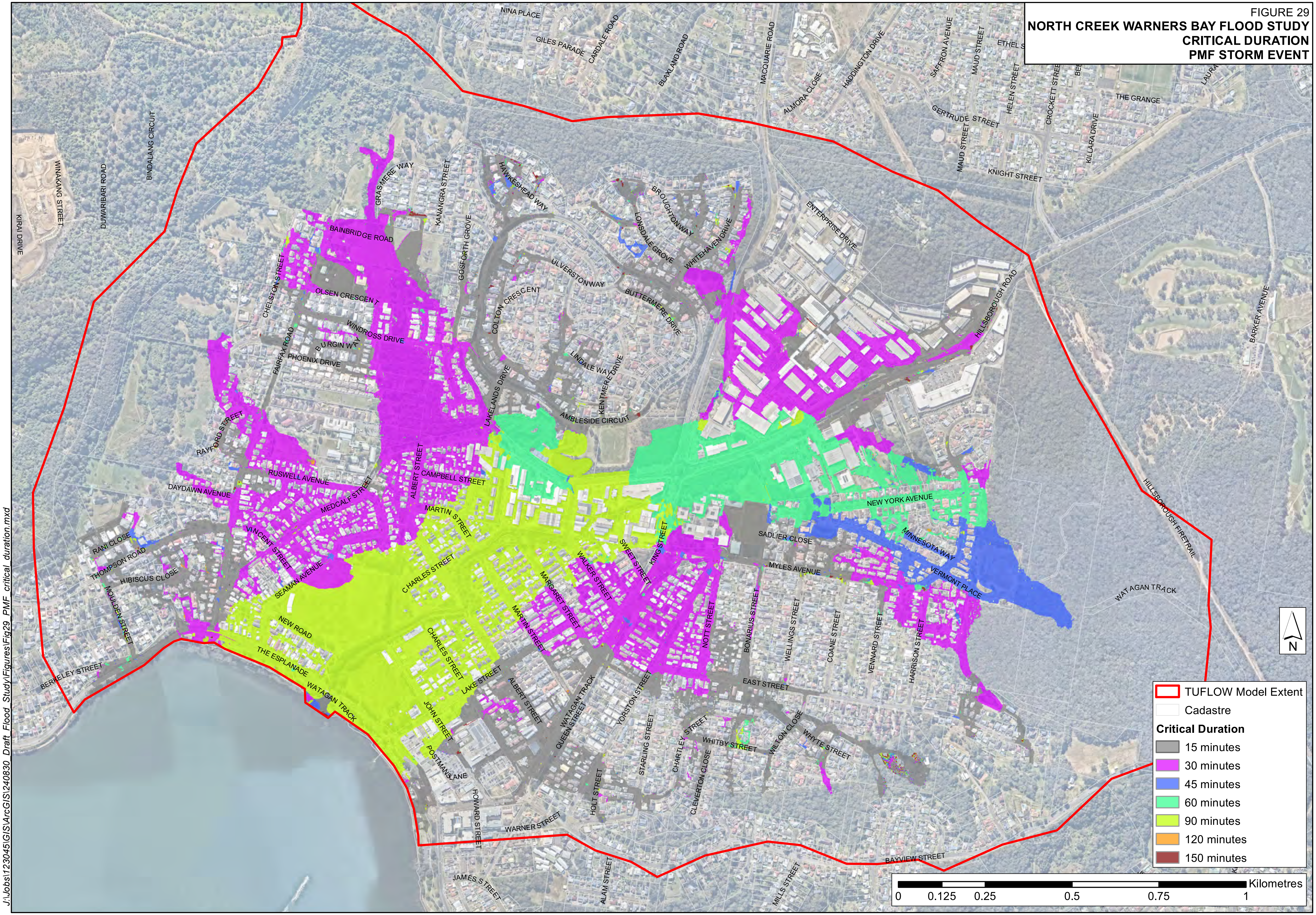
FIGURE 29
NORTH CREEK WARNERS BAY FLOOD STUDY
CRITICAL DURATION
PMF STORM EVENT



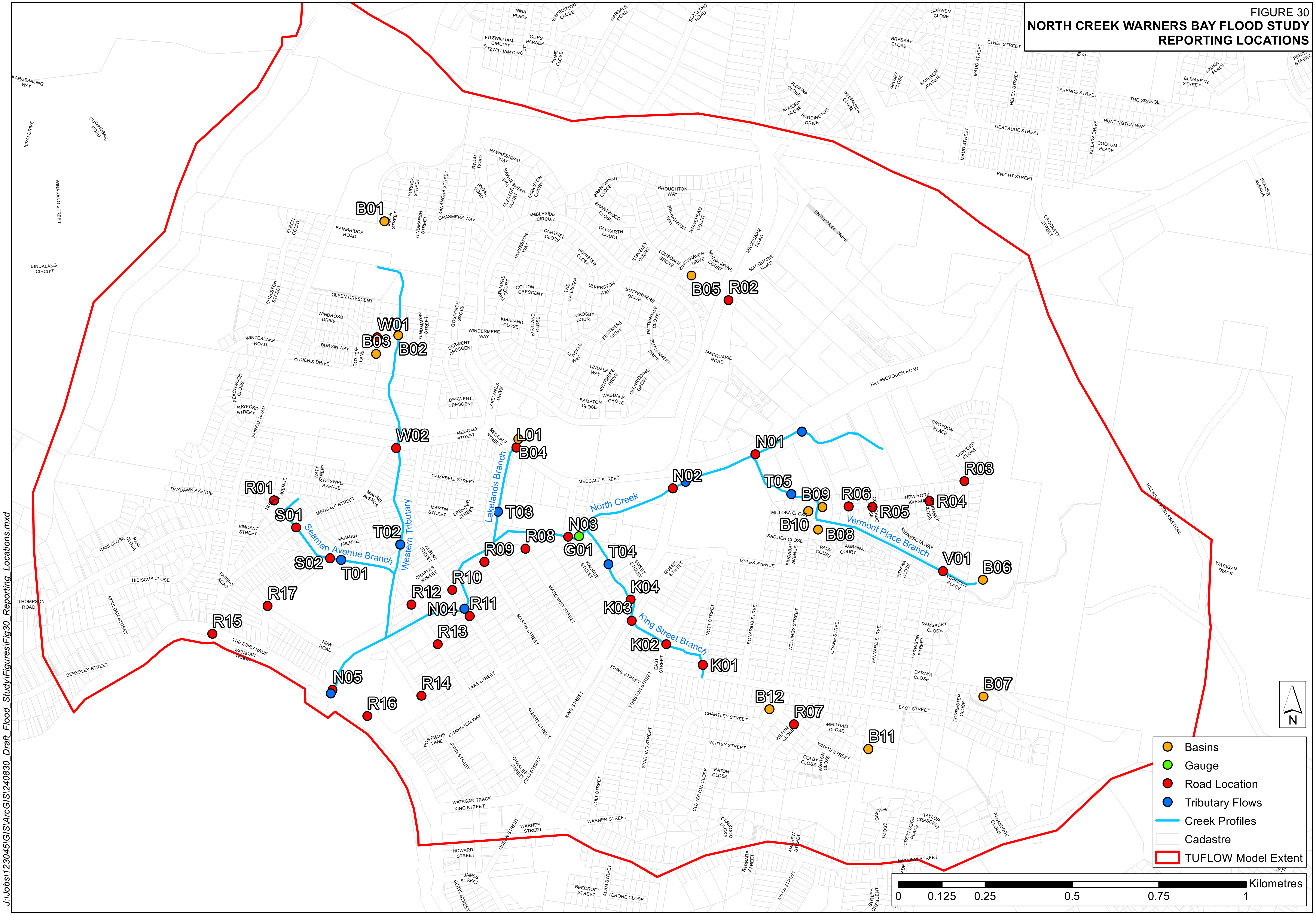


FIGURE 30
NORTH CREEK WARNERS BAY FLOOD STUDY
REPORTING LOCATIONS
North Creek
Western Tributary
Lakelands Branch
Seaman Avenue Branch
King Street Branch
Vermont Place Branch
B01
B02
B03
B04
B05
B06
B07
B08
B09
B10
B11
B12
W01
W02
L01
S01
S02
T01
T02
T03
T04
T05
N01
N02
N03
N04
N05
G01
K01
K02
K03
K04
V01
R01
R02
R03
R04
R05
R06
R07
R08
R09
R10
R11
R12
R13
R14
R15
R16
R17
Basins
Gauge
Road Location
Tributary Flows
Creek Profiles
Cadastre
TUFLOW Model Extent
N
0
0.125
0.25
0.5
0.75
1
Kilometres
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\Figures\Fig30_Reporting_Locations.mxd

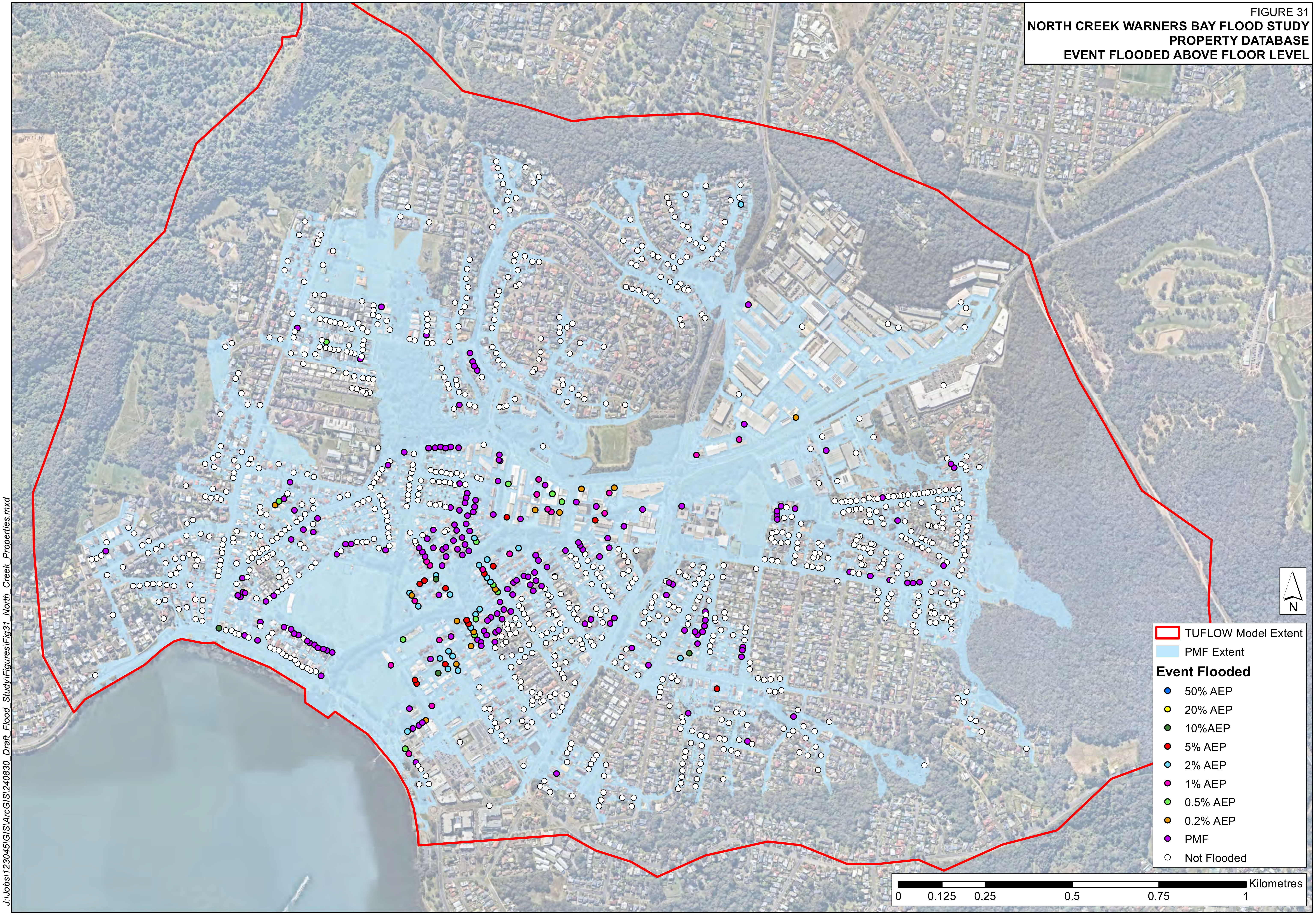


FIGURE 31
NORTH CREEK WARNERS BAY FLOOD STUDY
PROPERTY DATABASE
EVENT FLOODED ABOVE FLOOR LEVEL
N
TUFLOW Model Extent
PMF Extent
Event Flooded
50% AEP
20% AEP
10%AEP
5% AEP
2% AEP
1% AEP
0.5% AEP
0.2% AEP
PMF
Not Flooded
0 0.125 0.25 0.5 0.75 1
Kilometres
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Appendix A

APPENDIX A. GLOSSARY

Taken from the 2023 NSW Flood Risk Management Manual (Reference 2)

Afflux	Rise in water level in a waterway or flowpath caused by a structure, obstruction or impediment to flow.
Annual exceedance probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage
Astronomical tide	The variation in sea level caused by the gravitational effects of (principally) the moon and sun.
Australian height datum (AHD)	A common national surface level datum often used as a referenced level for ground, flood and flood levels
Australian Rainfall and Runoff (ARR)	A national guideline document, data and software suite that can be used for the estimation of design flood characteristics in Australia.
Average annual damage (AAD)	The average damage per year due to flooding that would occur in a nominated scenario in an area over a very long period of time.
Average recurrence interval (ARI)	The long-term average number of years between the occurrence of a flood equal to or larger in size than the selected event
Catchment	The area of land draining to a specific location
Backwater flooding	A mechanism by which upstream flooding is influenced by downstream conditions or controls.
Catchment flooding	Flooding due to prolonged or intense rainfall (e.g. severe thunderstorms, monsoonal rains in the tropics, tropical cyclones)
Chance	The likelihood of something happening that will have adverse or beneficial consequences
Coastal inundation	Inundation due to tidal or storm-driven coastal events, including storm surges in lower coastal waterways. This can be exacerbated by wind-wave generation from storm events
Consent authority	The authority or agency with the legislative power to determine the outcome of development and building applications
Consequence	The outcomes of an event or situation affecting objectives, expressed qualitatively or quantitatively
Continuing flood risk	Risk to existing and future development that may be reduced by EM measures
Defined flood event (DFE)	The flood event selected as a general standard for the management of flooding to development
Design flood	The flood selected as part of the FRM process that forms the basis for physical works to modify the impacts of flooding
Development	May be treated differently depending on the following categorisation:

- **infill development**: the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under current land zoning
- **new development**: development of a completely different nature to that associated with the former land-use (e.g. the urban subdivision of a previously rural area)
- **redevelopment**: rebuilding in an area (e.g. as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale)

Development control plan (DCP)	See Environmental Planning and Assessment Act 1979
Ecologically sustainable development (ESD)	As outlined in the Local Government Act 1993.
Emergency management (EM)	A comprehensive approach to dealing with risks to the community arising from hazards. It is a systematic method for identifying, analysing, evaluating and managing these risks
Emergency management plan (EMPLAN)	The overarching EM arrangements for New South Wales, including the agreed roles and functions of various agencies. All NSW Government agencies with responsibilities and functions in disaster response and recovery contribute to this plan.
Emergency management response strategy (EM response strategy)	A strategy identified by the combat agency typically used to plan, prepare for and respond to a hazard.
Events per year (EY)	Number of events per year.
Existing flood risk	The risk an existing community is exposed to as a result of its location on the floodplain
Flash flood	Flood that is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding that peaks within 6 hours of the causative rain.
Flood	A natural phenomenon that occurs when water covers land that is normally dry. It may result from coastal inundation (excluding tsunamis) or catchment flooding, or a combination of both
Flood classifications (used in flood warnings)	• **Minor flooding** – Causes inconvenience. Low-lying areas next to watercourses are inundated. Minor roads may be closed and low-level bridges submerged. Flooding is usually below the floor level of dwellings and may require removal of stock and equipment from low-lying areas. • **Moderate flooding** – In addition to the above, the area of inundation is more substantial. Main traffic routes may be affected. Some buildings may be affected above the floor level. Evacuation may be required. • **Major flooding** – In addition to the above, extensive rural areas and/or urban areas are inundated. Many buildings may be affected above the floor level. Properties and towns are likely to be isolated and major rail and traffic routes closed. Evacuation of flood affected areas may be required. Utility services may be impacted.
Flood affected land	Equivalent to flood prone land

Flood awareness	An appreciation of the likely effects of flooding, and a knowledge of the relevant flood warning, response and evacuation procedures facilitating prompt and effective community response to a flood threat
Flood constraints	Key constraints that flooding place on land
Flood damage	The tangible (direct and indirect) and intangible costs (financial, opportunity costs, clean-up) of flooding
Flood education	Seeks to provide information to raise community awareness of flooding so as to enable individuals to understand how to manage themselves and their property in response to flood warnings
Flood emergency response classification of communities (FERCC)	Classification of the floodplain in consideration of the EM constraints and consequences.
Flood evacuation	The movement of people from a place of danger to a place of relative safety, and their eventual return
Flood evacuation capability	The ability to safely evacuate to an area of relative safety within the effective warning time, having regard to the suitability and capacity of the route and the possible prevailing environmental conditions.
Flood fringe areas	That part of the flood extents for the event remaining after the flood function areas of floodway and flood storage areas have been defined
Flood function	The flood related functions of floodways, flood storage and flood fringe within the floodplain
Flood hazard	A flood that has the potential to cause harm or conditions with the potential to result in loss of life, injury and economic loss
Flood hazard categorisation	Categorisation of flood affected areas based on the degree of hazard that the flood conditions may present to people, vehicles and structures.
Flood impact and risk assessment (FIRA)	A study to assess flood behaviour, constraints and risk, understand offsite flood impacts on property and the community resulting from the development, and flood risk to the development and its users
Flood liable land	Equivalent to flood prone land
Flood mitigation standard	The design flood selected as part of the FRM process that forms the basis for physical works to modify the impacts of flooding.
Flood (hydrologic and hydraulic) modelling	Hydrologic and hydraulic computer models to simulate catchment processes of rainfall, run-off, stream flow and distribution of flows across the floodplain or similar
Flood plan (local or state)	A sub-plan of an EM plan that deals specifically with flooding; they can exist at state, zone and local levels
Flood planning area (FPA)	The area of land below the FPL
Flood planning constraint categories (FPCCs)	Categorisation of the floodplain into areas of different degrees and types of flood related constraints.

Flood planning level (FPL)	The combination of the flood level from the DFE and freeboard selected for FRM purposes
Flood prone land	Land susceptible to flooding by the PMF event
Flood proofing	Measures incorporated in the design, construction or alteration of individual buildings or structures that are subject to flooding, to reduce structural damage and potentially, in some cases, reduce contents damage.
Flood risk	Risk is based on the consideration of the consequences of the full range of flood behaviour on communities and their social settings, and the natural and built environment
Flood risk management (FRM)	The management of flood risk to communities
Flood storage areas	Areas of the floodplain that are outside floodways which generally provide for temporary storage of floodwaters during the passage of a flood and where flood behaviour is sensitive to changes that impact on temporary storage of water during a flood
Flood study	A comprehensive technical investigation of flood behaviour undertaken in accordance with the principles in this manual and consistent with associated guidelines A flood study defines the nature of flood behaviour and hazard across the floodplain by providing information on the extent, level and velocity of floodwaters, and on the distribution of flood flows considering the full range of flood events up to and including extreme events, such as the PMF
Flood warnings	Warnings issued when there is more certainty that flooding is expected, are more targeted and are issued for specific catchments
Flood watches	Provide the community with early advice of a developing situation that may lead to flooding.
Floodplain	Equivalent to flood prone land
Floodways	Areas of the floodplain which generally convey a significant discharge of water during floods and are sensitive to changes that impact flow conveyance. They often align with naturally defined channels or form elsewhere in the floodplain
Flow	The rate of flow of water measured in volume per unit time, for example, cubic metres per second (m^3/s)
Freeboard	A factor of safety typically used in relation to the setting of minimum floor levels or levee crest levels
Frequency	The measure of likelihood expressed as the number of occurrences of a specified event in a given time
FRM measures	Measures that can reduce flood risk
FRM options	The FRM measures that might be feasible for the management of a particular area of the floodplain
FRM plan	A management plan developed in accordance with the principles in this manual and its supporting guidelines

FRM study	A management study developed in accordance with the principles in this manual and its supporting guidelines
Future flood risk	The risk future development and its users are exposed to as a result of its location on the floodplain
Gauge height	The height of a flood level at a particular water level gauge site related to a specified datum
Habitable room	In a residential development – a room used for normal domestic activities that: • includes a bedroom, living room, lounge room, music room, television room, kitchen, dining room, sewing room, study, playroom, family room, home theatre and sunroom • includes a bedroom, living room, lounge room, music room, television room, kitchen, dining room, sewing room, study, playroom, family room, home theatre and sunroom In an industrial or commercial situation – an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
Hazard	A source of potential harm or conditions that may result in loss of life, injury and economic loss due to flooding
Hydraulics	The study of water flow in waterways and flowpaths; in particular, the evaluation of flow parameters such as water level and velocity
Hydrograph	A graph that shows how the discharge or stage/flood level at any location varies with time during a flood.
Hydrology	The study of the rainfall and run-off process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods
Integrated planning and reporting framework (IP&R framework)	The IP&R framework includes a suite of integrated plans that set out a vision and goals and strategic actions to achieve them. It involves a reporting structure to communicate progress to council and the community as well as a structured timeline for review to ensure the goals and actions are still relevant
Lifecycle costing	All of the costs associated with the project. This usually includes investigation, design, construction, operation, monitoring, maintenance, asset and performance management and, in some cases, renewal, upgrade, decommissioning and disposal of a management measure.
Likelihood	A qualitative description of probability and frequency
Likelihood of occurrence	The likelihood that a specified event will occur
Local environmental plan (LEP)	See Environmental Planning and Assessment Act 1979
Local government area (LGA)	The area serviced by the local government council
Local overland flooding (LOF)	Inundation by local run-off on its way to a waterway, rather than overbank flow from a waterway
Local strategic planning statement (LSPS)	Local strategic planning statements assist councils to implement the priorities set out in their community strategic plan and actions in regional and district plans

Loss	Any negative consequence or adverse effect, financial or otherwise
Mainstream flooding	Inundation resulting from overbank flow from a waterway rather than by local run-off.
Merit-based approach	Weighs social, economic, ecological and cultural impacts of land-use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and wellbeing of the state's rivers and floodplains
NSW Floodplain Management Program	The NSW Government's program of technical support and financial assistance to local councils to enable them to understand and manage their flood risk
NSW Flood prone land policy	The NSW Flood prone land policy included in the NSW Flood Risk Management Manual (2023)
Peak flow	The maximum flow occurring during a flood of a given annual exceedance probability.
Prevention, preparedness, response and recovery (PPRR)	Involves: • **prevention:** to eliminate or reduce the level of the risk or severity of emergencies • **preparedness:** enhances the capacity of agencies and communities to cope with the consequences of emergencies • **response:** to ensure the immediate consequences of emergencies to communities are minimized • **recovery:** measures that support individuals and communities affected by emergencies in the reconstruction of physical infrastructure and restoration of physical, emotional, environmental and economic wellbeing
Probability	A statistical measure of the expected chance of a flood
Probable maximum flood (PMF)	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation (PMP), and where applicable, snow melt, coupled with the worst flood-producing catchment conditions
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 Organization 1986)
Rainfall intensity	The rate at which rain falls, typically measured in millimetres per hour (mm/h)
Residual flood risk	The risk to the existing and future community that remains with FRM, EM and land-use planning measures in place to address flood risk
Risk	'The effect of uncertainty on objectives' (ISO 2018)
Risk analysis	The systematic use of available information to determine how often specified (flood) events occur and the magnitude of their likely consequences
Run-off	The amount of rainfall that ends up as streamflow, also known as rainfall excess
Severe thunderstorm warnings	Warnings provided to communities of the threat of dangerous thunderstorms. They are issued when a severe thunderstorm is occurring or likely to occur.

Severe weather warnings	Warnings provided for potentially hazardous or dangerous weather that is not solely related to severe thunderstorms, tropical cyclones or bushfires. They are issued whenever severe weather is occurring in an area or is expected to develop or move into an area.
State environmental planning policy	See Environmental Planning and Assessment Act 1979
Scenario	A scenario may relate to current, historical or assumed future floodplain, catchment and climate conditions
Stage	Equivalent to water level; measured with reference to a specified datum
Stage hydrograph	A graph that shows how the water levels at a particular location change with time during a flood. It must be referenced to a particular datum.
Storm surge	The increases in coastal water levels above predicted astronomical tide level (i.e. tidal anomaly) resulting from a range of location-dependent factors
Survey plan	A plan prepared by a registered surveyor.
Temporal pattern	The variation of rainfall intensity with time during a rainfall event.
Tidal anomaly	The difference between recorded storm surge levels and predicted astronomical tide level.
Tipping point	The critical point in a situation, process or system beyond which a significant and often unstoppable effect or change takes place.
Total warning system (TWS)	A total warning system describes a means of collecting information about an impending emergency, understanding the nature of the threat, communicating that information to those likely to be affected by it, and facilitating protective action and timely response.
Total warning system for flood (TWSF)	An integrated system defining the level of flooding at which a warning will be initiated, the physical means by which it will be relayed, and the persons to whom it will be given. The system includes all necessary hardware such as water level actuators, and radio transmitting and receiving equipment.
Velocity	The speed of floodwaters, measured in metres per second (m/s)
Vulnerability	The degree of susceptibility and resilience of a community, its social setting, and the built environment to flooding
Water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
Wave set-up	The increase in water levels in coastal waters (within the breaker zone) caused by waves transporting water shoreward. The zone of wave set-up against the shore is balanced by a zone of wave 'set-down' (i.e. reduced water levels) seawards of the breaker zone.
Wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.
Wind set-up	The increase in water levels in coastal and inland waterways caused by the wind driving the water shoreward and 'piling it up' against the shore.

APPENDIX B. COMMUNITY CONSULTATION SUMMARY

Appendix B

NORTH CREEK FLOOD STUDY

ENGAGEMENT SUMMARY 29 NOVEMBER 2023 - 10 JANUARY 2024

Lake Macquarie City Council is working with consultant WMAwater to prepare a flood study of North Creek.

As part of this work, residents from suburbs along North Creek, including Warners Bay, Lakelands and part of Speers Point, were invited to share their stories and observations of historical flooding in the area.

WHAT WE ASKED

- Has your property been affected by flooding in the past?
- How long have you lived in the property?
- Where has your property been affected by flooding?
- During which storm events has your property been impacted?
- Can you describe the flooding that occurred in each event?
- Do you have any photos or videos of past floods or flood level marks?
- Are you aware of any other locations within the North Creek Flood Study area that have experienced flooding in the past?

We reached

- **345** visits to our Shape Lake Mac webpage
- **11** survey responses
- **2800** letters/flyers sent to properties within the study area
- **7115** recipients reached through Council e-newsletters

ENGAGEMENT SNAPSHOT

Residents from Warners Bay, Lakelands and Speers Point shared their experiences of flooding around North Creek.

Respondents have lived around North Creek for varying periods from 12 months to 24 years, and experienced flood events in February 1990, April 2001, June 2007 and April 2015.

The majority of respondents (81.8 per cent) said their property hadn't been affected by flooding in the past.

Of the properties that had been affected by floods, flooding occurred in the front and backyards and in a garage/shed.

COMMUNITY COMMENTS

"We had to get our pool fixed from the June [2007] floods. We had rocks and dirt in our pool from the hill behind our house".

"Deep across back and down [the] side of building – mud everywhere".

"The roundabout near Warners Bay High School, the creek at Walker Street [has experienced flooding in the past]. Lots of places I have seen in my years living in Warners Bay".

NEXT STEPS

Community feedback will be combined with other data sources to develop modelling that will help us better understand and manage future flood risks along North Creek.

We expect the North Creek Flood Study will be complete and presented to Council for adoption in early 2025.

30602 24.1.24

APPENDIX C. DESIGN FLOOD FIGURES

Appendix C

FIGURE C1
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH
50% AEP EVENT

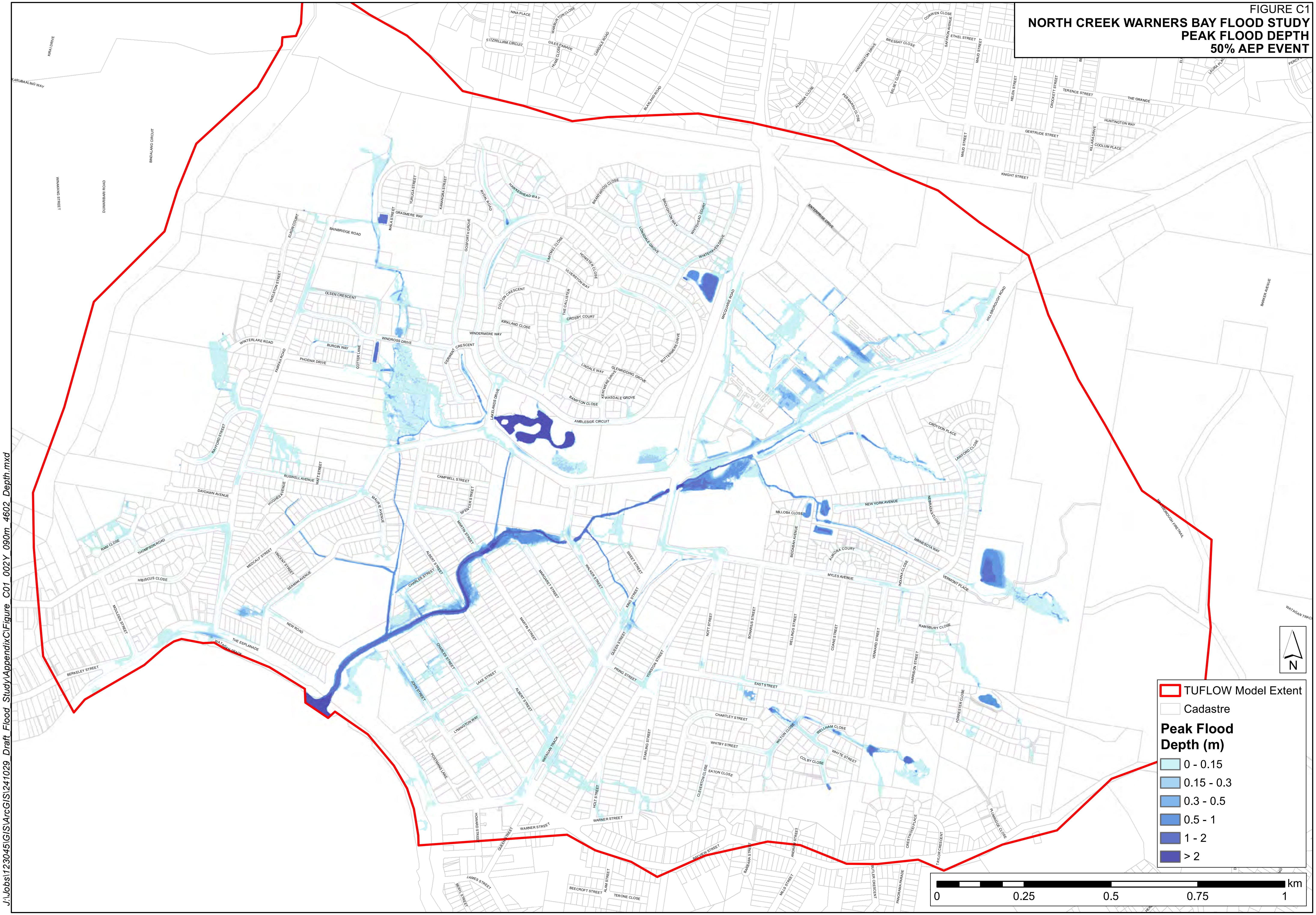

FIGURE C2
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH
20% AEP EVENT

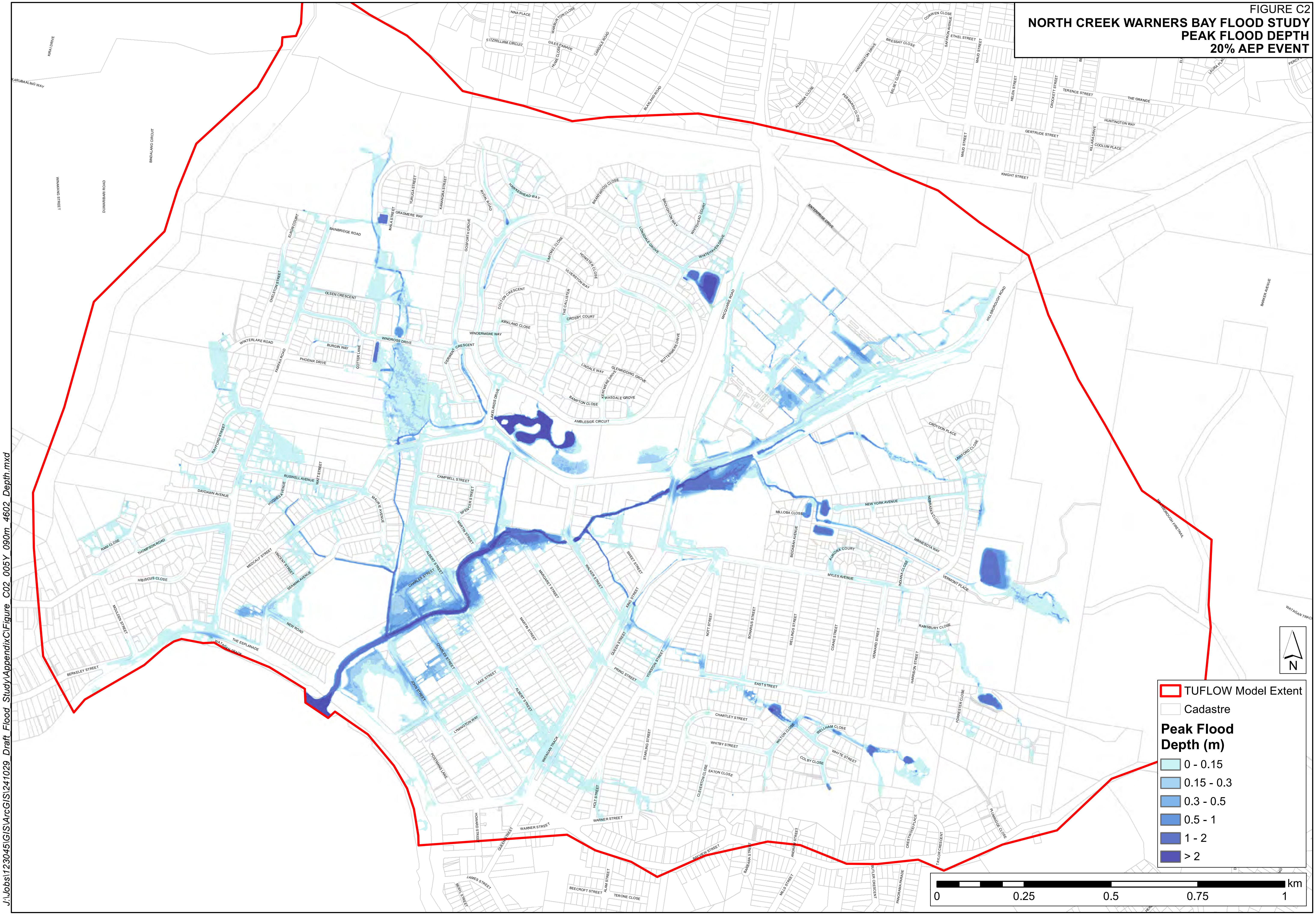

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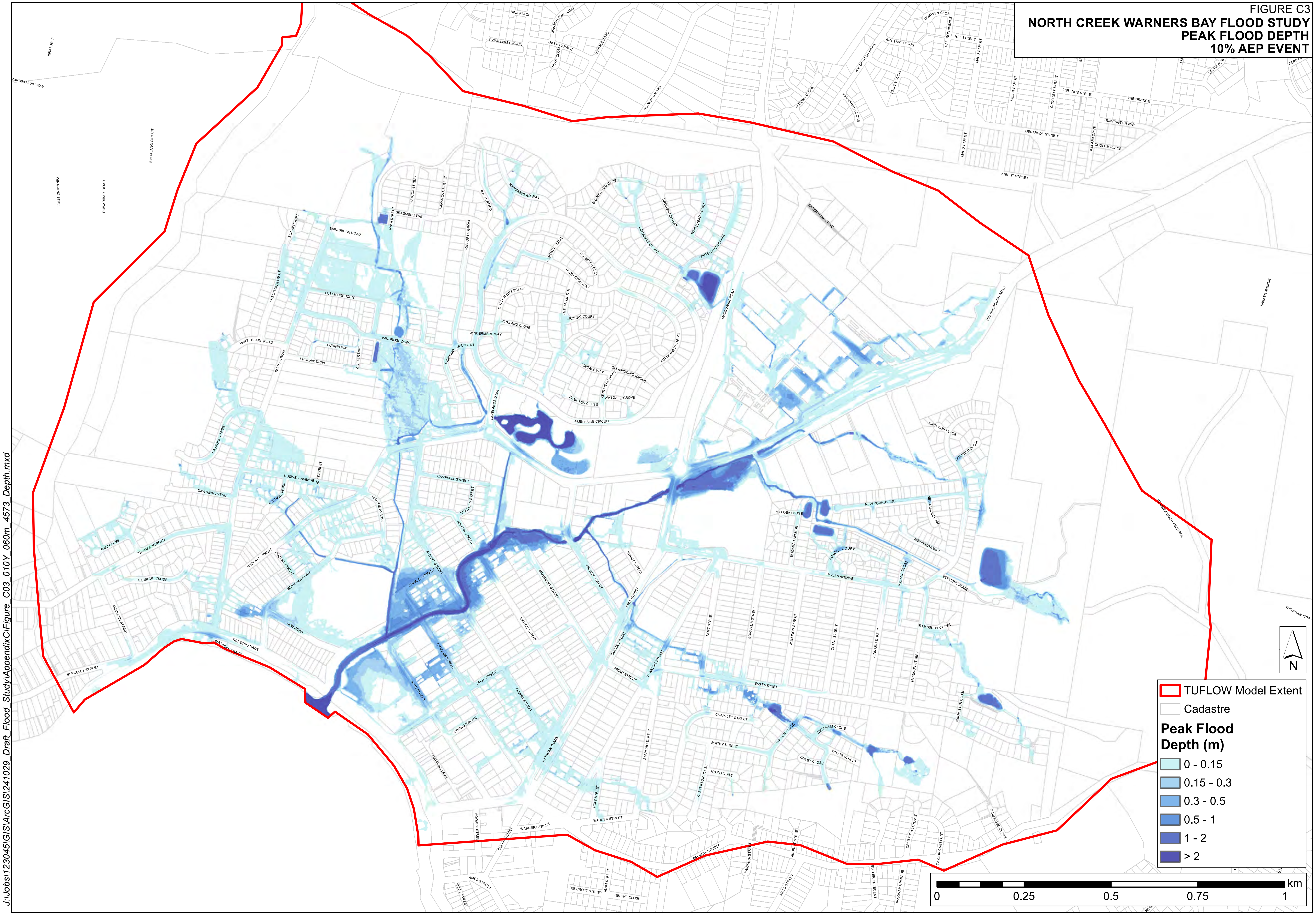
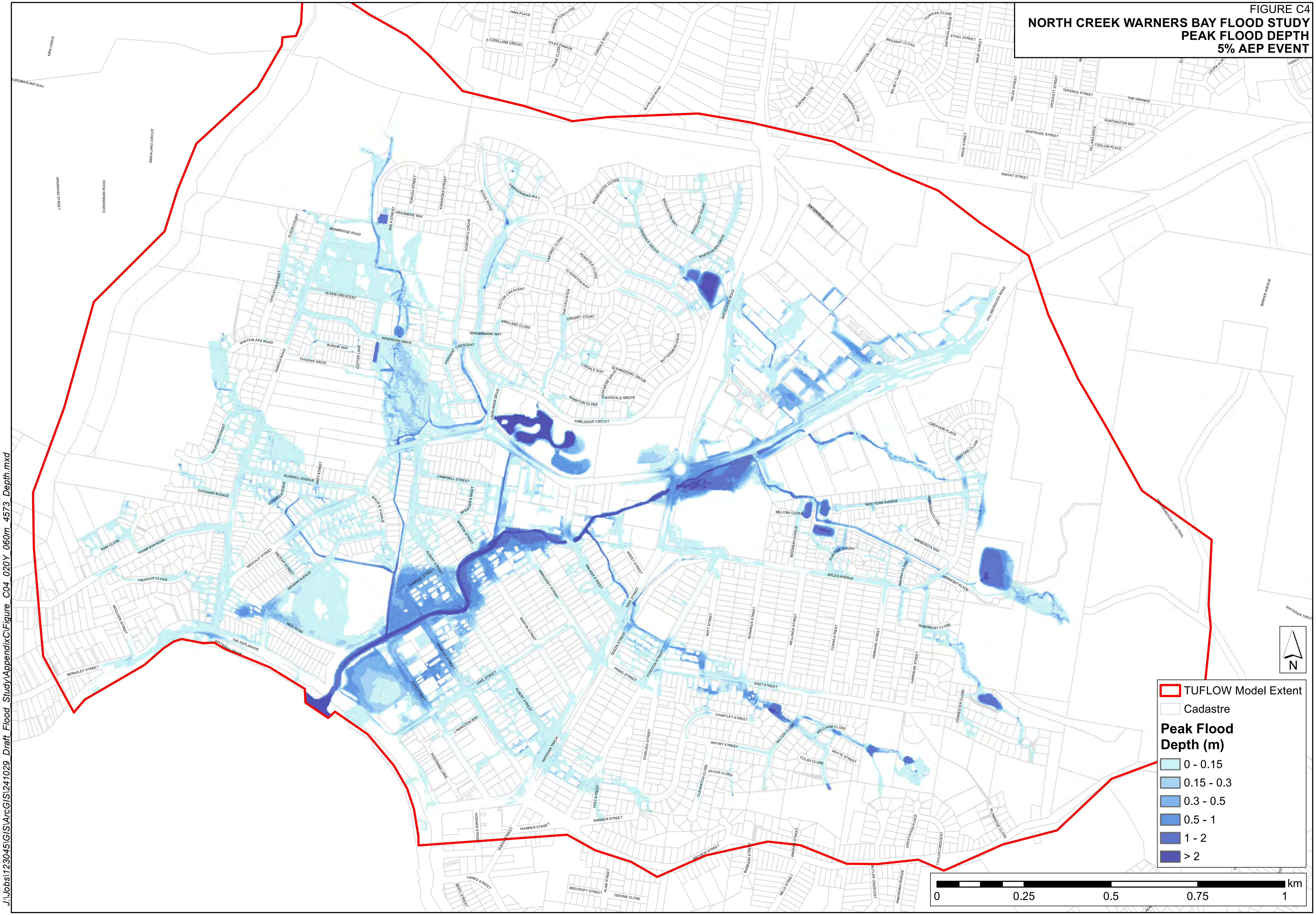
FIGURE C3

NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH
10% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029 Draft Flood Study\AppendixC\Figure C03 010Y 060m 4573 Depth.mxd



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C04_020Y_060m_4573_Depth.mxd

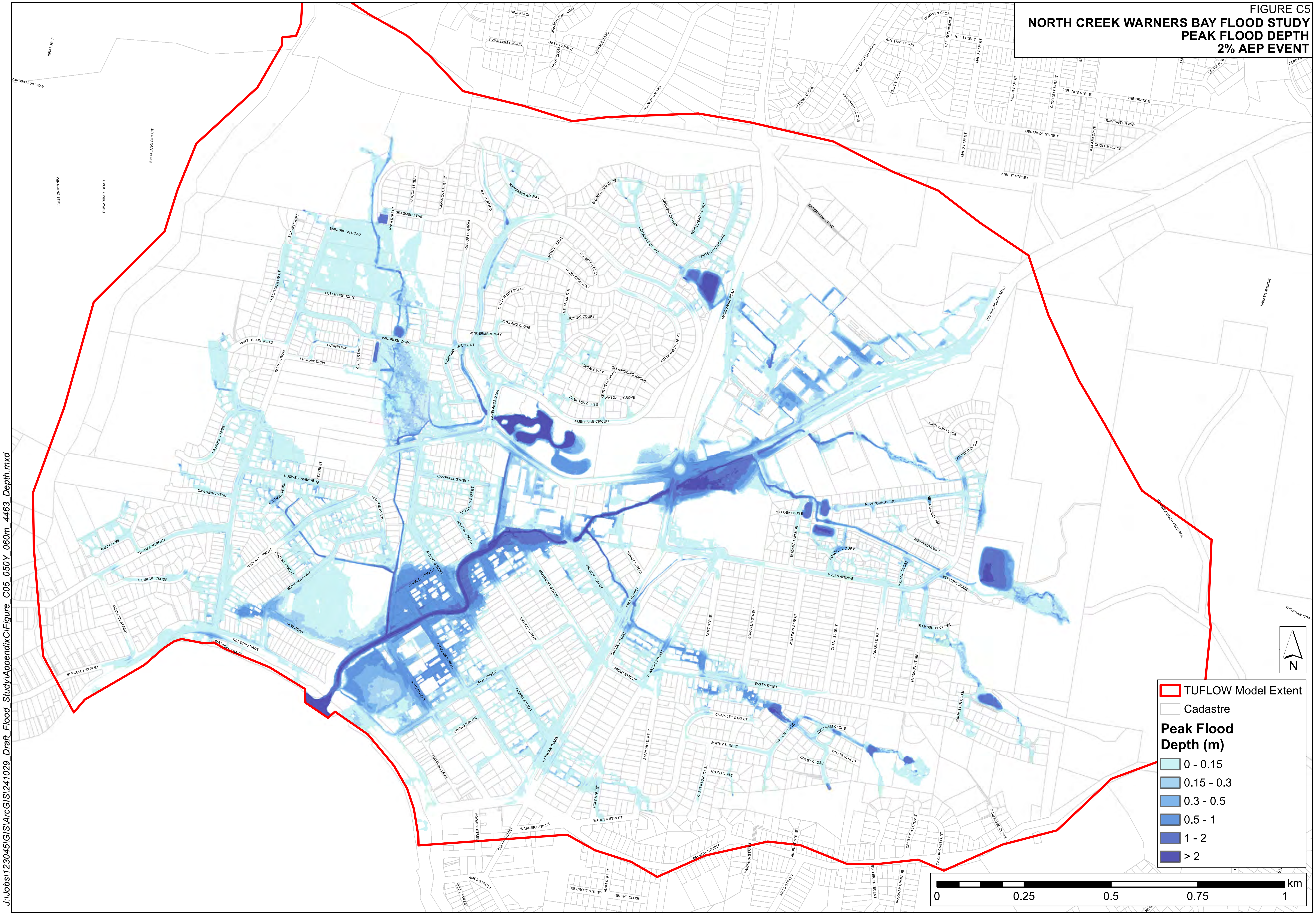
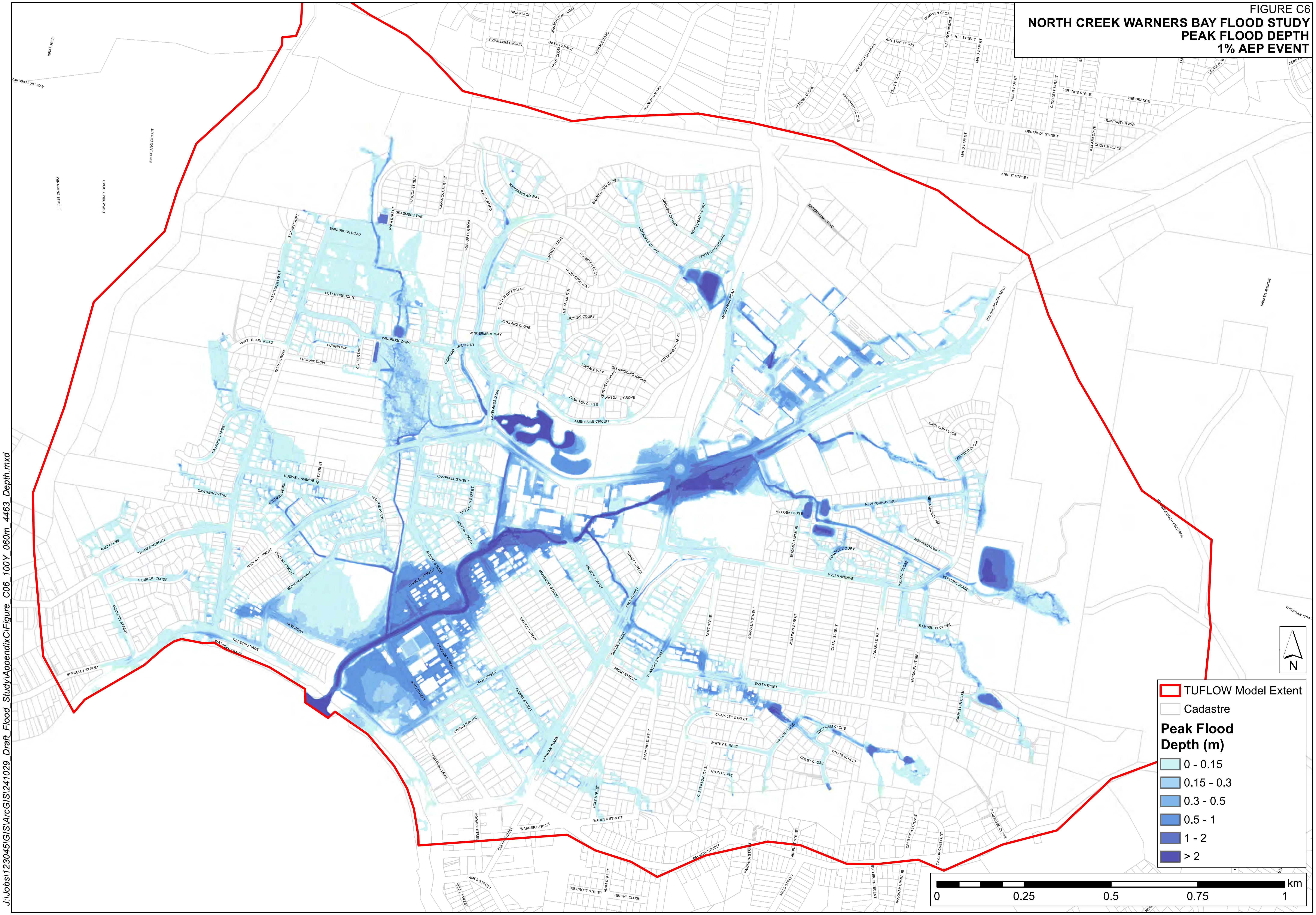


FIGURE C5
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH
2% AEP EVENT
TUFLOW Model Extent
Cadastre
Peak Flood Depth (m)
0 - 0.15
0.15 - 0.3
0.3 - 0.5
0.5 - 1
1 - 2
> 2
N
km
0
0.25
0.5
0.75
1
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C05_050Y_060m_4463_Depth.mxd

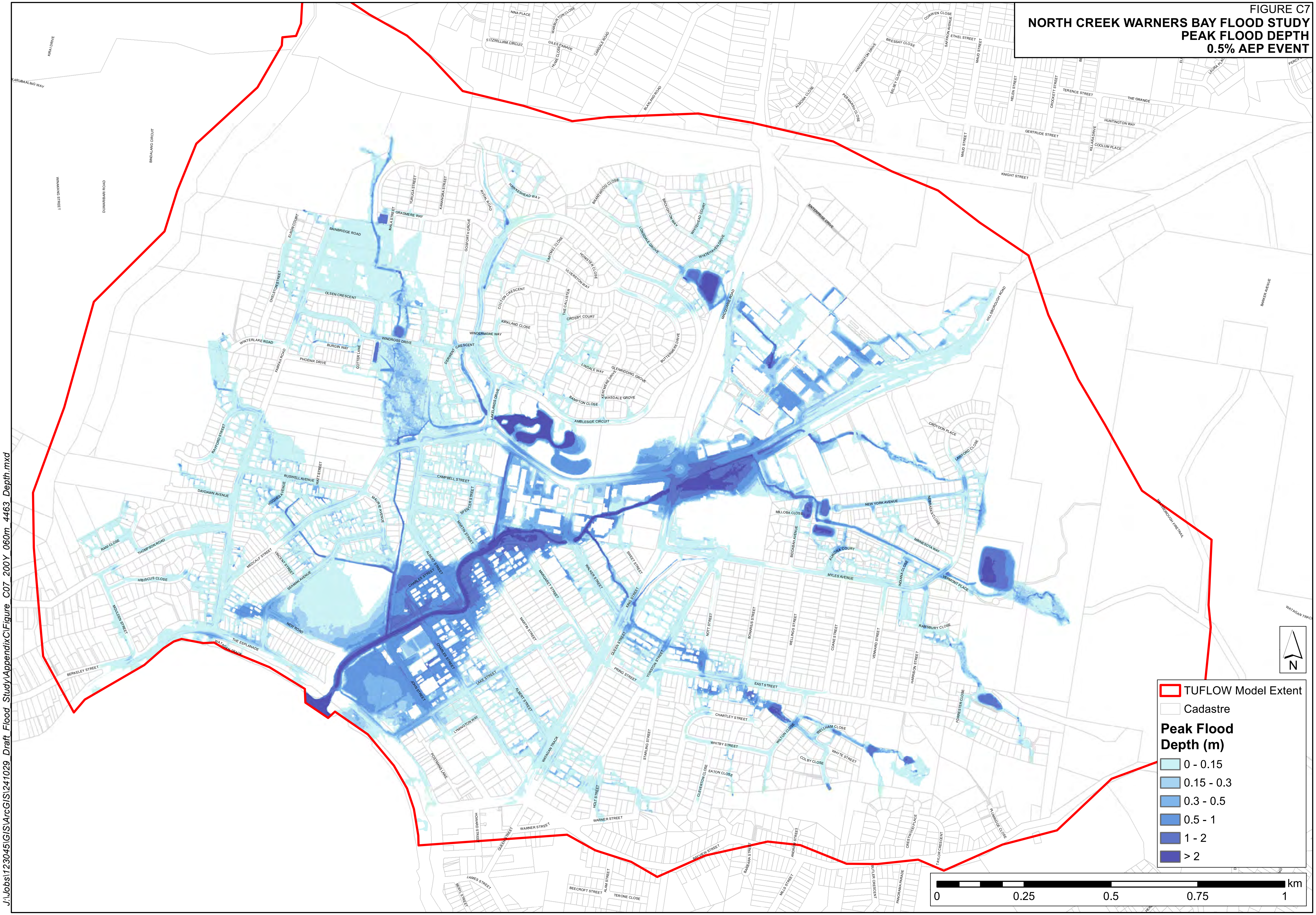
FIGURE C6
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH
1% AEP EVENT



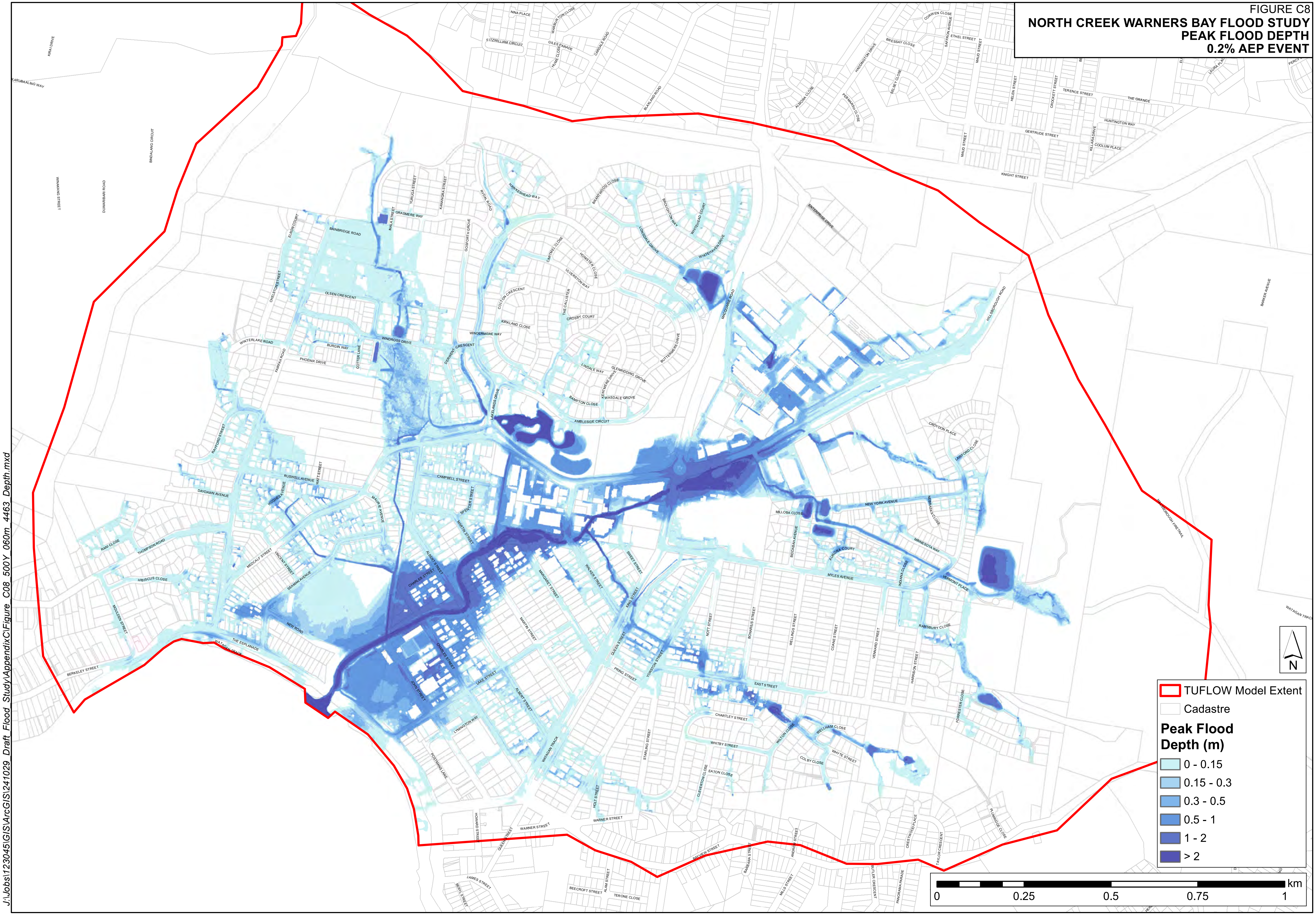
J:\Jobs\123045\GIS\ArcGIS\241029 Draft Flood Study\AppendixC\Figure C06 100Y 060m 4463 Depth.mxd

FIGURE C7

NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD DEPTH
0.5% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029 Draft Flood Study\AppendixC\Figure C07 200Y 060m 4463 Depth.mxd



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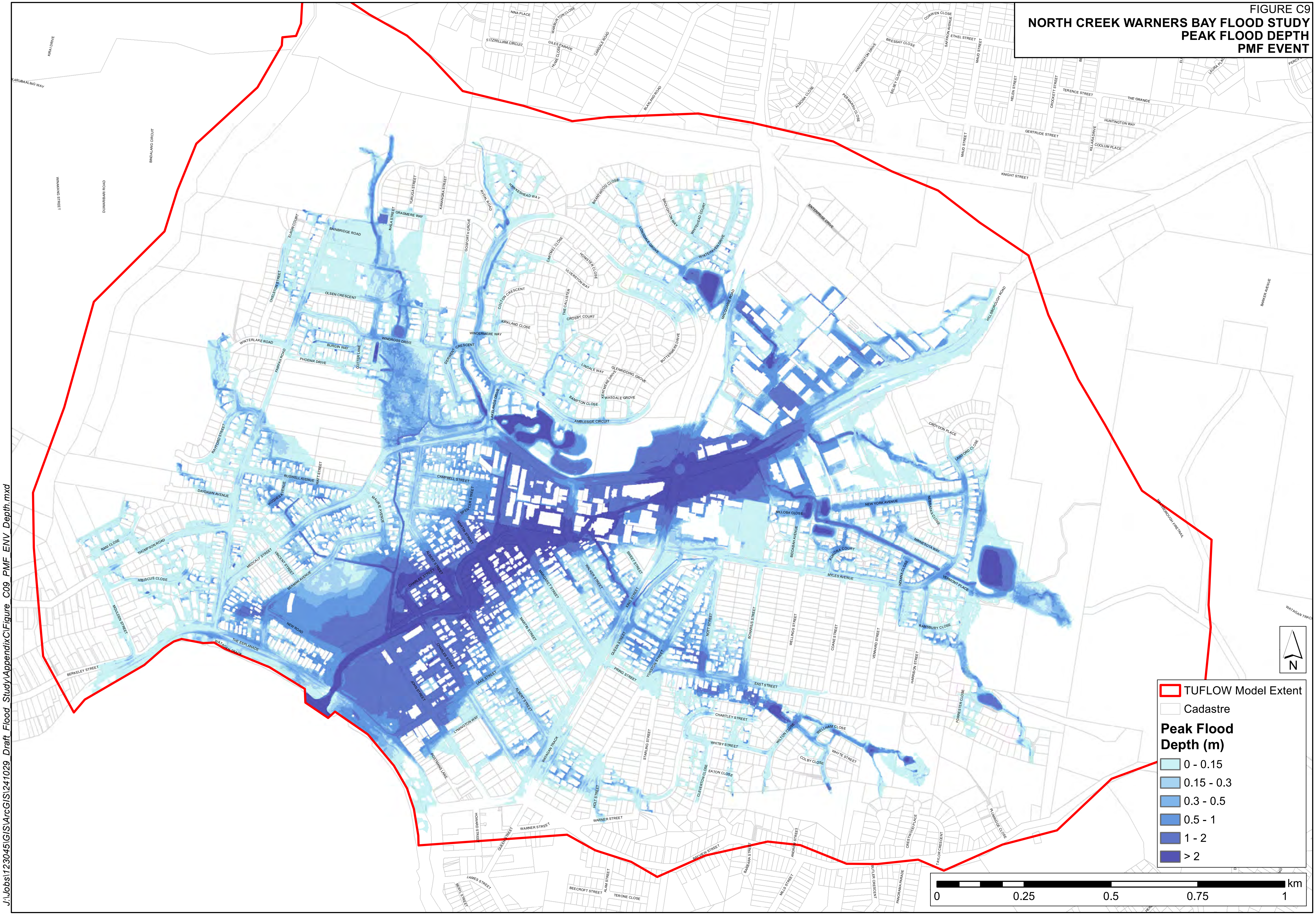


J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C09_PMF_ENV_Depth.mxd

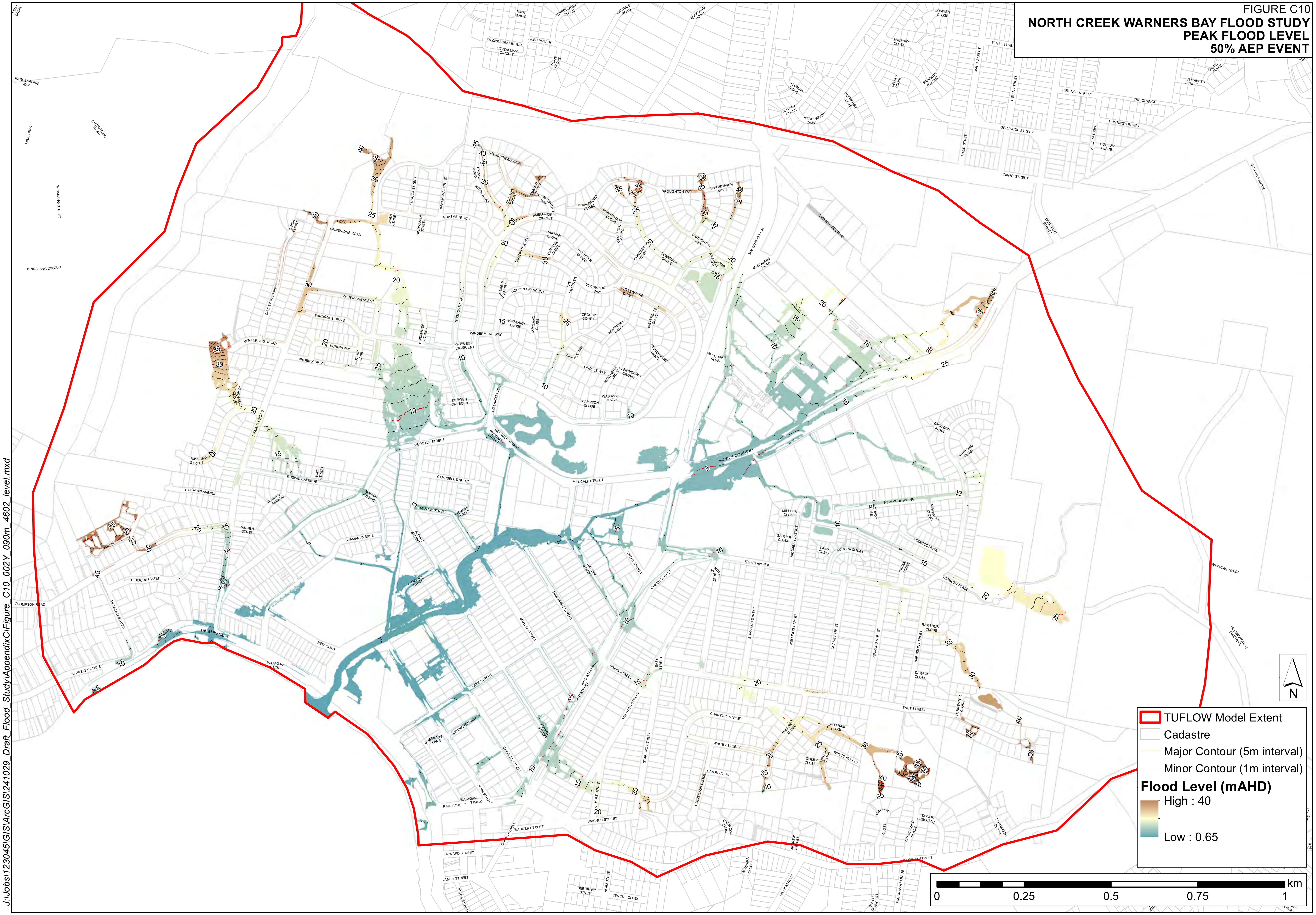
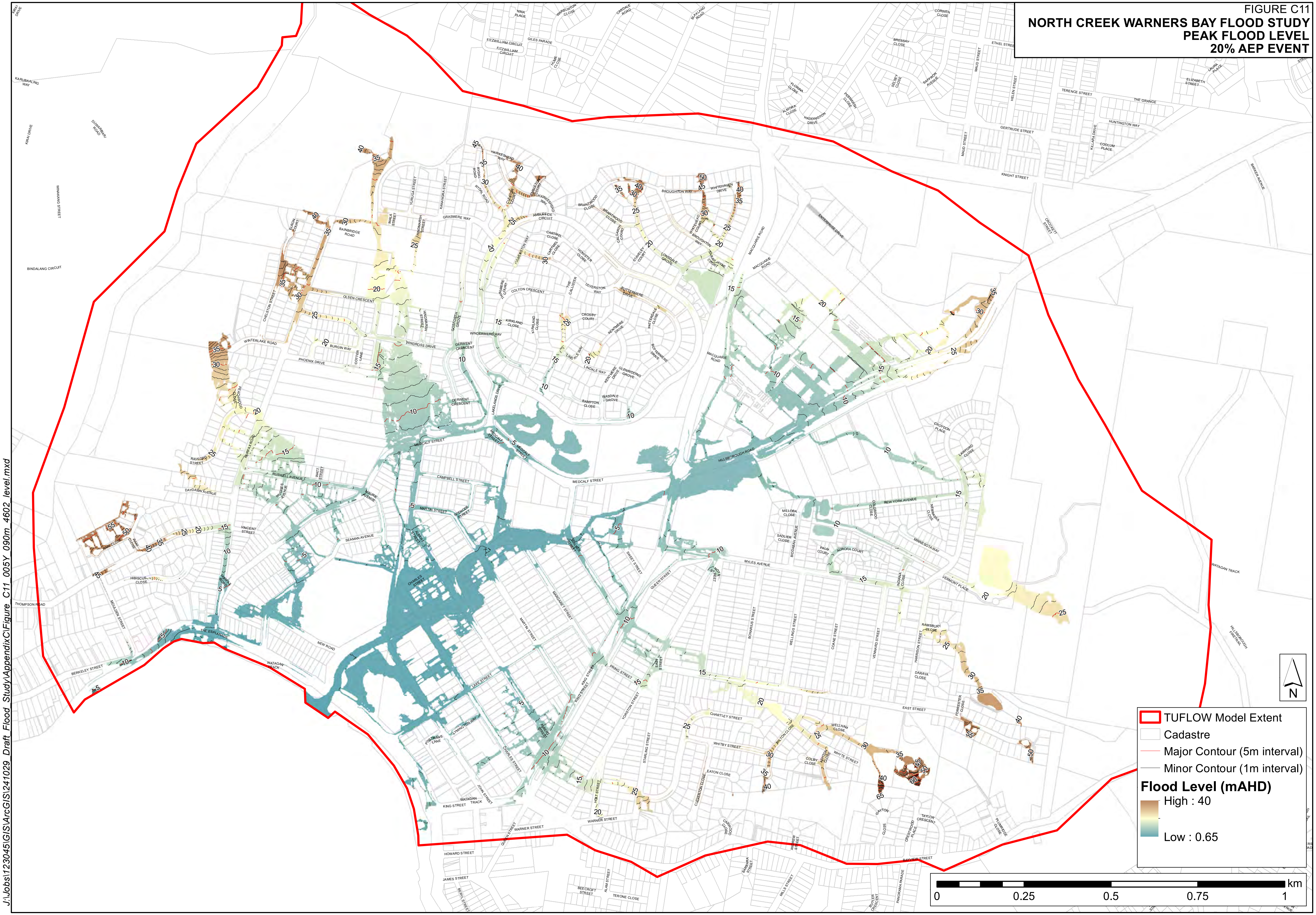


FIGURE C10
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
50% AEP EVENT
TUFLOW Model Extent
Cadastre
Major Contour (5m interval)
Minor Contour (1m interval)
Flood Level (mAHD)
High : 40
Low : 0.65
N
km
0
0.25
0.5
0.75
1
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C10_002Y_090m_4602_level.mxd

FIGURE C11
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
20% AEP EVENT



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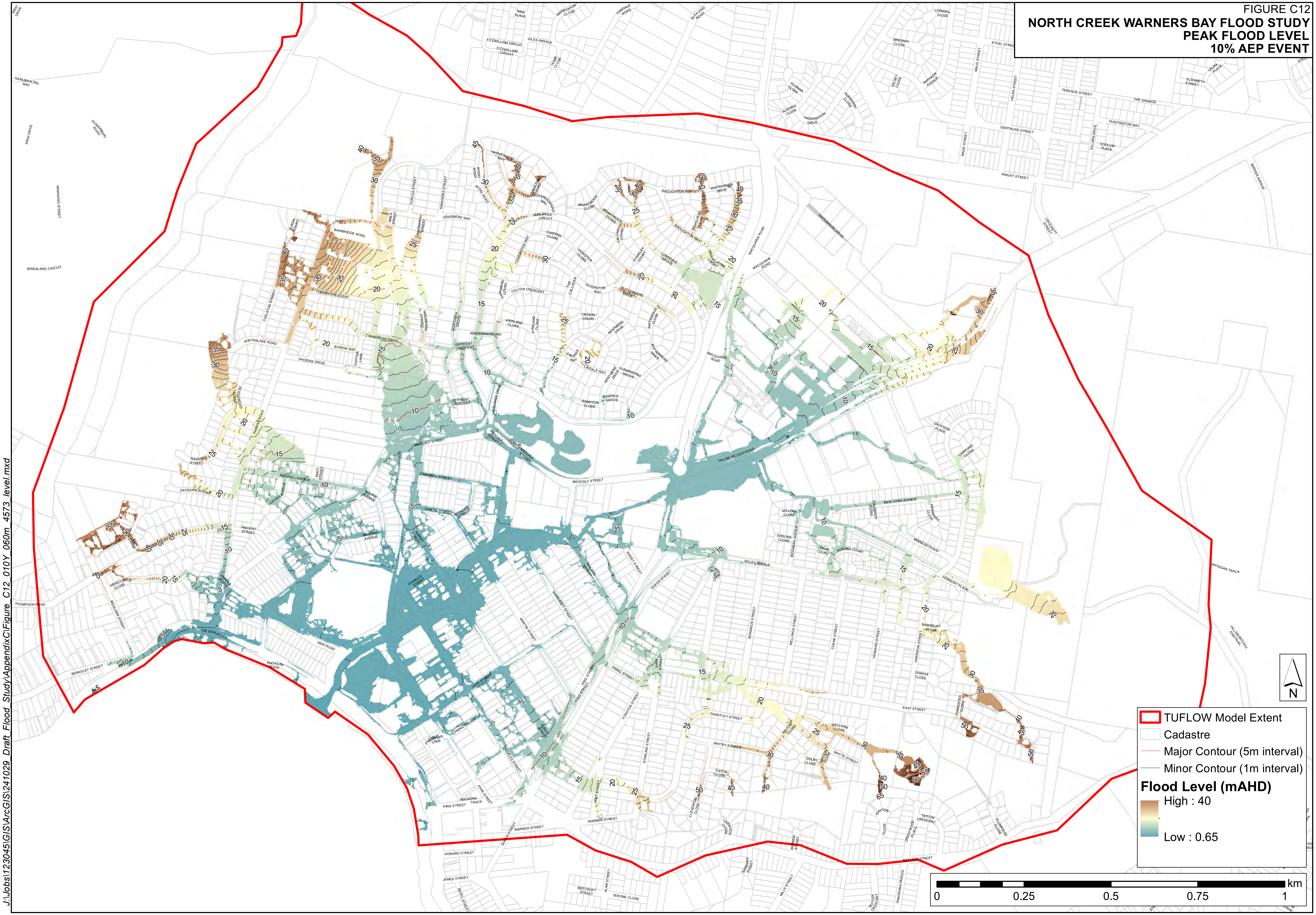
FIGURE C12
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
10% AEP EVENT
TUFLOW Model Extent
Cadastre
Major Contour (5m interval)
Minor Contour (1m interval)
Flood Level (mAHD)
High : 40
Low : 0.65
N
km
0
0.25
0.5
0.75
1
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C12_010Y_060m_4573_level.mxd

FIGURE C13
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
5% AEP EVENT

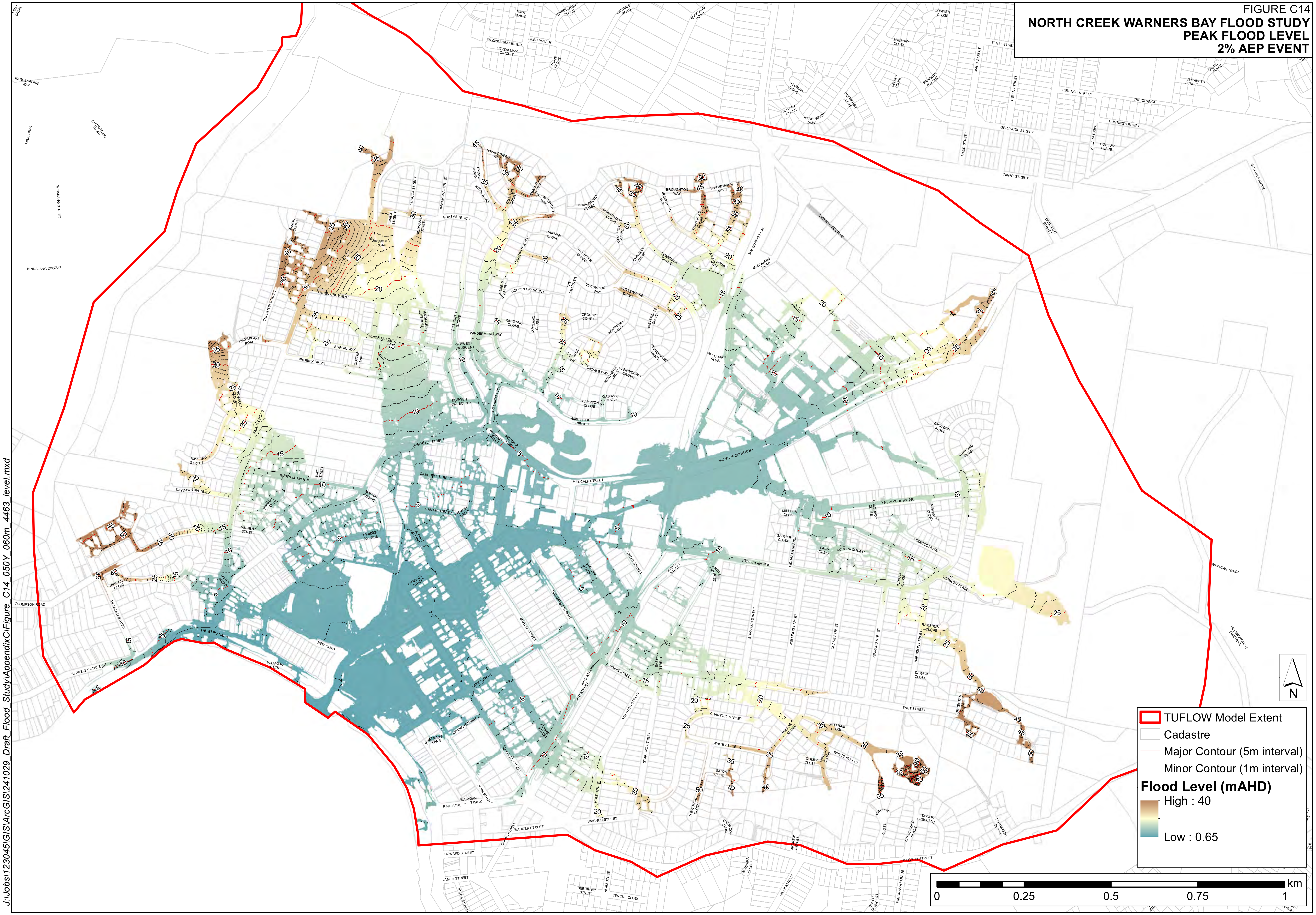
FIGURE C14
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
2% AEP EVENT
TUFLOW Model Extent
Cadastre
Major Contour (5m interval)
Minor Contour (1m interval)
Flood Level (mAHD)
High : 40
Low : 0.65
N
km
0
0.25
0.5
0.75
1
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C14_050Y_060m_4463_level.mxd

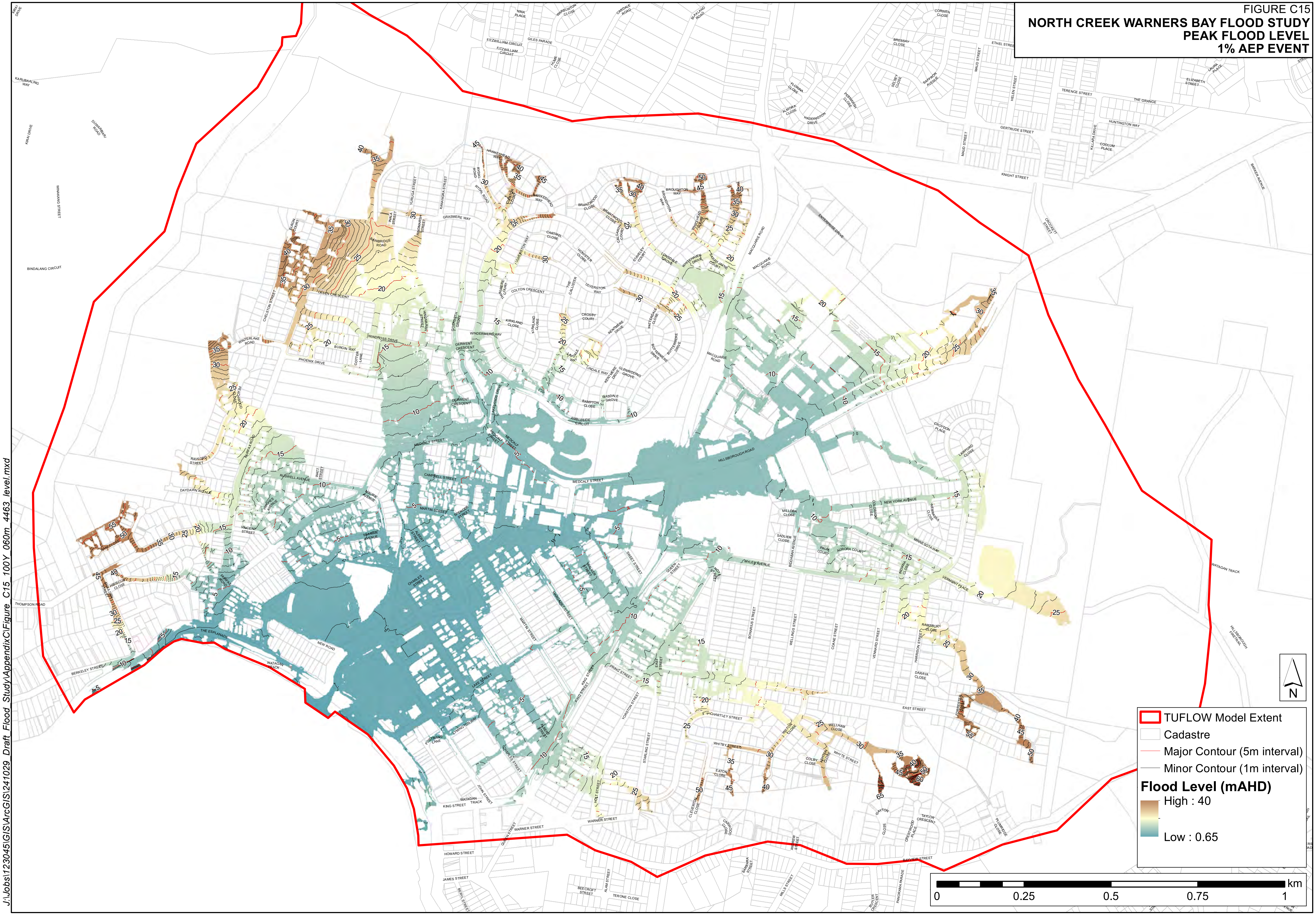
FIGURE C15
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
1% AEP EVENT
TUFLOW Model Extent
Cadastre
Major Contour (5m interval)
Minor Contour (1m interval)
Flood Level (mAHD)
High : 40
Low : 0.65
N
0
0.25
0.5
0.75
1
km
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C15_100Y_060m_4463_level.mxd

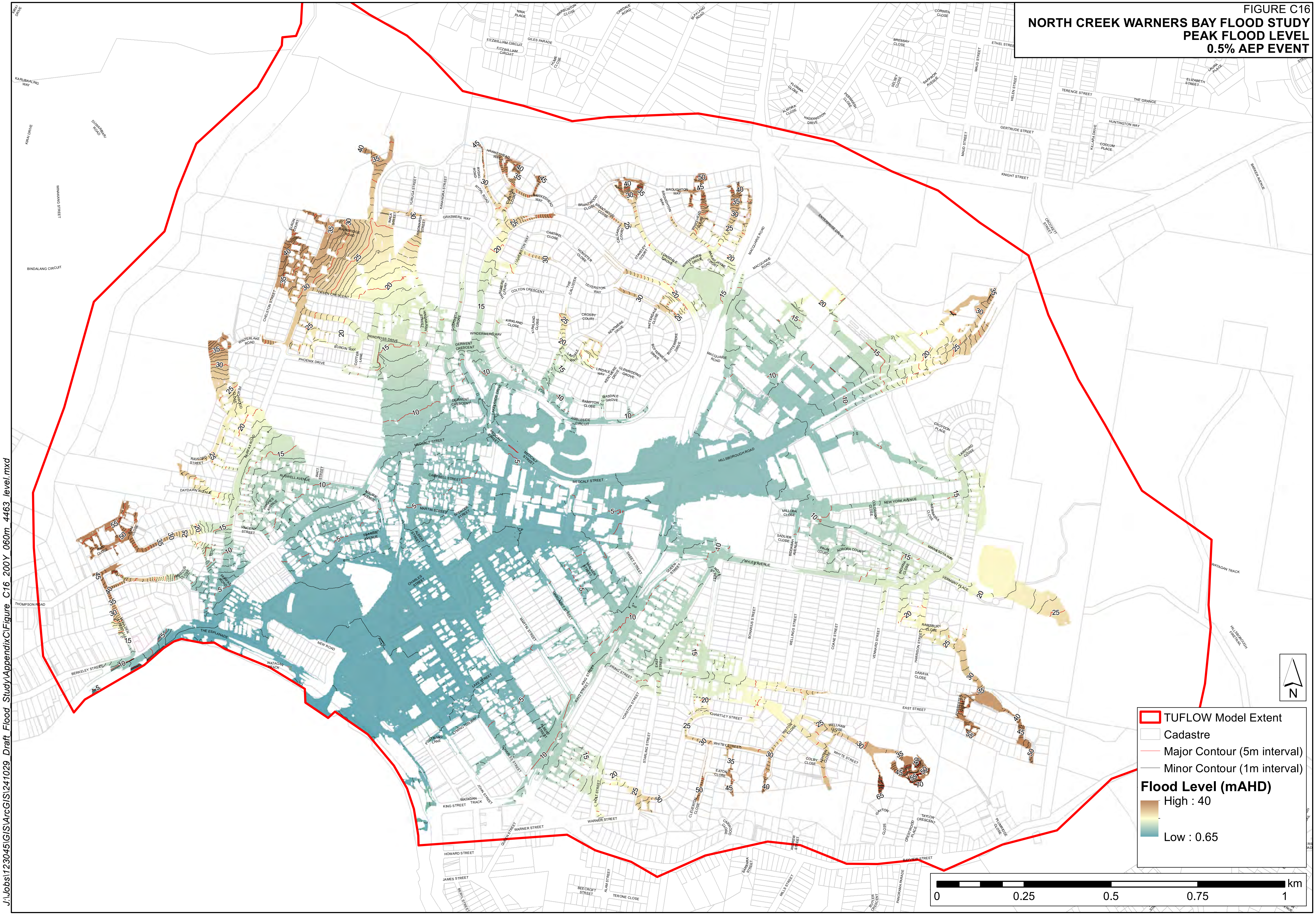

FIGURE C16
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
0.5% AEP EVENT
TUFLOW Model Extent
Cadastre
Major Contour (5m interval)
Minor Contour (1m interval)
Flood Level (mAHD)
High : 40
Low : 0.65
N
0 0.25 0.5 0.75 1 km
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C16_200Y_060m_4463_level.mxd

FIGURE C17

NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
0.2% AEP EVENT

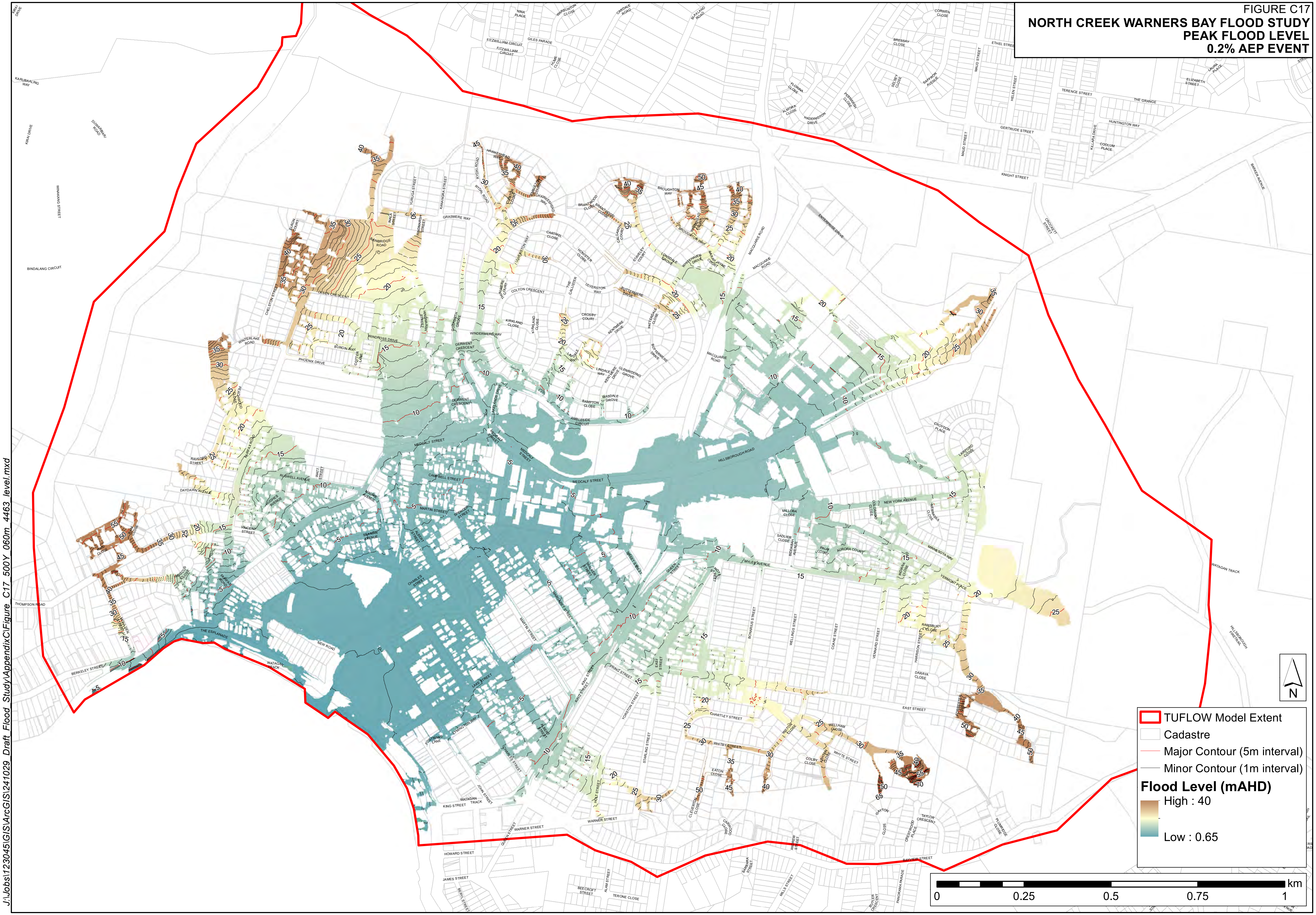

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FIGURE C18

NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK FLOOD LEVEL
PMF EVENT

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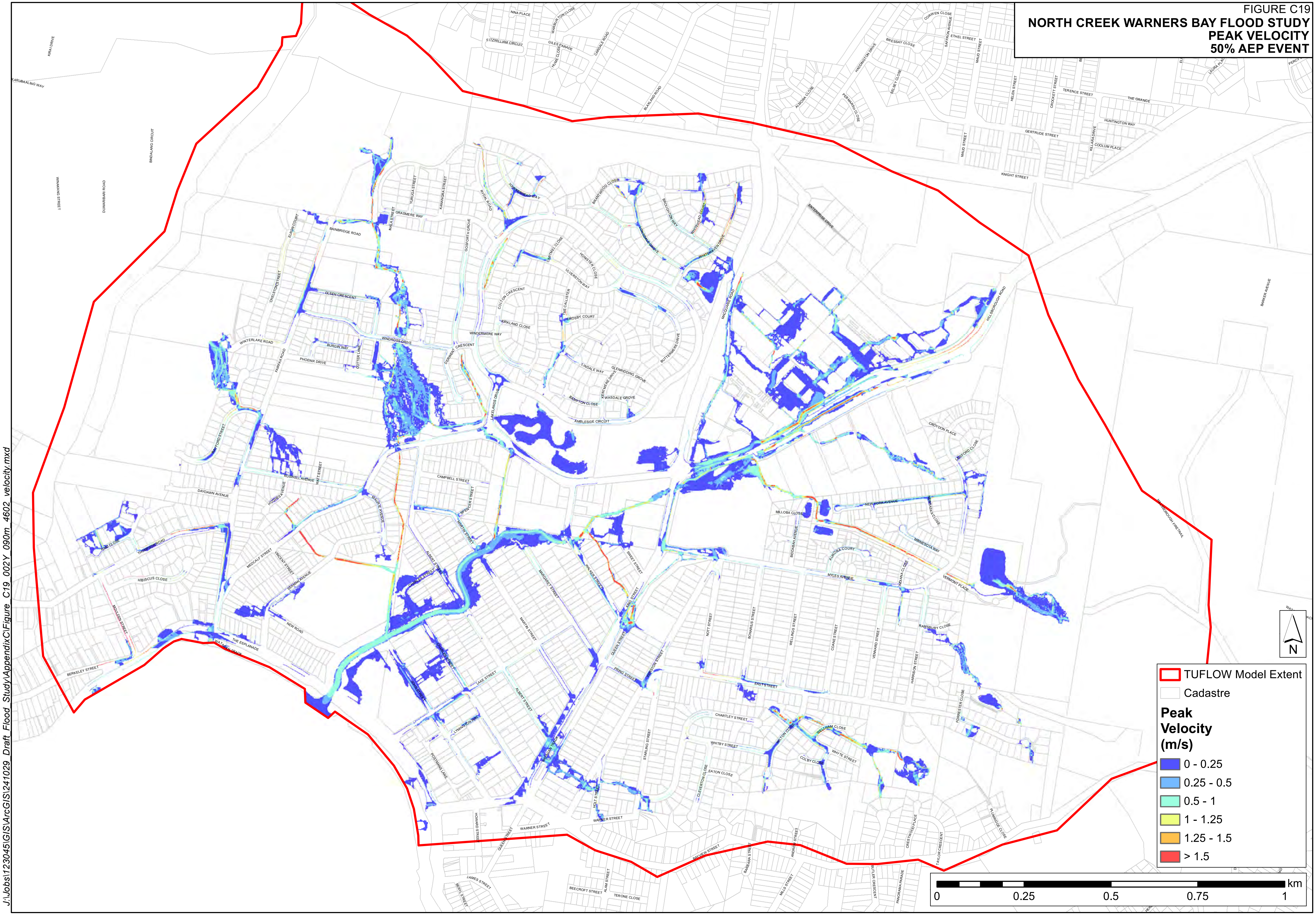
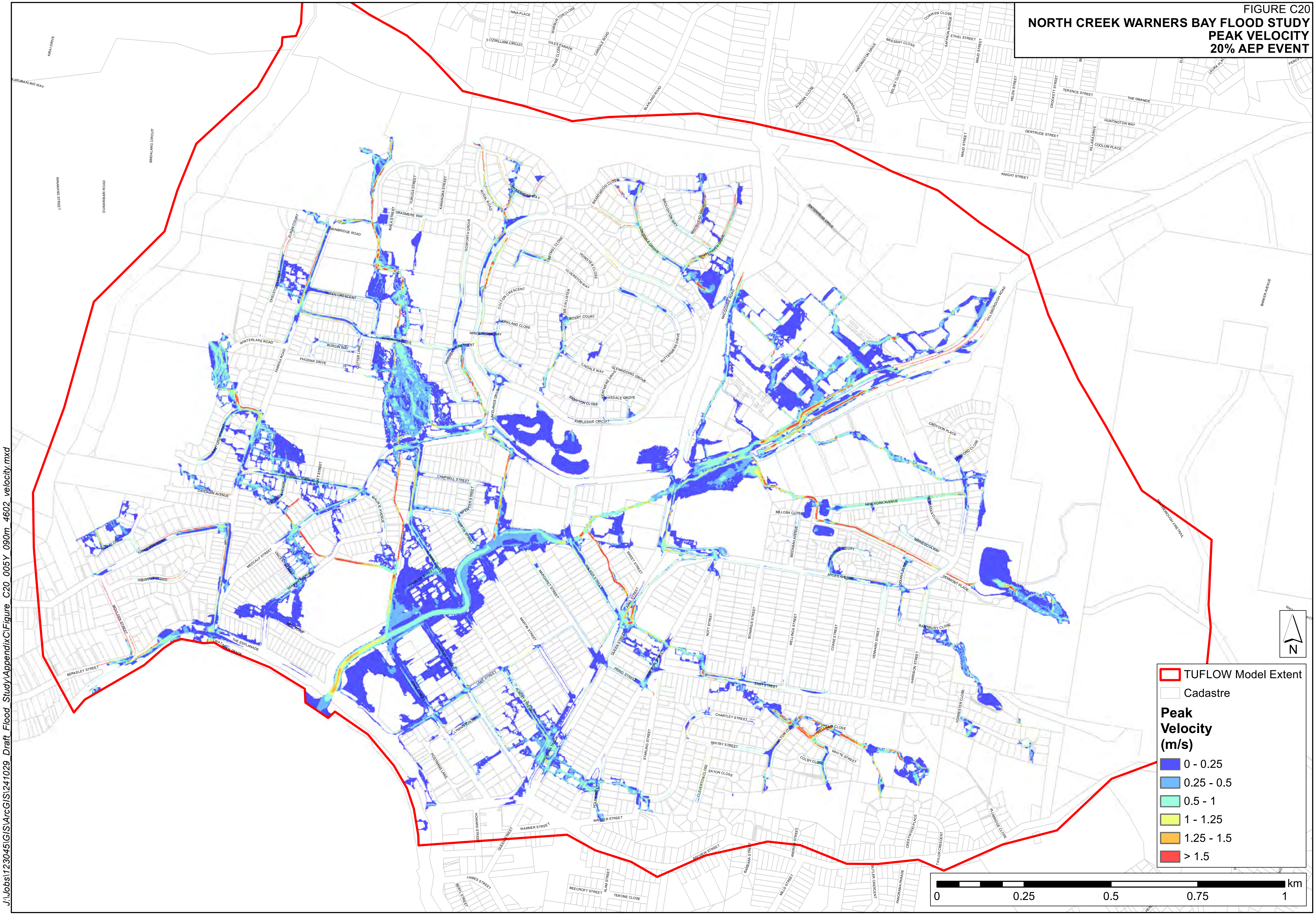
FIGURE C19
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK VELOCITY
50% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C19_002Y_090m_4602_velocity.mxd



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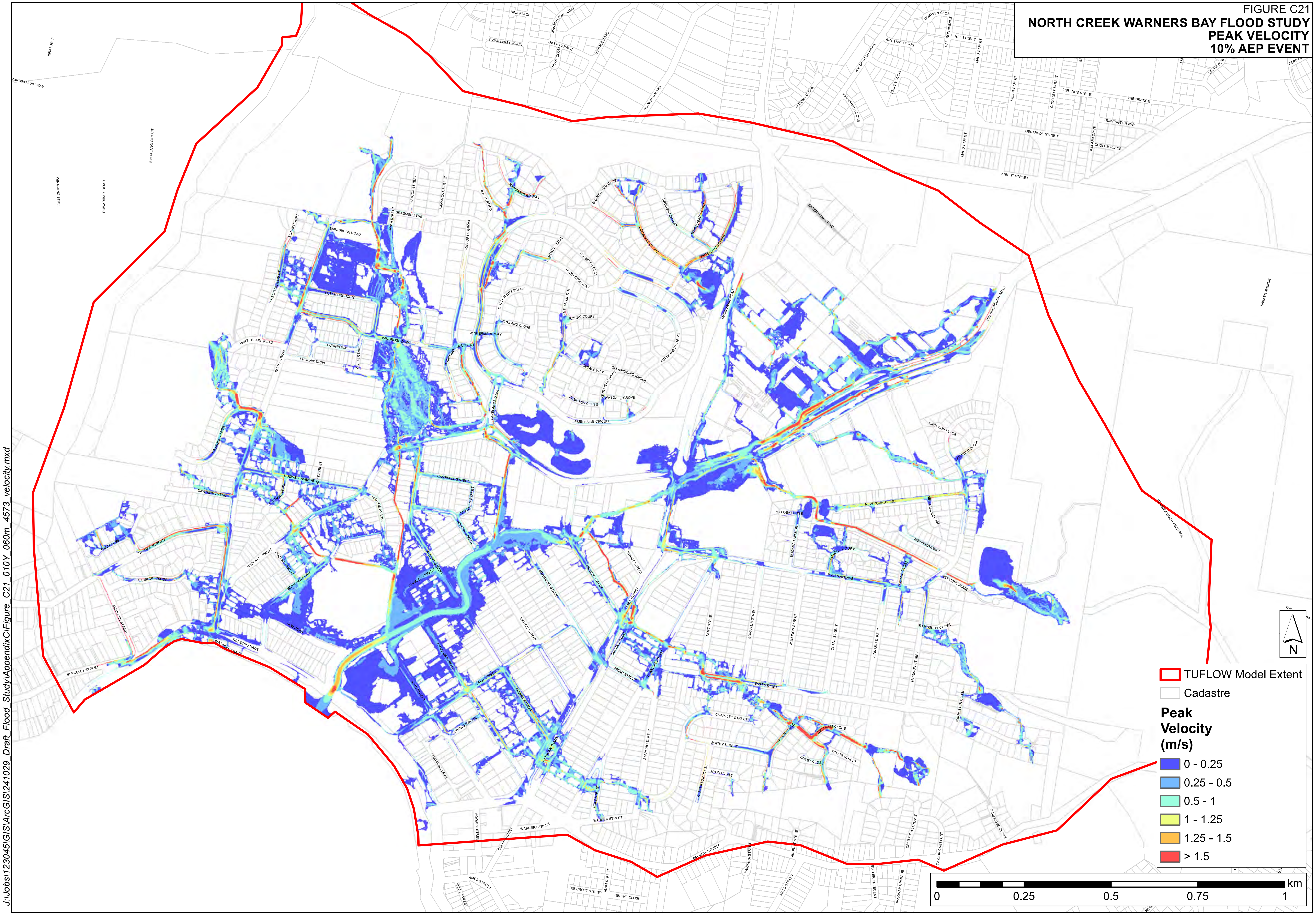
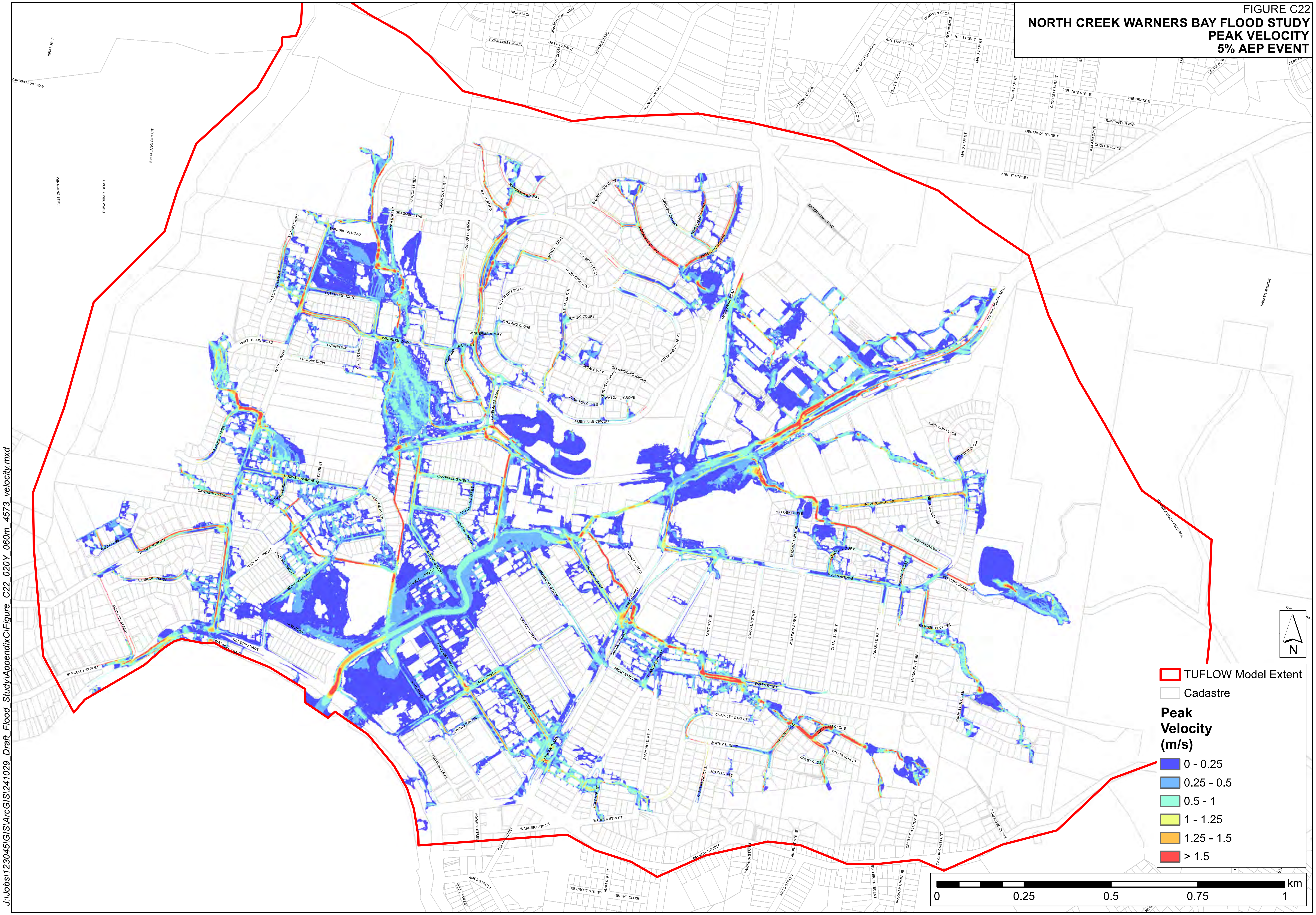


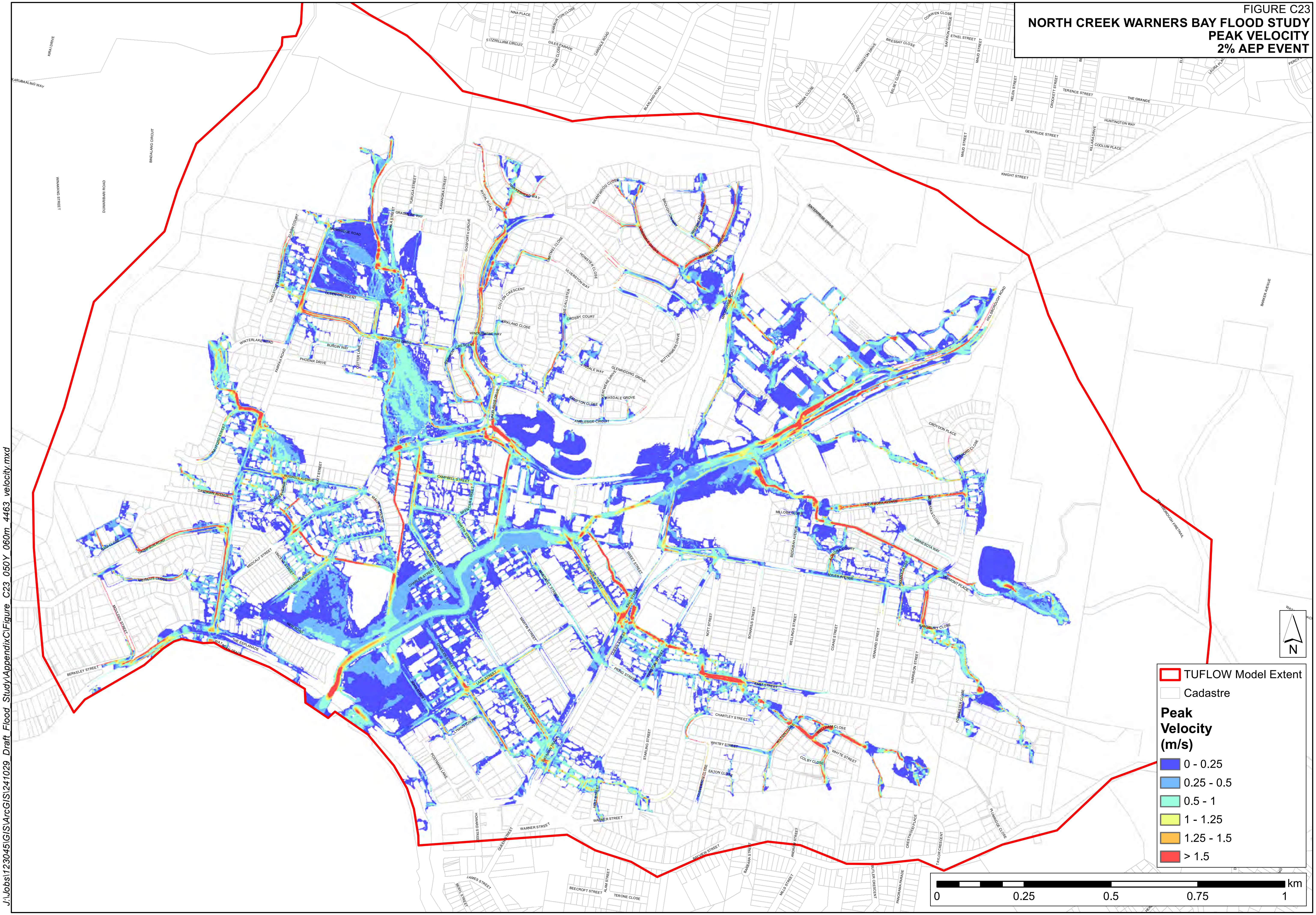
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C21_010Y_060m_4573_velocity.mxd

FIGURE C22
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK VELOCITY
5% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C22_020Y_060m_4573_velocity.mxd

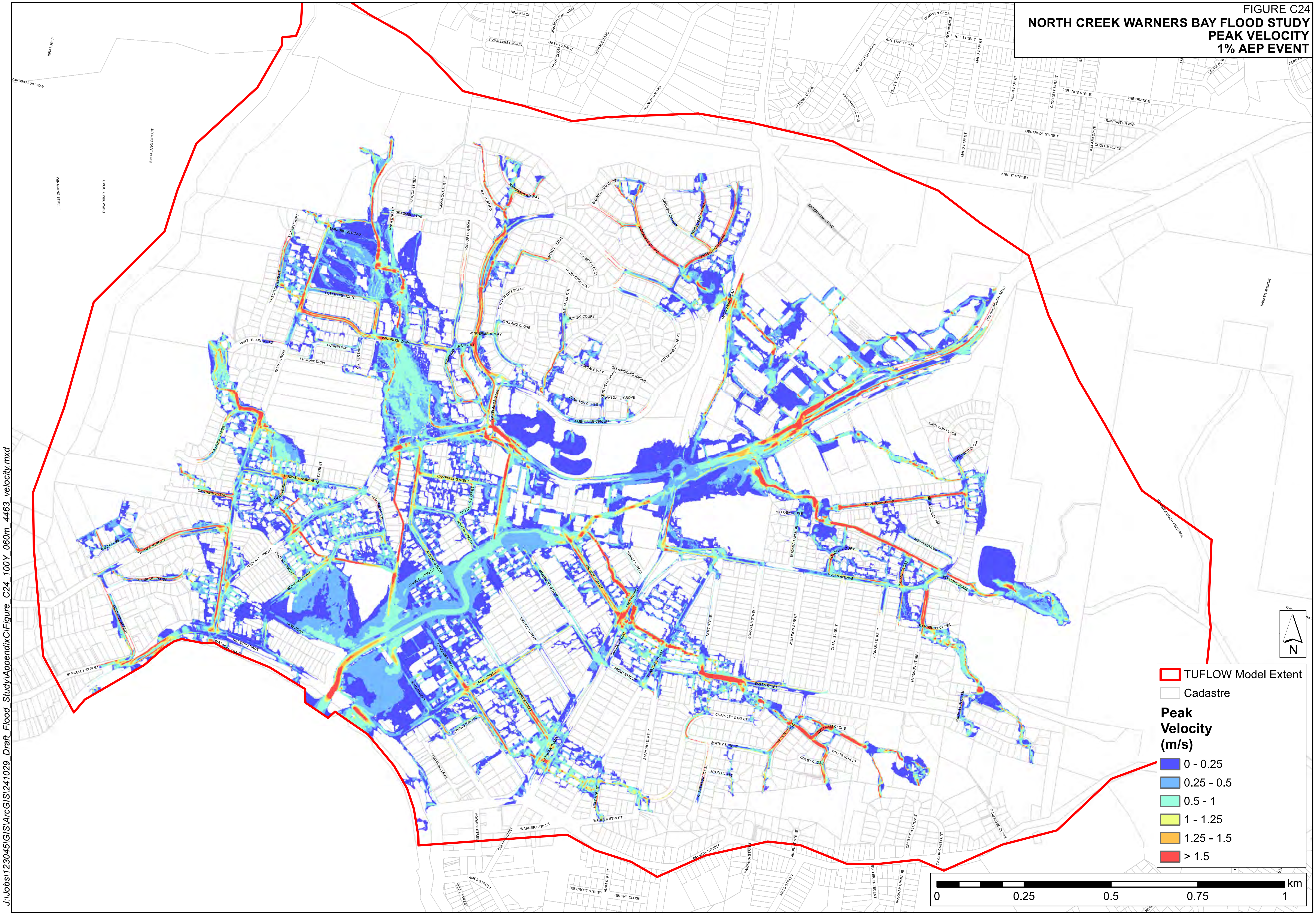
FIGURE C23
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK VELOCITY
2% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C23_050Y_060m_4463_velocity.mxd

FIGURE C24

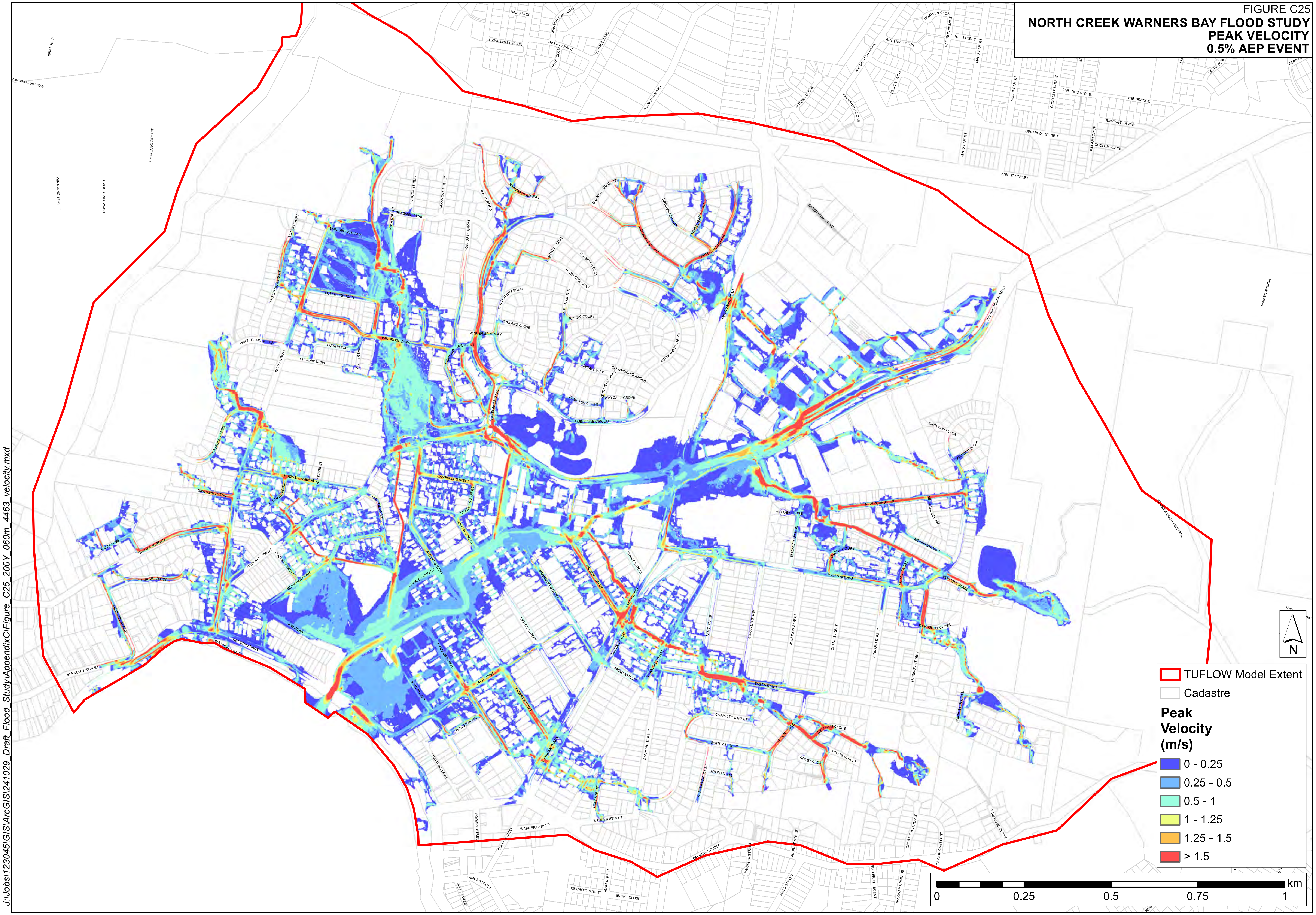
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK VELOCITY
1% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C24_100Y_060m_4463_velocity.mxd

FIGURE C25

NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK VELOCITY
0.5% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C25_200Y_060m_4463_velocity.mxd

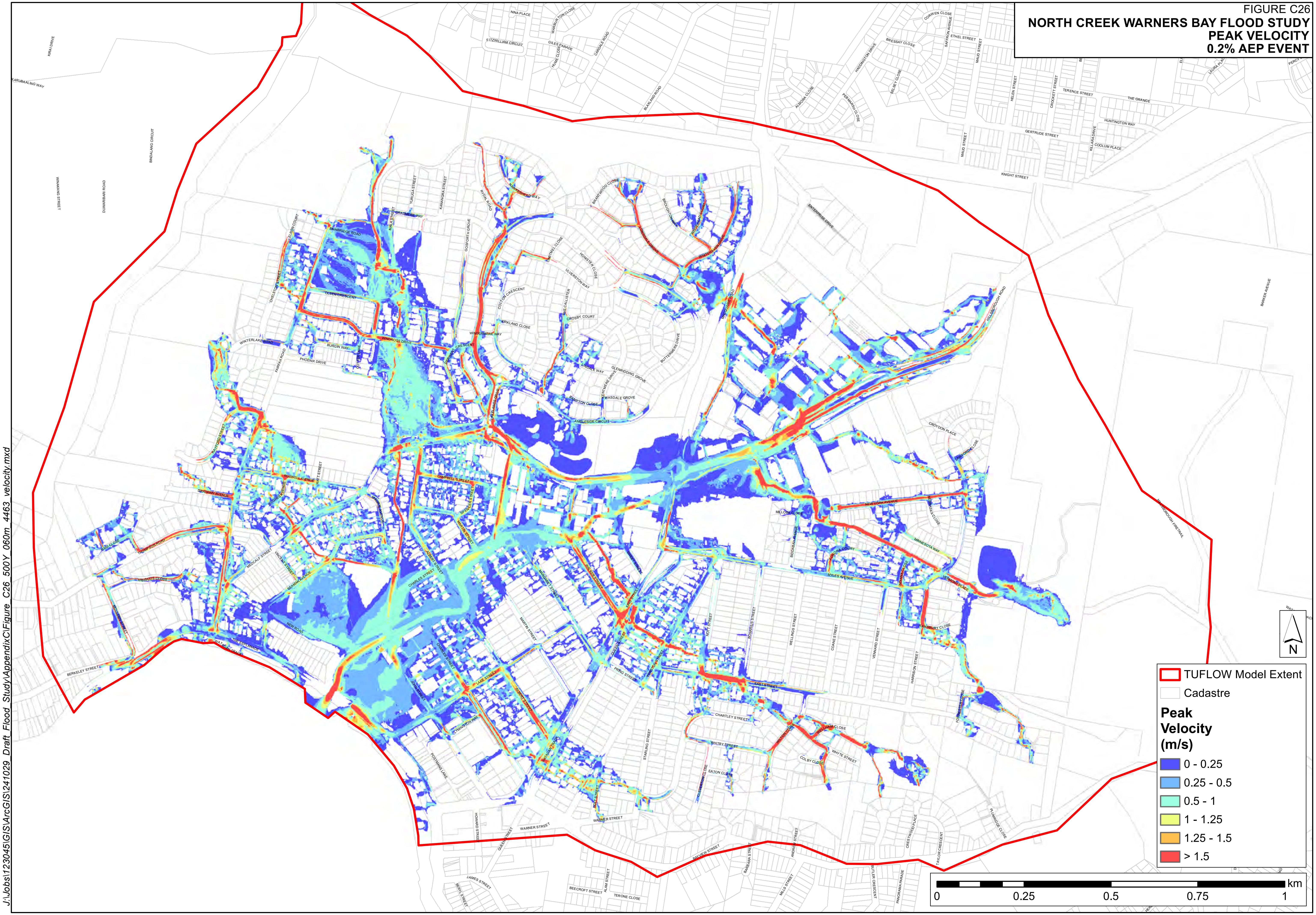
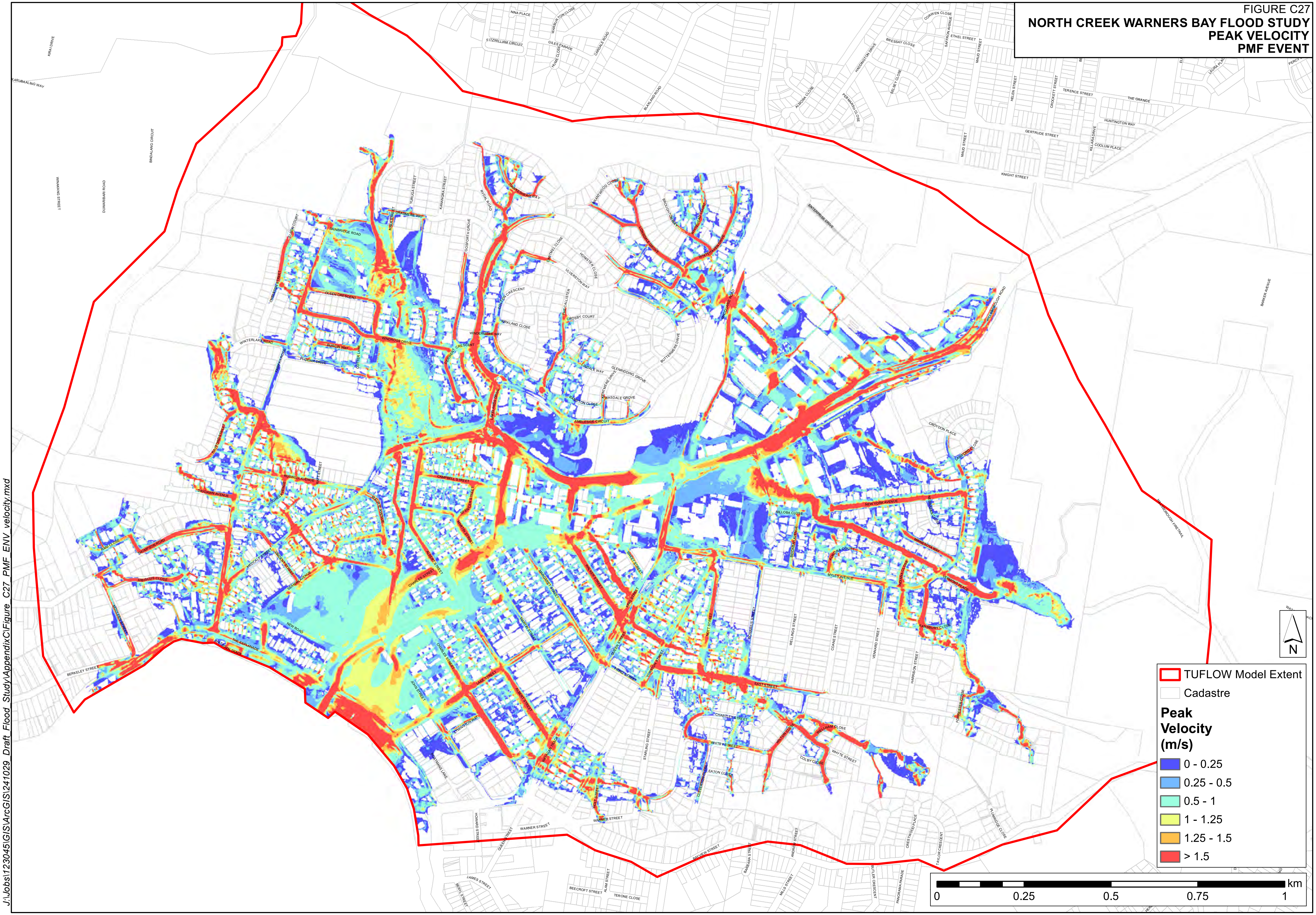
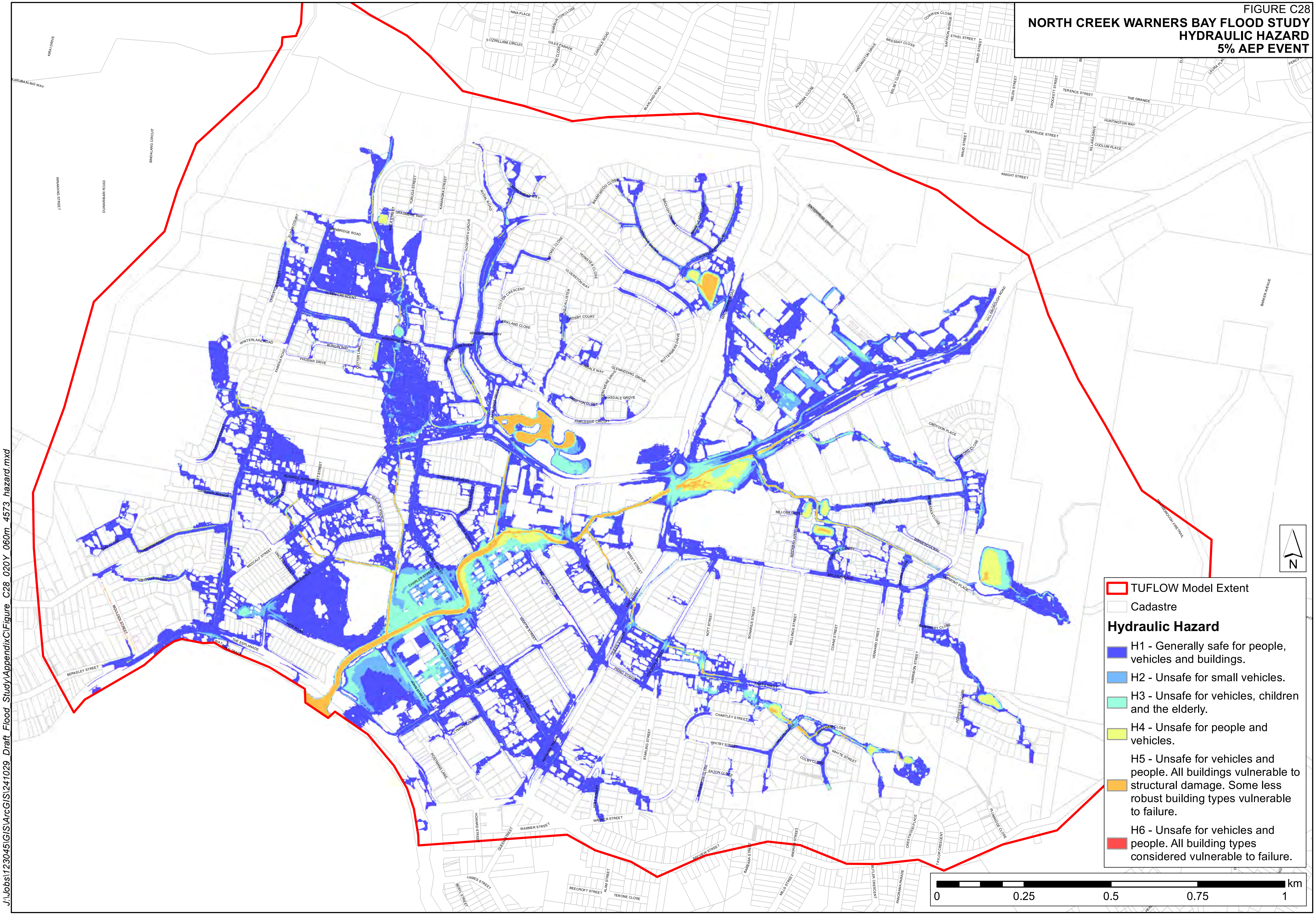


FIGURE C26
NORTH CREEK WARNERS BAY FLOOD STUDY
PEAK VELOCITY
0.2% AEP EVENT
TUFLOW Model Extent
Cadastre
Peak Velocity (m/s)
0 - 0.25
0.25 - 0.5
0.5 - 1
1 - 1.25
1.25 - 1.5
> 1.5
N
0
0.25
0.5
0.75
1
km
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C26_500Y_060m_4463_velocity.mxd



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C27_PMF_ENV_velocity.mxd



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C28_020Y_060m_4573_hazard.mxd

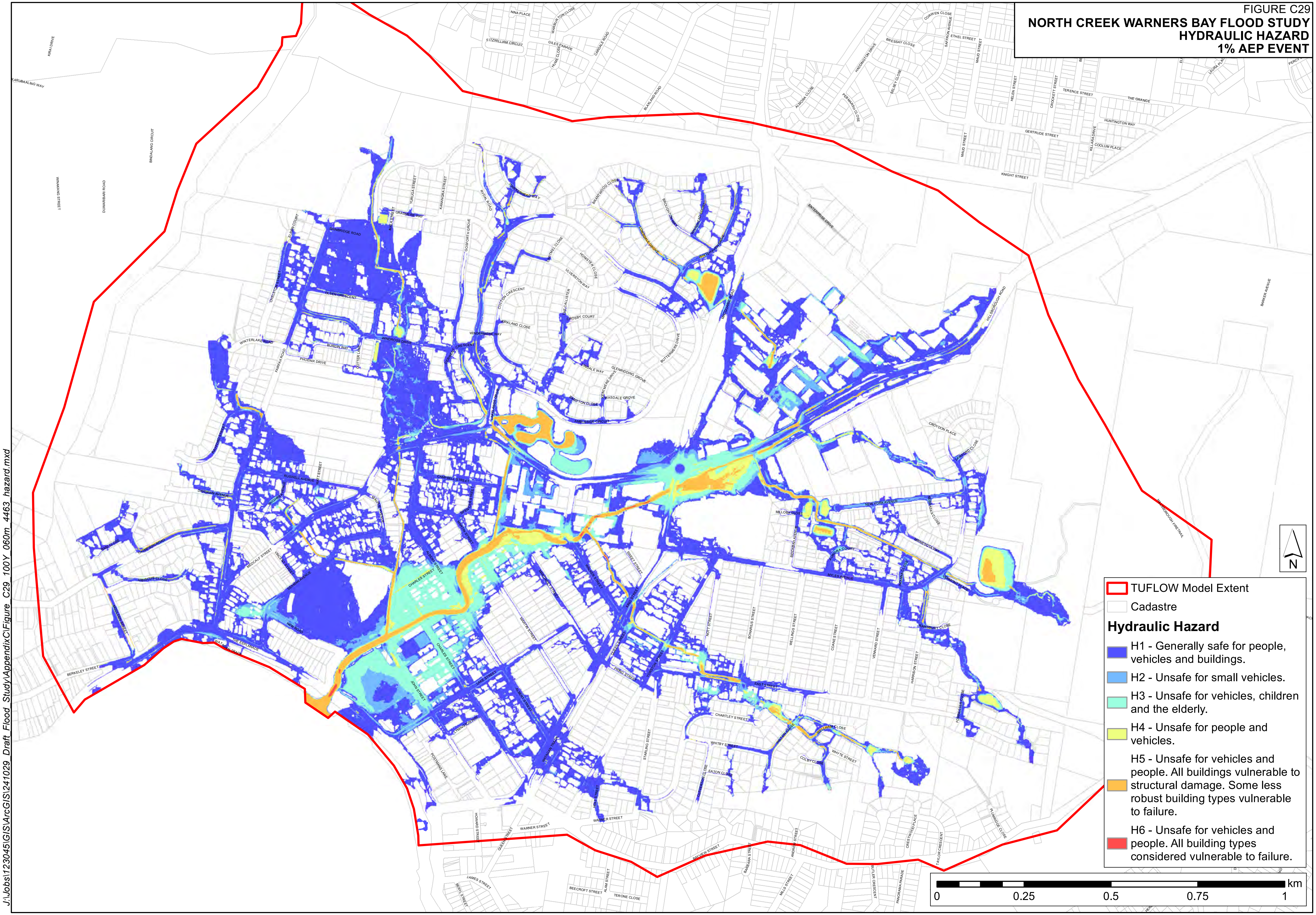
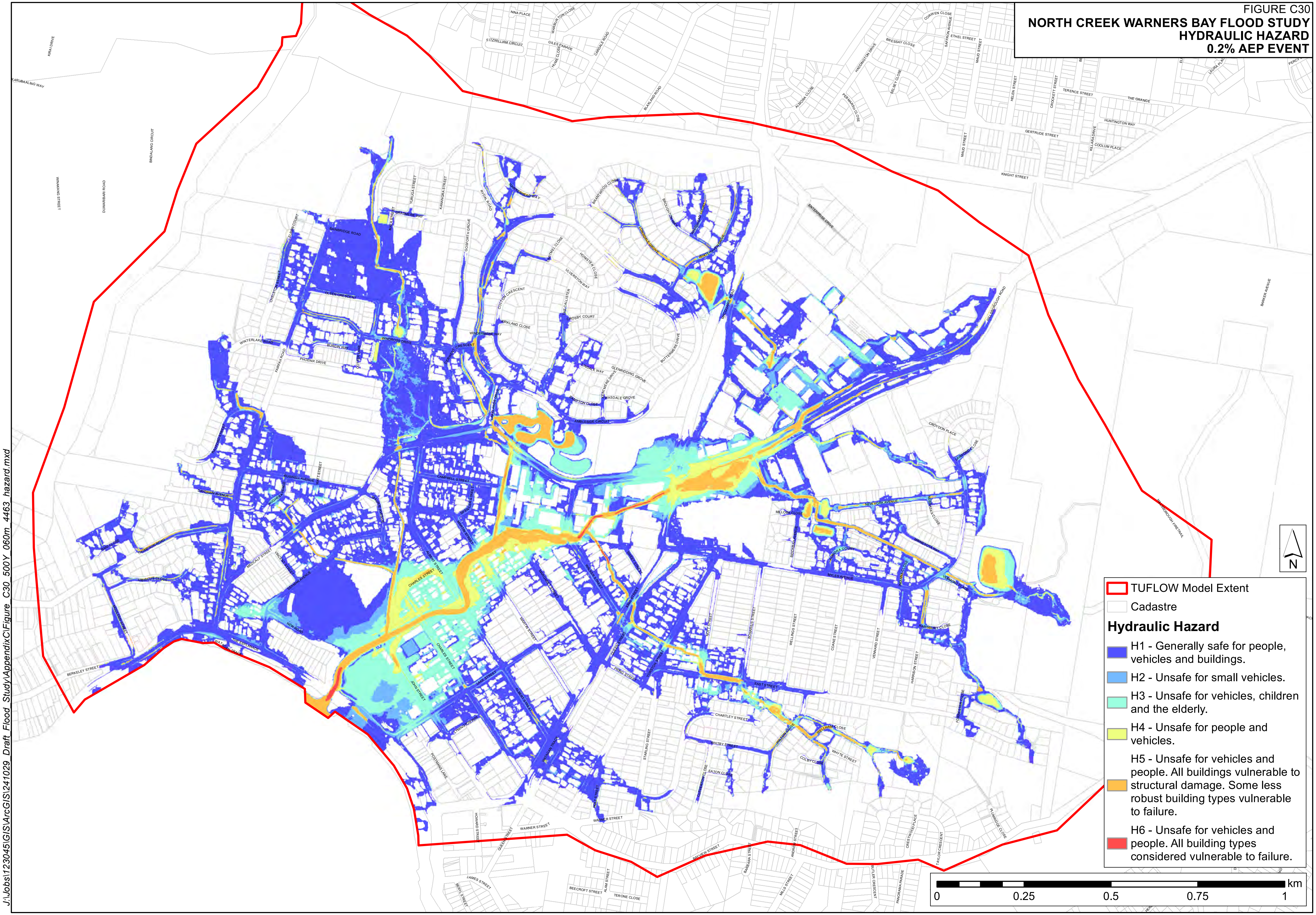


FIGURE C29
NORTH CREEK WARNERS BAY FLOOD STUDY
HYDRAULIC HAZARD
1% AEP EVENT
TUFLOW Model Extent
Cadastre
Hydraulic Hazard
H1 - Generally safe for people, vehicles and buildings.
H2 - Unsafe for small vehicles.
H3 - Unsafe for vehicles, children and the elderly.
H4 - Unsafe for people and vehicles.
H5 - Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust building types vulnerable to failure.
H6 - Unsafe for vehicles and people. All building types considered vulnerable to failure.
N
0 0.25 0.5 0.75 1 km
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C29_100Y_060m_4463_hazard.mxd

FIGURE C30

NORTH CREEK WARNERS BAY FLOOD STUDY
HYDRAULIC HAZARD
0.2% AEP EVENT



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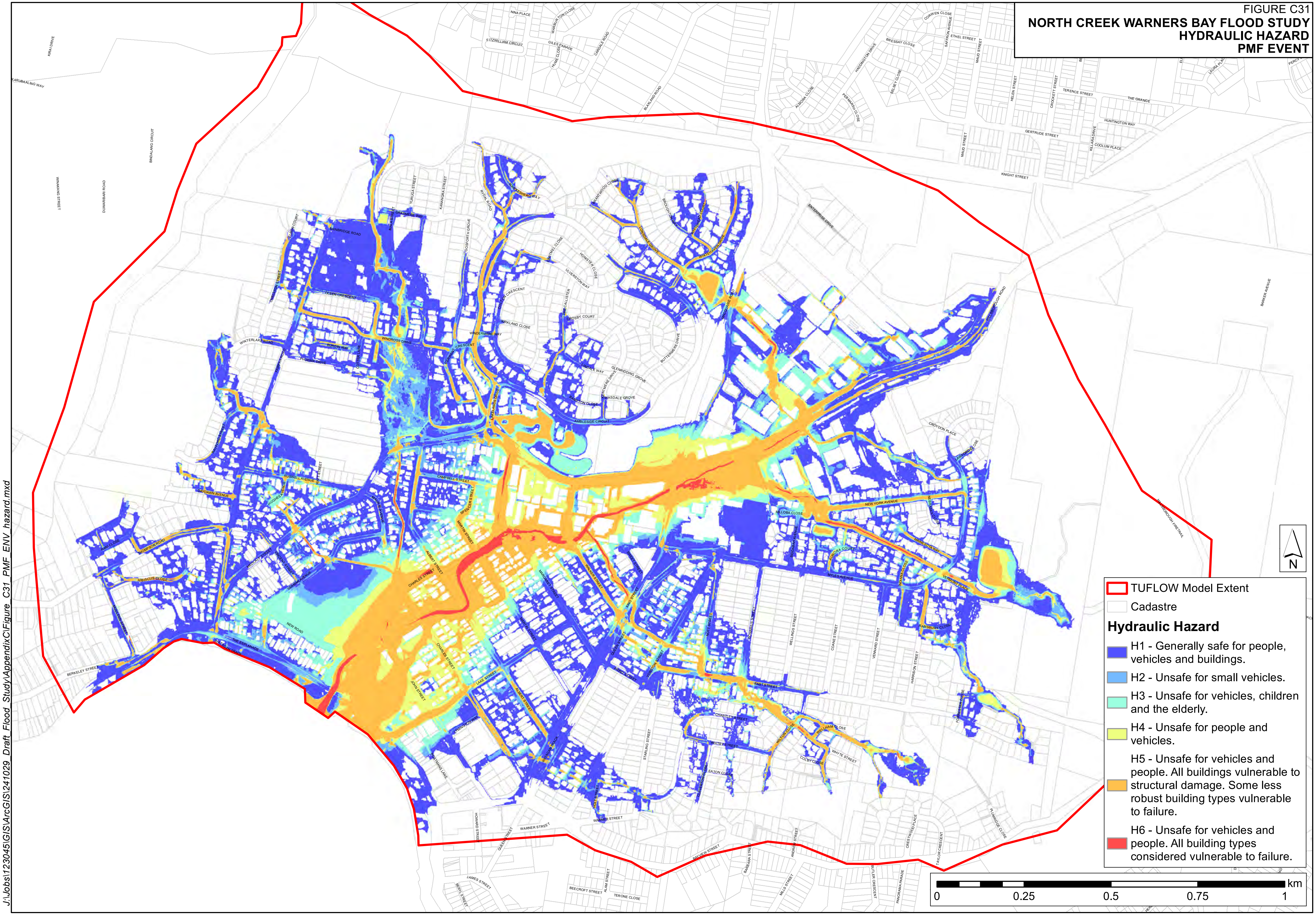
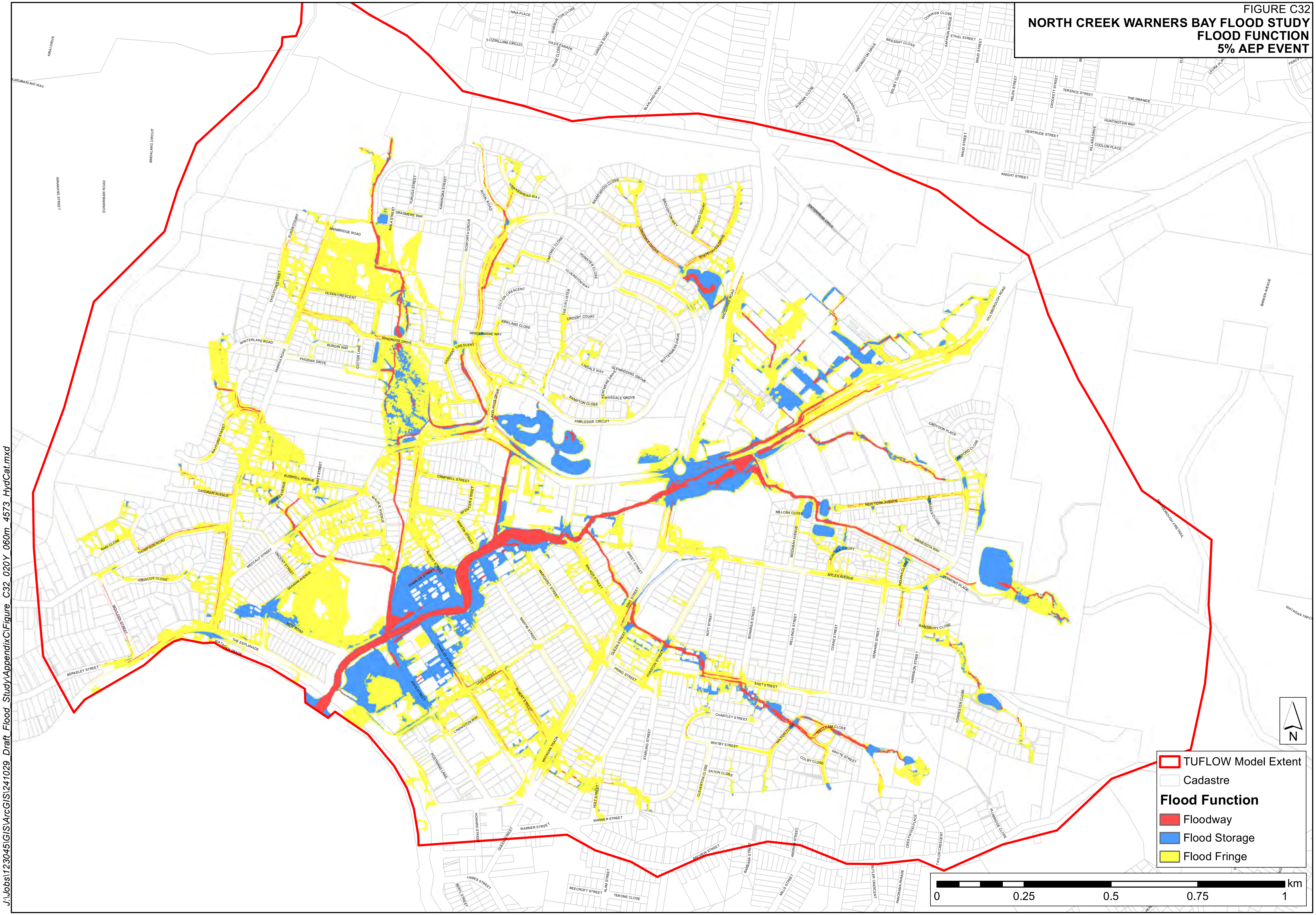


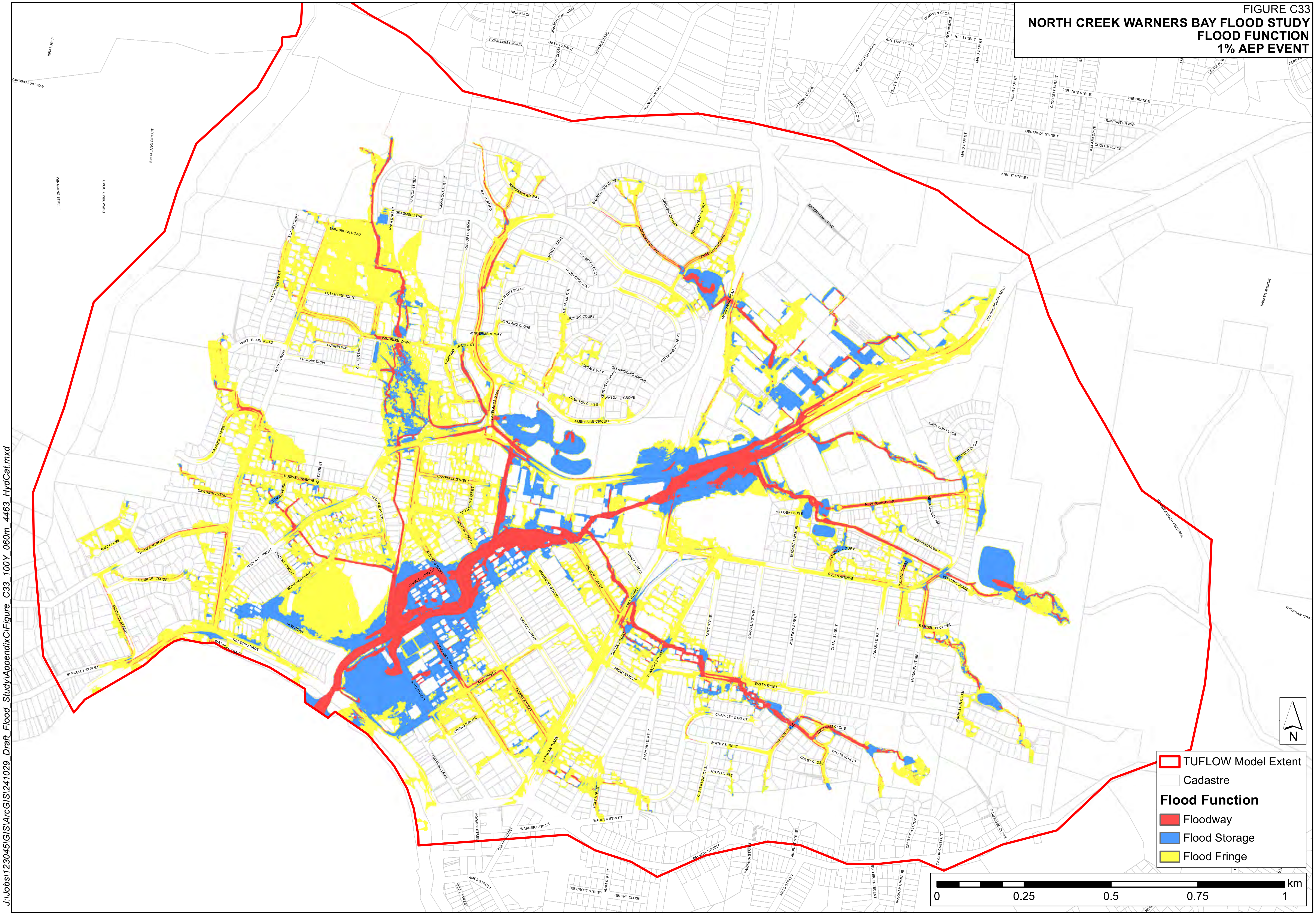
FIGURE C31
NORTH CREEK WARNERS BAY FLOOD STUDY
HYDRAULIC HAZARD
PMF EVENT
TUFLOW Model Extent
Cadastre
Hydraulic Hazard
H1 - Generally safe for people, vehicles and buildings.
H2 - Unsafe for small vehicles.
H3 - Unsafe for vehicles, children and the elderly.
H4 - Unsafe for people and vehicles.
H5 - Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust building types vulnerable to failure.
H6 - Unsafe for vehicles and people. All building types considered vulnerable to failure.
N
0
0.25
0.5
0.75
1
km
J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C31_PMF_ENV_hazard.mxd



J:\Jobs\123045\GIS\ArcGIS\241029_Draft_Flood_Study\AppendixC\Figure_C32_020Y_060m_4573_HydCat.mxd

FIGURE C33

NORTH CREEK WARNERS BAY FLOOD STUDY
FLOOD FUNCTION
1% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\241029 Draft Flood Study\AppendixC\Figure C33 100Y 060m 4463 HydCat.mxd

FIGURE C34

NORTH CREEK WARNERS BAY FLOOD STUDY
FLOOD FUNCTION
0.2% AEP EVENT

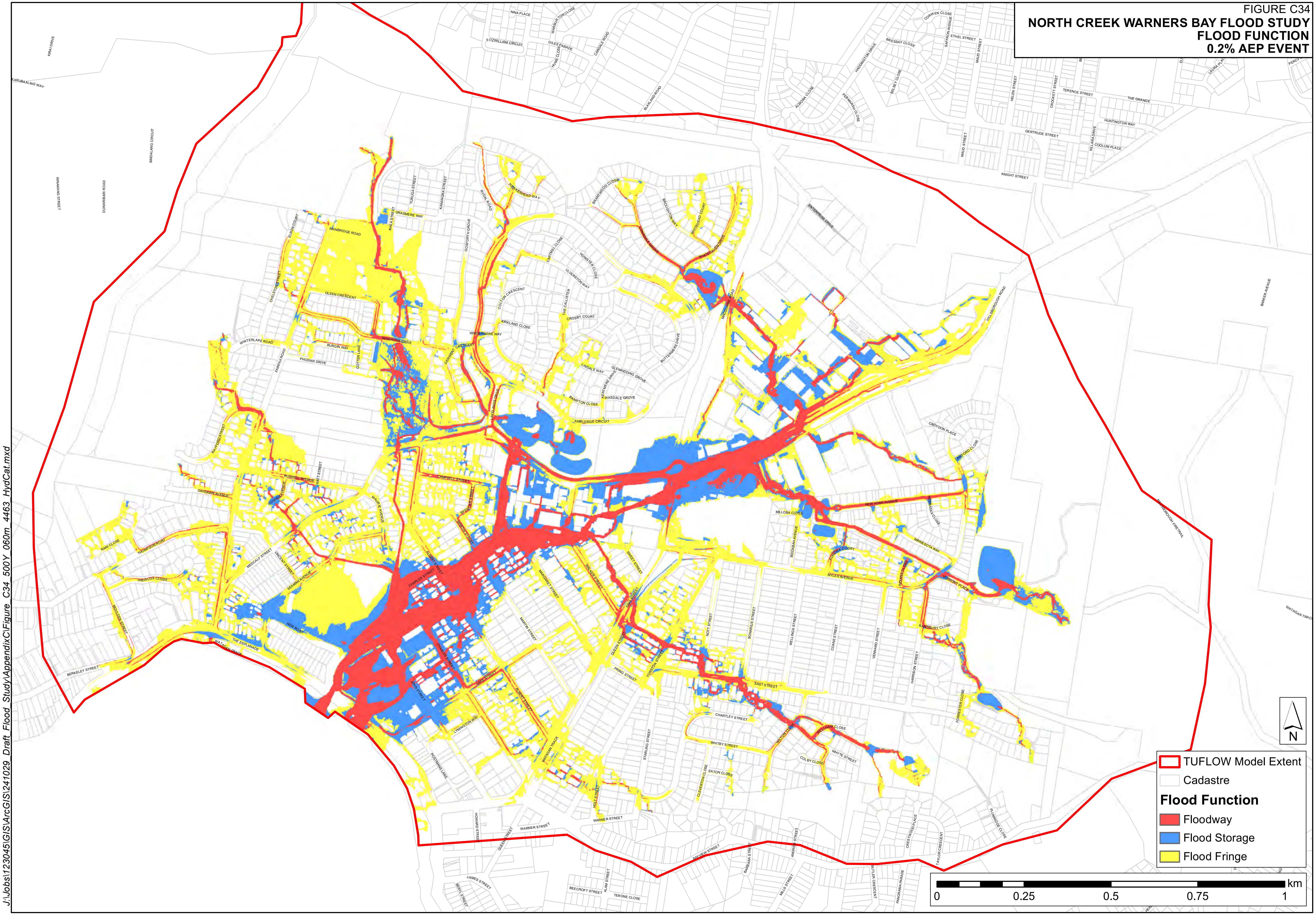
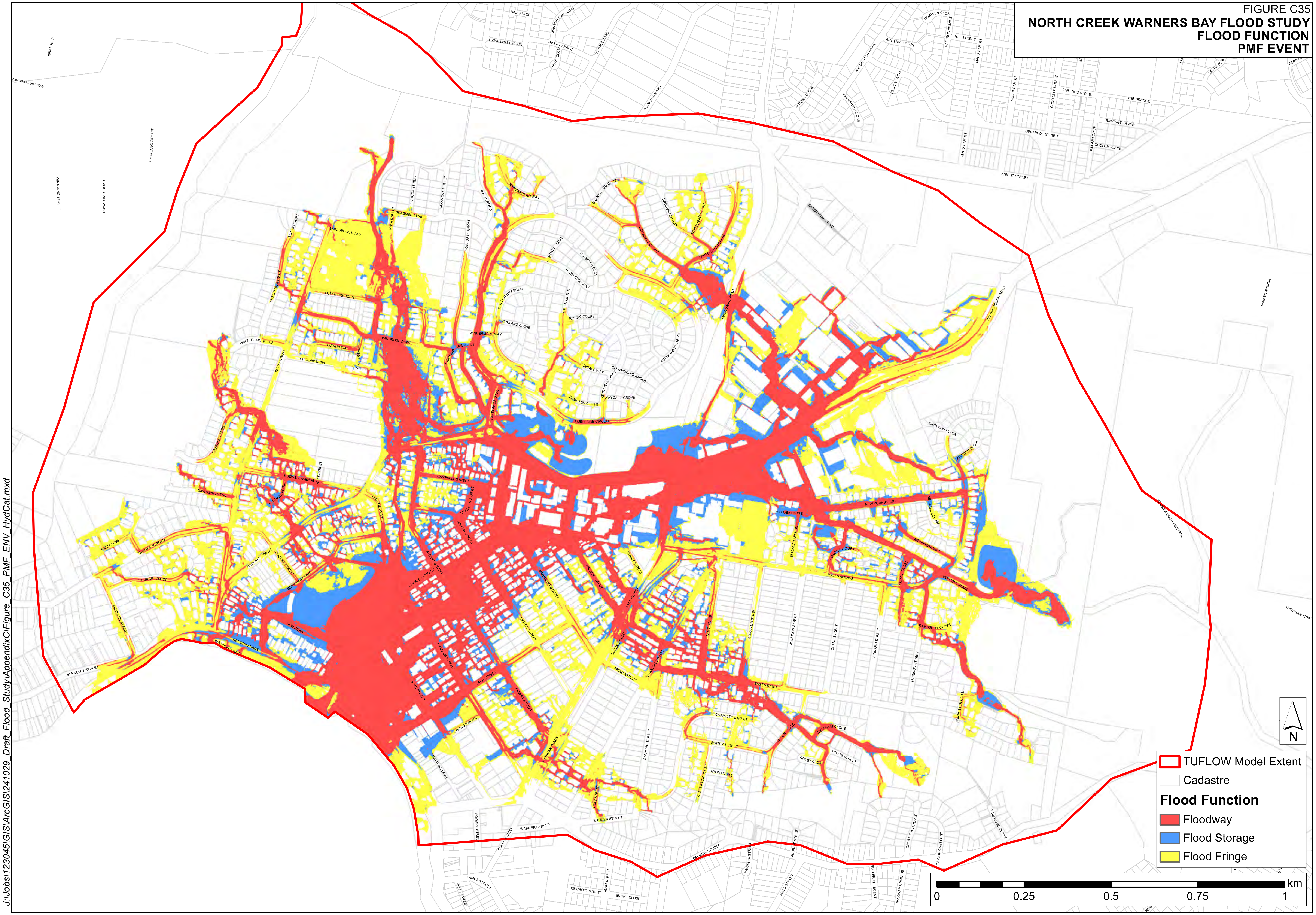


FIGURE C35

NORTH CREEK WARNERS BAY FLOOD STUDY
FLOOD FUNCTION
PMF EVENT



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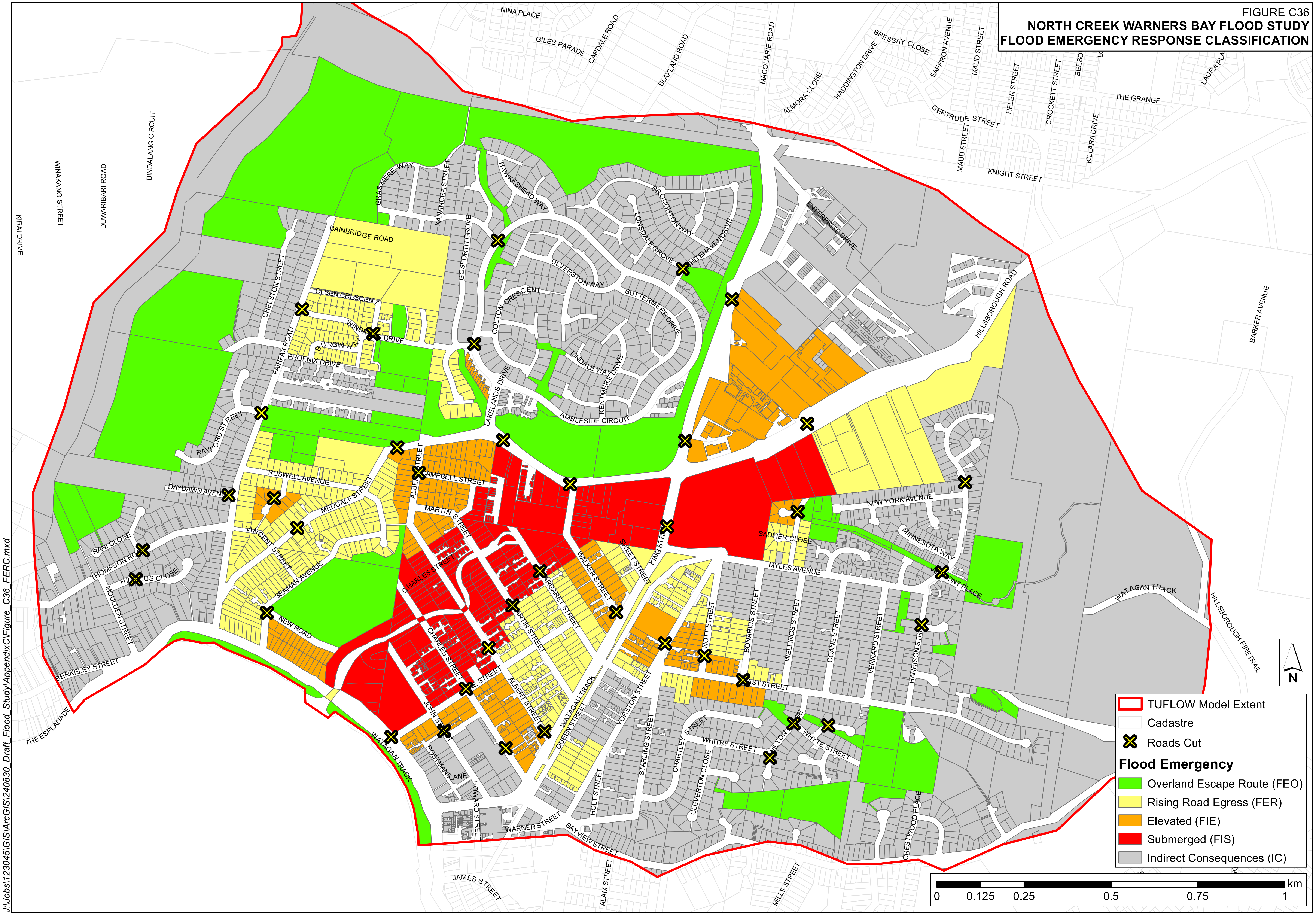
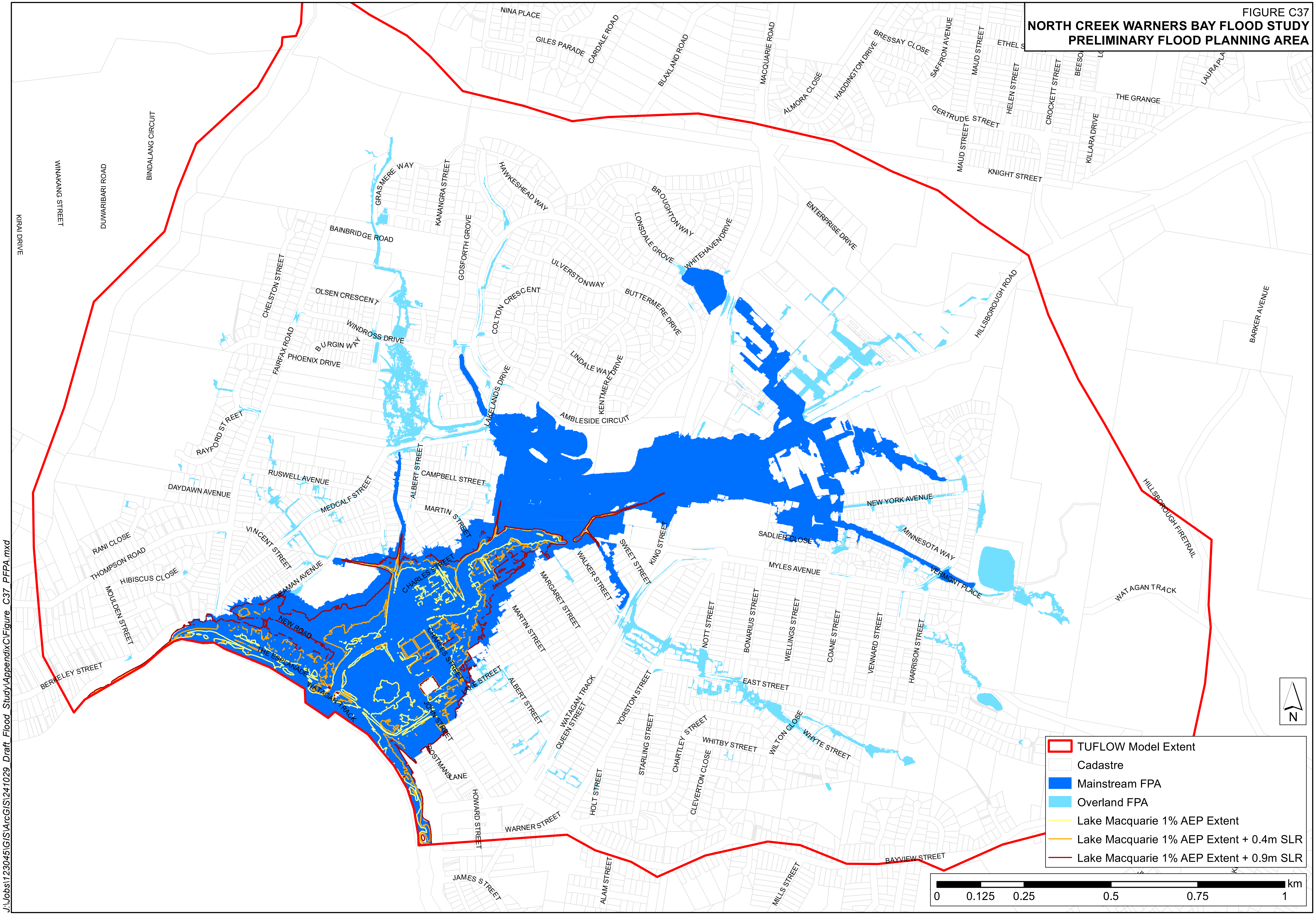


FIGURE C36
NORTH CREEK WARNERS BAY FLOOD STUDY
FLOOD EMERGENCY RESPONSE CLASSIFICATION
TUFLOW Model Extent
Cadastre
Roads Cut
Flood Emergency
Overland Escape Route (FEO)
Rising Road Egress (FER)
Elevated (FIE)
Submerged (FIS)
Indirect Consequences (IC)
0
0.125
0.25
0.5
0.75
1
km
N
J:\Jobs\123045\GIS\ArcGIS\240830 Draft Flood Study\AppendixC\Figure C36 FERC.mxd
NINA PLACE
GILES PARADE
CARDALE ROAD
BLAXLAND ROAD
MACQUARIE ROAD
ALMORA CLOSE
HADDINGTON DRIVE
BRESSAY CLOSE
SAFFRON AVENUE
MAUD STREET
HELEN STREET
CROCKETT STREET
THE GRANGE
GERTRUDE STREET
KILLARA DRIVE
KNIGHT STREET
LAURA PLA
BINDALANG CIRCUIT
DUWARIBARI ROAD
WINAKANG STREET
KIRAI DRIVE
GRASMERE WAY
KANANGRA STREET
HAWKESHEAD WAY
BROUGHTON WAY
LONSDALE GROVE
WHITEHAVEN DRIVE
ENTERPRISE DRIVE
BAINBRIDGE ROAD
GOSFORTH GROVE
ULVERSTON WAY
BUTTERMERE DRIVE
COLTON CRESCENT
CHELSTON STREET
OLSEN CRESCENT
WINDROSS DRIVE
BURGIN WAY
PHOENIX DRIVE
FAIRFAX ROAD
LAKELANDS DRIVE
LINDALE WAY
KENTMERE DRIVE
AMBLESIDE CIRCUIT
HILLSBOROUGH ROAD
BARKER AVENUE
RAYFORD STREET
RUSWELL AVENUE
CAMPBELL STREET
ALBERT STREET
MARTIN STREET
DAYDAWN AVENUE
MEDCALF STREET
VINCENT STREET
NEW YORK AVENUE
SADLIER CLOSE
MINNESOTA WAY
MYLES AVENUE
KING STREET
SWEET STREET
WALKER STREET
MARGARET STREET
CHARLES STREET
RANI CLOSE
THOMPSON ROAD
HIBISCUS CLOSE
SEAMAN AVENUE
NEW ROAD
MOULDEN STREET
BERKELEY STREET
THE ESPLANADE
WATAGAN TRACK
HILLSBOROUGH FIRETRAIL
NOTT STREET
BONARIUS STREET
WELLINGS STREET
COANE STREET
VENNARD STREET
HARRISON STREET
JOHN STREET
POSTMANS LANE
HOWARD STREET
QUEEN STREET
YORSTON STREET
STARLING STREET
CHARTLEY STREET
WHITBY STREET
CLEVERTON CLOSE
WHYTE STREET
HOLT STREET
CRESTWOOD PLACE
WARNER STREET
BAYVIEW STREET
ALAM STREET
MILLS STREET
JAMES STREET

FIGURE C37
NORTH CREEK WARNERS BAY FLOOD STUDY
PRELIMINARY FLOOD PLANNING AREA



APPENDIX D. DESIGN FLOOD RESULTS

Appendix D

Table D1: Peak Flows on North Creek and Tributaries (m^3/s)

ID[1]	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
T01	Seaman Avenue Branch	3.1	4.1	4.6	4.9	5.4	5.8	6.2	6.8	13.1
T02	Western Tributary	2.7	3.8	4.7	5.7	7.2	8.3	9.2	10.6	22.8
T03	Lakelands Branch	2.6	4.8	6.4	8.9	13.2	17.1	20.4	26.2	72.6
T04	King Street Branch	3.9	5.6	6.9	8.7	11.5	13.6	15.4	18.3	42.5
T05	Vermont Place Branch	3.4	5.1	6.5	8.4	12.1	16.3	19.5	23.8	60.4
N01	North Creek at Hillsborough Road service road	0.9	1.5	2.2	2.8	3.6	4.3	4.9	5.7	14.8
N02	North Creek at King Street	7.0	11.1	13.8	17.4	24.1	31.6	38.6	50.4	180.6
N03	North Creek at Walker Street	9.9	16.0	20.0	24.4	30.6	37.6	44.8	56.9	156.3
N04	North Creek at Albert Street	12.7	21.0	26.2	33.4	44.4	56.7	68.8	90.3	345.2
N05	North Creek at the Esplanade	17.9	27.5	31.5	39.1	52.8	67.8	82.3	106.9	451.4

1. Locations shown on Figure 30

Table D2: Detention Basin Performance

ID[1]	Name	Result[2]	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
B01	Inala Street Basin	PWL	24.9	25.0	25.1	25.1	25.1	25.1	25.1	25.1	25.2
		LF	0.0	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2
		HF	0.1	0.6	0.9	1.0	1.1	1.2	1.4	1.6	4.4
B02	Wetland	PWL	15.2	15.5	15.7	15.8	16.0	16.2	16.2	16.3	16.6
		LF	2.0	4.0	5.3	6.3	7.5	8.1	8.3	8.5	9.2
		HF	0.0	0.0	0.0	0.0	0.0	0.4	1.3	2.7	17.0
B03	Burgin Way	PWL	16.6	16.6	16.6	16.6	16.6	16.6	16.6	16.6	16.8
		LF	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
		HF	0.1	0.1	0.3	0.4	0.5	0.7	0.8	1.0	7.3
B04	Lakelands Pond	PWL	5.2	5.5	5.7	5.8	5.9	5.9	6.0	6.0	6.3
		LF	2.5	4.3	4.6	4.6	4.6	4.6	4.6	4.6	4.6
		HF	0.0	0.0	0.5	2.1	4.3	8.2	11.2	15.6	42.3
B05	Whitehaven Drive	PWL	15.1	15.2	15.4	15.5	15.7	15.7	15.8	15.8	16.1
		LF	0.6	0.7	0.8	1.2	1.5	1.7	1.8	1.9	2.5

ID[1]	Name	Result[2]	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
		HF	0.0	0.1	0.5	1.9	4.8	7.2	8.9	11.1	35.7
B06	Vermont Place	PWL	20.5	20.9	20.9	21.1	21.4	21.5	21.5	21.6	22.0
		LF1	0.3	0.9	1.0	1.4	1.6	1.7	1.7	1.8	2.0
		LF2	0.0	0.0	0.0	0.0	0.3	2.2	3.8	6.3	28.4
		HF1	35.2	35.8	35.9	36.0	36.0	36.0	36.0	36.1	36.2
		HF2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
B07	Forrester Close	PWL	0.0	0.0	0.5	1.4	2.1	2.7	3.1	3.8	10.8
		LF	9.9	10.5	10.7	10.9	11.0	11.0	11.0	11.0	11.2
		HF	0.7	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.0
B08	Aurora Circuit	PWL	0.0	0.0	0.0	0.1	1.0	1.6	1.9	2.4	7.7
		LF	9.1	9.3	9.4	9.4	9.5	9.6	9.7	9.8	10.3
		HF	1.0	1.4	1.9	2.7	4.1	6.3	8.3	11.4	49.0
B09	New York Avenue	PWL	7.9	8.1	8.5	8.7	8.9	9.1	9.1	9.2	9.7
		HF	0.1	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3
B10	Biddabah Avenue	PWL	0.0	0.0	0.0	0.1	0.3	0.5	0.2	0.2	0.1
		LF	31.1	31.2	31.3	31.3	31.3	31.4	31.4	31.4	31.7
		HF	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.4
B11	Wellham Close	PWL	0.8	1.6	2.4	3.1	3.7	4.3	5.0	6.1	16.6
		LF	19.3	20.5	20.8	20.9	21.0	21.1	21.1	21.2	21.8
		HF	1.9	2.7	2.7	2.7	2.8	2.8	2.8	2.8	2.8
B12	Wilton Close	PWL	0.0	0.3	2.5	4.2	5.8	7.3	8.7	11.0	34.3
		LF	0.0	0.1	0.5	1.9	4.8	7.2	8.9	11.1	35.7
		HF	20.5	20.9	20.9	21.1	21.4	21.5	21.5	21.6	22.0

1. Locations shown on Figure 30
2. Results: PWL = Peak Water Level [mAHD], LF = Low Flow Pipe Outlet [m^3/s], HF = High Flow Weir/Overtopping [m^3/s]

Table D3: Peak Flood Levels (mAHD) at Key Locations

ID[1]	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
S01	Medcalf Street at Seaman Avenue Branch	7.3	7.4	7.5	7.5	7.5	7.6	7.6	7.6	7.7
S02	Seaman Avenue at Seaman Avenue Branch	4.1	4.1	4.1	4.1	4.2	4.2	4.2	4.2	4.4
W01	Windross Drive at Western Tributary	16.4	16.5	16.6	16.6	16.6	16.7	16.7	16.8	17.0
W02	Medcalf Street at Western Tributary	8.3	8.4	8.4	8.4	8.4	8.5	8.5	8.5	8.6
L01	Medcalf Street at Lakelands Branch	4.7	4.9	5.0	5.1	5.1	5.1	5.2	5.2	6.2
K01	Nott Street at King Street Branch	14.4	14.7	14.8	14.9	15.1	15.2	15.3	15.4	15.8
K02	Yorston Street at King Street Branch	12.6	12.8	12.9	13.0	13.0	13.1	13.1	13.1	13.3
K03	Queen Street at King Street Branch	10.3	10.3	10.3	10.4	10.5	10.5	10.5	10.6	10.8
K04	King Street at King Street Branch	8.7	8.7	8.7	8.8	9.0	9.0	9.1	9.1	9.3
N01	Hillsborough Road service road at North Creek	5.4	5.4	5.7	5.8	6.0	6.1	6.2	6.4	7.7
N02	King Street at North Creek	4.9	5.1	5.3	5.6	5.9	6.0	6.2	6.4	7.6
N03	Walker Street at North Creek	2.8	2.9	3.0	3.1	3.3	3.4	3.6	3.8	5.3
N05	The Esplanade at North Creek	0.7	0.7	0.7	0.7	0.8	1.0	1.1	1.3	2.4
V01	Myles Avenue at Vermont Place Branch	17.0	17.0	17.0	17.1	17.2	17.3	17.3	17.4	17.6
R01	Hughes Avenue	10.2	10.3	10.4	10.5	10.5	10.5	10.6	10.7	11.1
R02	Macquarie Road	14.0	14.0	14.1	14.2	14.2	14.3	14.3	14.3	14.6
R03	Myles Avenue (north)	15.4	15.6	15.7	15.7	15.8	15.8	15.8	15.9	16.1
R04	Nebraska Close	13.5	13.6	13.7	13.7	13.8	13.8	13.8	13.9	14.2
R05	Colorado Close	11.4	11.5	11.6	11.6	11.7	11.7	11.7	11.8	12.3
R06	New York Avenue	10.7	10.7	10.8	10.8	10.9	10.9	10.9	11.0	11.4

ID[1]	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
R07	Wilton Close	22.2	22.3	22.4	22.4	22.5	22.5	22.5	22.6	23.0
R08	Margaret Street	2.0	2.2	2.3	2.5	2.7	2.9	3.0	3.3	4.9
R09	Martin Street (east)	1.6	2.0	2.1	2.3	2.4	2.6	2.7	3.0	4.3
R10	Albert Street (west)	1.5	1.7	1.8	1.9	2.1	2.2	2.3	2.5	3.7
R11	Albert Street (east)	1.6	1.8	1.9	2.0	2.1	2.3	2.4	2.5	3.7
R12	Charles Street (west)	1.2	1.5	1.6	1.8	2.0	2.1	2.2	2.3	3.3
R13	Charles Street (east)	1.3	1.5	1.6	1.8	1.9	2.1	2.2	2.3	3.2
R14	John Street	1.2	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.8
R15	The Esplanade (west)	1.4	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.8
R16	The Esplanade (east)	1.5	1.5	1.5	1.6	1.6	1.7	1.7	1.8	2.4
R17	Seaman Avenue South	2.1	2.2	2.3	2.4	2.5	2.5	2.5	2.6	3.0

1. Locations shown on Figure 30

Table D4: Peak Flood Depths (m) at Key Locations

ID[1]	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
S01	Medcalf Street at Seaman Avenue Branch	<0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.5
S02	Seaman Avenue at Seaman Avenue Branch	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.3
W01	Windross Drive at Western Tributary	<0.1	<0.1	0.1	0.2	0.2	0.3	0.3	0.3	0.6
W02	Medcalf Street at Western Tributary	<0.1	<0.1	<0.1	<0.1	0.1	0.1	0.1	0.2	0.3
L01	Medcalf Street at Lakelands Branch	0.4	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.9
K01	Nott Street at King Street Branch	<0.1	0.4	0.4	0.6	0.8	0.9	1.0	1.0	1.4
K02	Yorston Street at King Street Branch	NF	0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.8
K03	Queen Street at King Street Branch	0.3	0.3	0.3	0.4	0.5	0.5	0.5	0.6	0.7

ID[1]	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
K04	King Street at King Street Branch	<0.1	<0.1	<0.1	0.1	0.3	0.3	0.4	0.4	0.6
N01	Hillsborough Road service road at North Creek	0.4	0.4	0.6	0.8	0.9	1.0	1.2	1.4	2.6
N02	King Street at North Creek	<0.1	0.2	0.4	0.7	1.0	1.2	1.3	1.5	2.7
N03	Walker Street at North Creek	0.3	0.3	0.4	0.6	0.7	0.9	1.0	1.2	2.8
N05	The Esplanade at North Creek	NF	NF	NF	NF	NF	NF	NF	NF	0.1
V01	Myles Avenue at Vermont Place Branch	NF	NF	NF	<0.1	0.1	0.2	0.3	0.4	0.6
R01	Hughes Avenue	NF	0.1	0.2	0.3	0.3	0.4	0.4	0.5	0.9
R02	Macquarie Road	NF	NF	<0.1	0.1	0.2	0.2	0.3	0.3	0.6
R03	Myles Avenue (north)	<0.1	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.7
R04	Nebraska Close	0.2	0.2	0.3	0.3	0.4	0.4	0.4	0.5	0.8
R05	Colorado Close	<0.1	0.1	0.2	0.2	0.3	0.3	0.3	0.4	0.9
R06	New York Avenue	<0.1	<0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.7
R07	Wilton Close	0.2	0.3	0.4	0.4	0.5	0.5	0.6	0.6	1.1
R08	Margaret Street	<0.1	0.1	0.3	0.4	0.7	0.8	1.0	1.3	2.9
R09	Martin Street (east)	<0.1	0.4	0.6	0.7	0.9	1.0	1.2	1.4	2.8
R10	Albert Street (west)	<0.1	0.2	0.3	0.5	0.6	0.8	0.9	1.1	2.3
R11	Albert Street (east)	NF	0.2	0.3	0.4	0.5	0.7	0.8	0.9	2.1
R12	Charles Street (west)	<0.1	0.3	0.5	0.6	0.8	0.9	1.0	1.2	2.1
R13	Charles Street (east)	0.1	0.3	0.4	0.5	0.7	0.8	0.9	1.1	1.9
R14	John Street	<0.1	0.3	0.3	0.5	0.6	0.7	0.8	0.9	1.7
R15	The Esplanade (west)	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.4	0.5
R16	The Esplanade (east)	<0.1	<0.1	<0.1	<0.1	0.2	0.2	0.3	0.3	0.9
R17	Seaman Avenue South	<0.1	0.2	0.2	0.3	0.4	0.4	0.5	0.5	1.0

1. Locations shown on Figure 30

Table D5: Peak Flood Flows (m^3/s) at Key Locations

ID[1]	Name	Result[2]	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
S01	Medcalf Street at Seaman Avenue Branch	C	3.0	3.4	3.5	3.6	3.7	3.7	3.8	4.9	5.4
		OT	0.0	1.0	2.3	3.2	4.5	5.6	6.5	7.8	24.9
S02	Seaman Avenue at Seaman Avenue Branch	C	3.1	4.1	4.6	5.0	5.0	5.0	5.0	5.1	5.4
		OT	0.0	0.0	0.2	0.6	1.6	2.4	3.2	4.3	15.0
W01	Windross Drive at Western Tributary	C	2.0	4.0	5.3	6.3	7.5	8.1	8.3	8.5	9.2
		OT	0.0	0.0	0.6	0.9	1.5	2.7	4.3	6.8	31.5
W02	Medcalf Street at Western Tributary	C	2.4	3.0	3.0	3.0	3.1	3.1	3.1	3.1	3.1
		OT	0.0	1.2	2.7	4.1	6.2	7.9	9.6	12.5	41.2
L01	Medcalf Street at Lakelands Branch	C	2.5	4.3	4.6	4.6	4.6	4.6	4.6	4.6	4.6
		OT	0.0	0.0	1.0	3.2	7.7	12.3	16.2	22.2	70.2
K04	King Street at King Street Branch	C	3.7	5.3	6.5	8.0	8.5	8.5	8.5	8.5	8.8
		OT	0.0	0.1	0.4	1.2	4.4	7.5	10.1	14.2	53.4
N02	King Street at North Creek	C	3.7	5.9	8.1	8.4	8.4	8.5	8.5	8.5	10.8
		OT	0.1	0.1	0.4	2.5	6.7	9.4	12.0	17.5	67.7
N03	Walker Street at North Creek	C	7.1	9.5	9.6	9.5	9.6	9.6	9.7	9.8	10.7
		OT	0.0	3.9	8.4	12.7	20.6	28.9	36.3	48.8	190.5
N05	The Esplanade at North Creek	CH	10.0	16.1	18.5	19.9	20.7	21.0	21.1	21.1	22.9
		OT W	0.0	0.1	1.7	4.9	10.7	18.0	25.7	39.7	167.2
		OT E	17.9	27.5	31.5	38.6	47.8	56.9	65.0	76.1	135.7

1. Locations shown on Figure 30
2. Results: C = Culvert flow [m^3/s], OT = Overtopping flow [m^3/s], CH = Channel flow under bridge [m^3/s]

FIGURE D1
WATER LEVEL PROFILE
NORTH CREEK
DESIGN STORMS

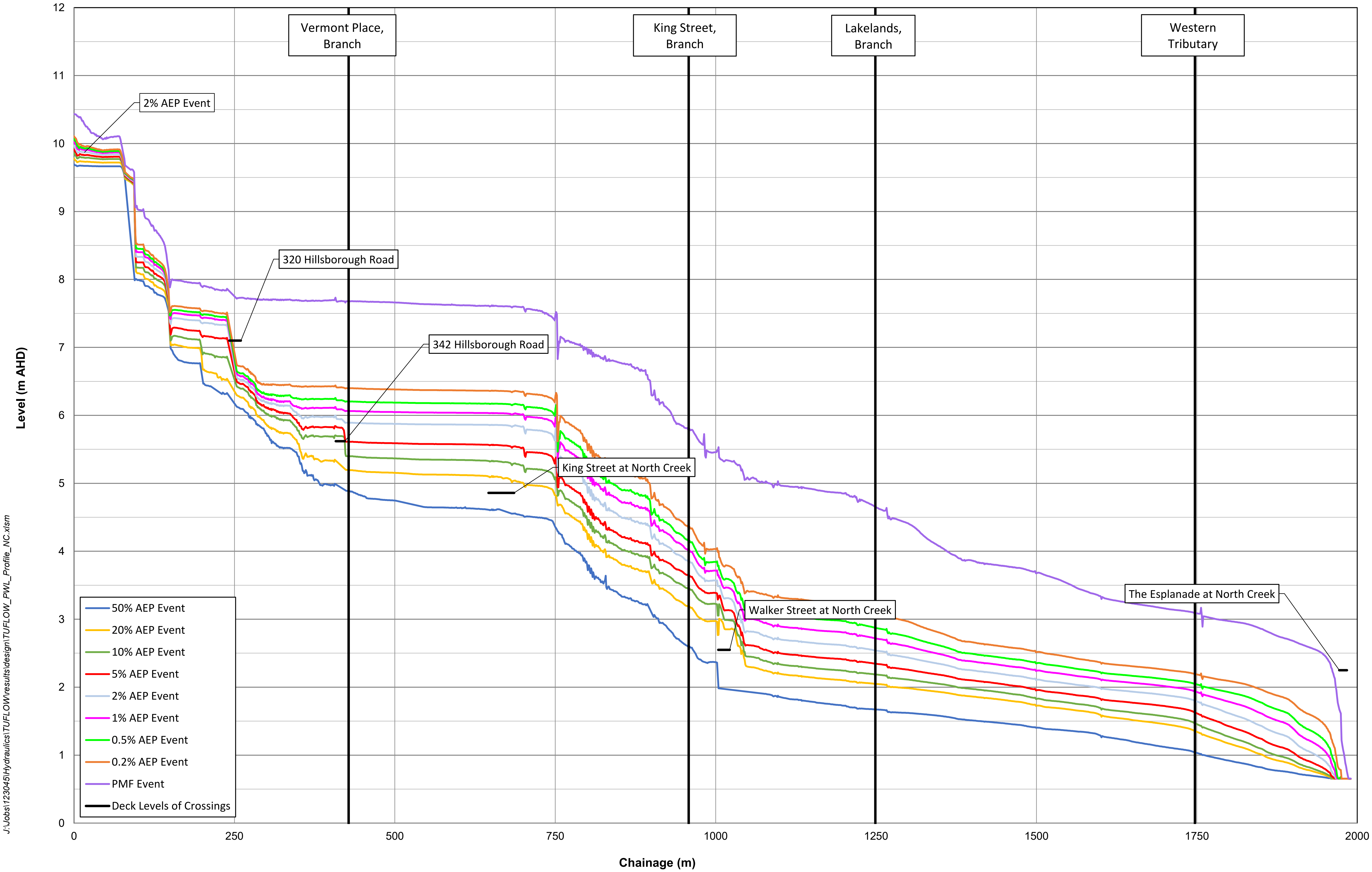


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FIGURE D2
WATER LEVEL PROFILE
SEAMAN AVENUE BRANCH
DESIGN STORMS

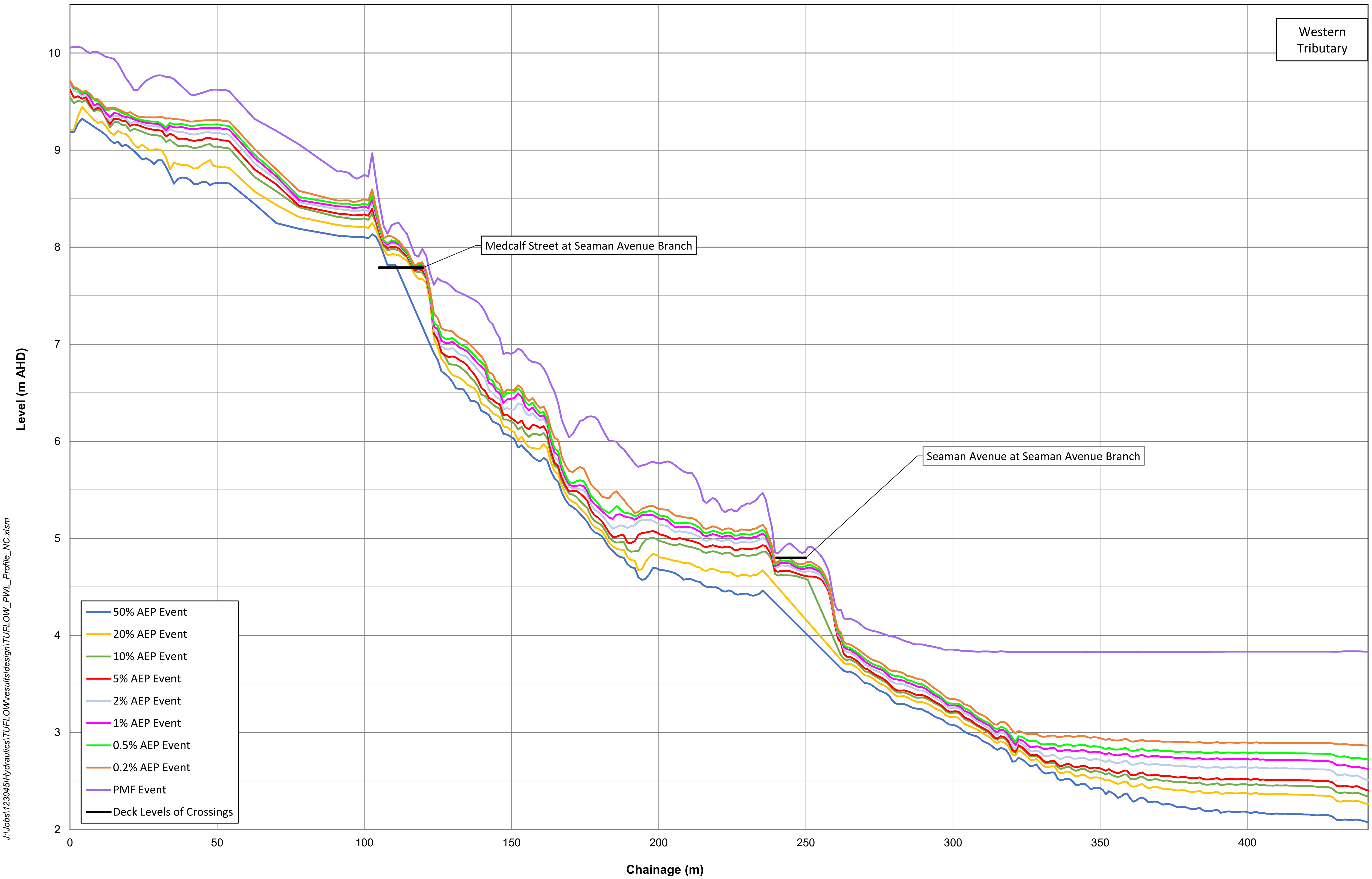

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FIGURE D3
WATER LEVEL PROFILE
WESTERN TRIBUTARY
DESIGN STORMS

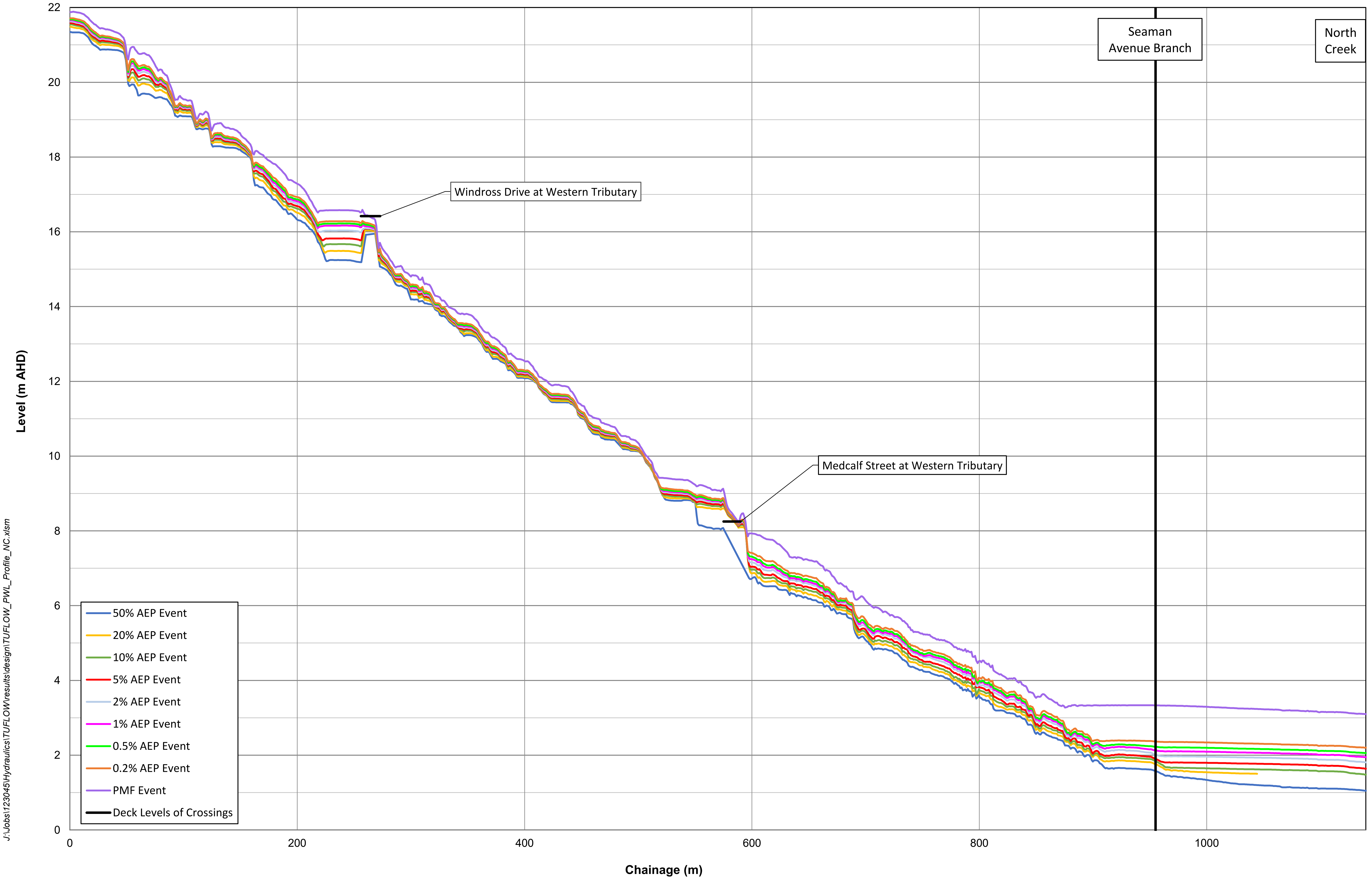

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FIGURE D4
WATER LEVEL PROFILE
LAKESLAND BRANCH
DESIGN STORMS

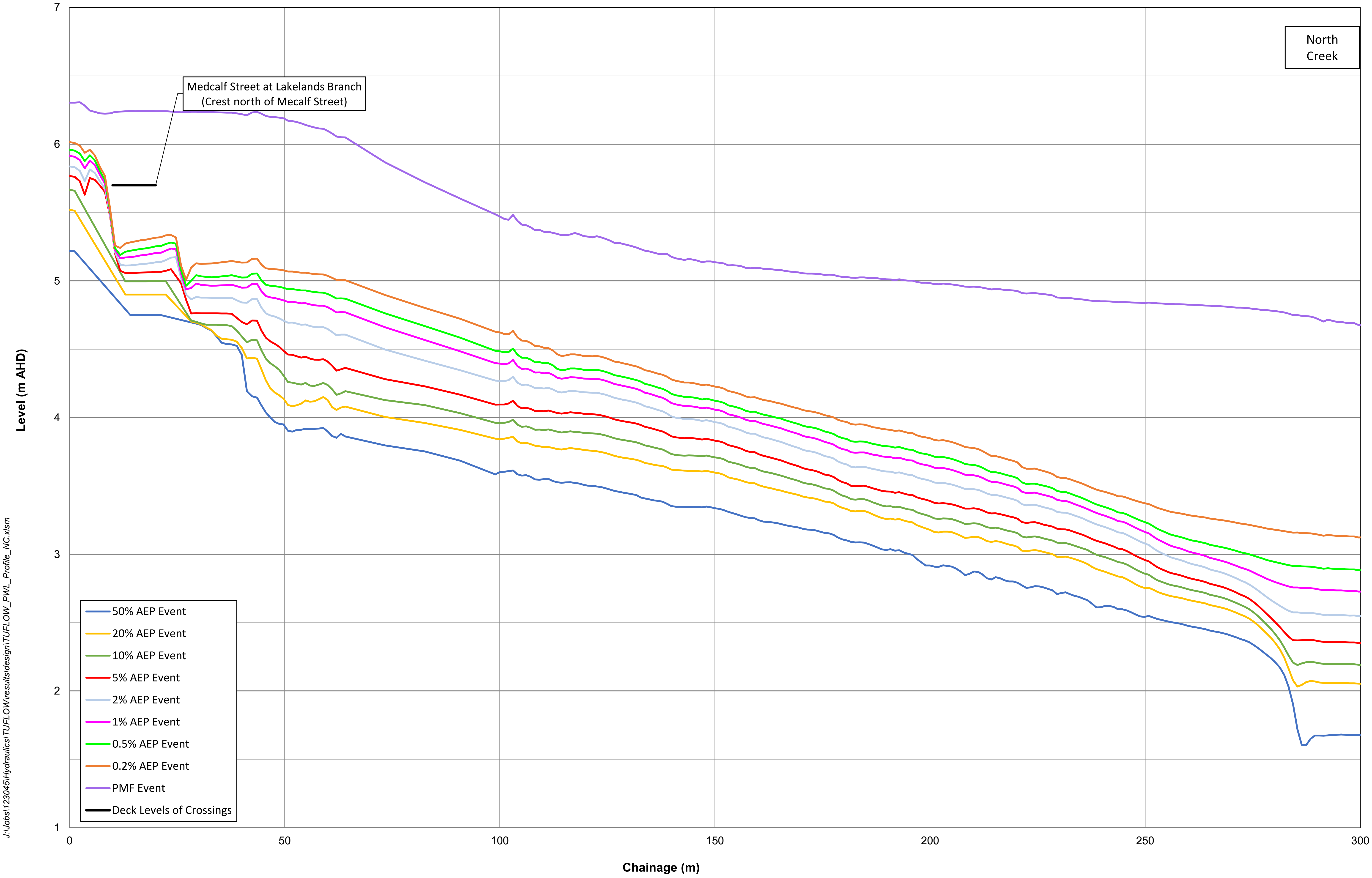

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FIGURE D5
WATER LEVEL PROFILE
VERMONT PLACE BRANCH
DESIGN STORMS

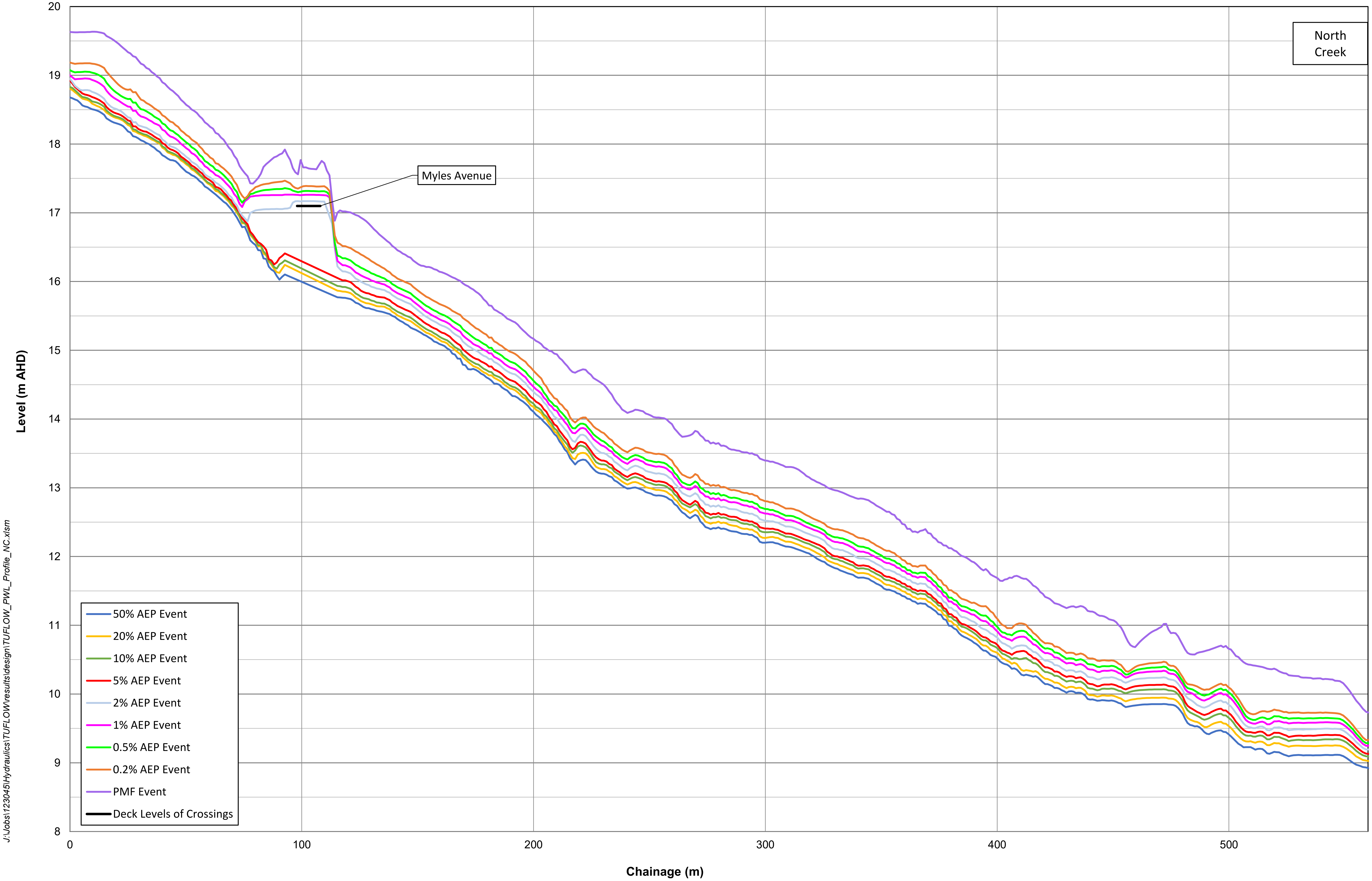

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FIGURE D6
WATER LEVEL PROFILE
KING STREET BRANCH
DESIGN STORMS

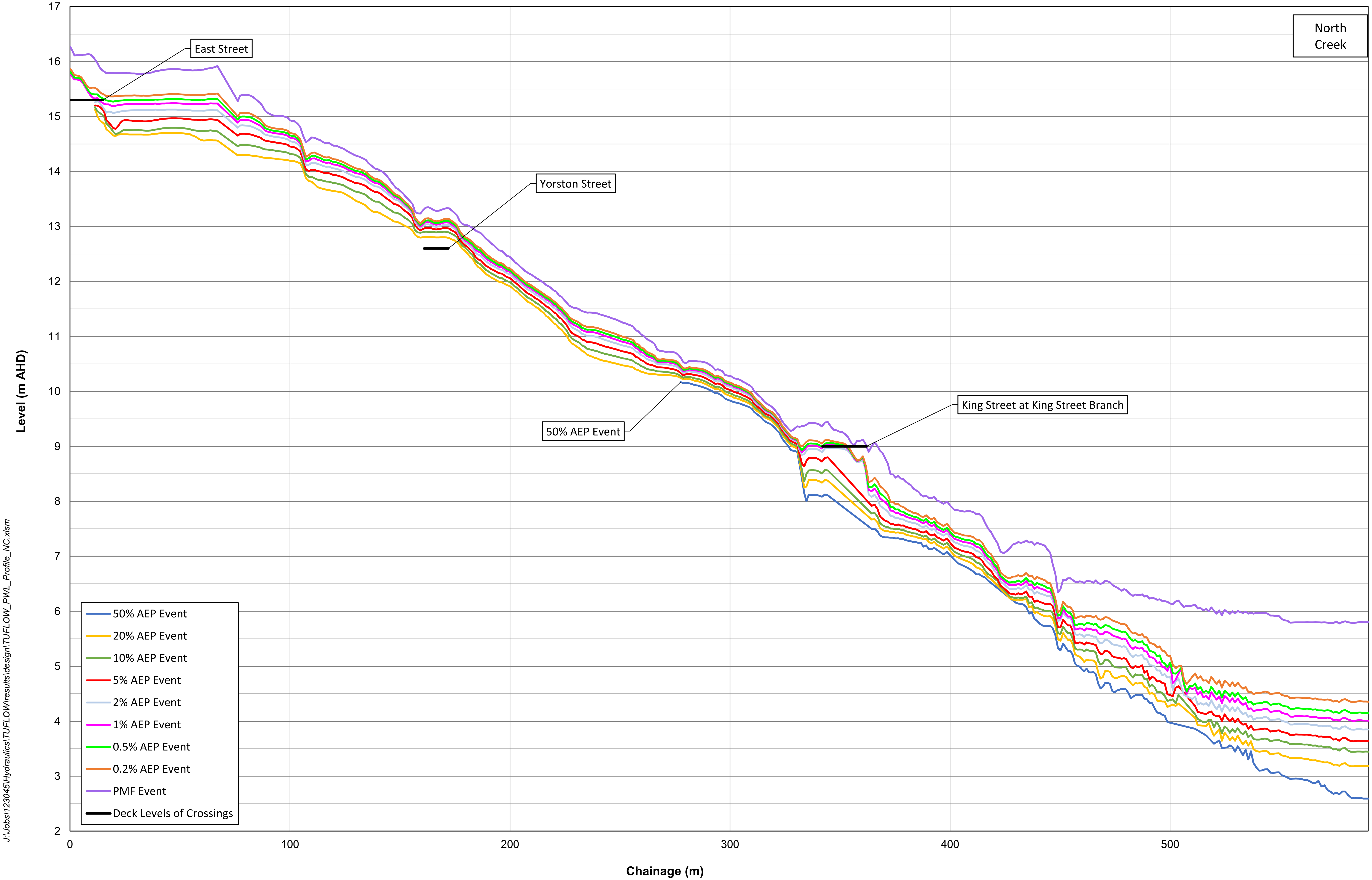

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FIGURE D7

STAGE HYDROGRAPH
S01: MEDCALF STREET AT SEAMAN AVENUE BRANCH
DESIGN STORMS

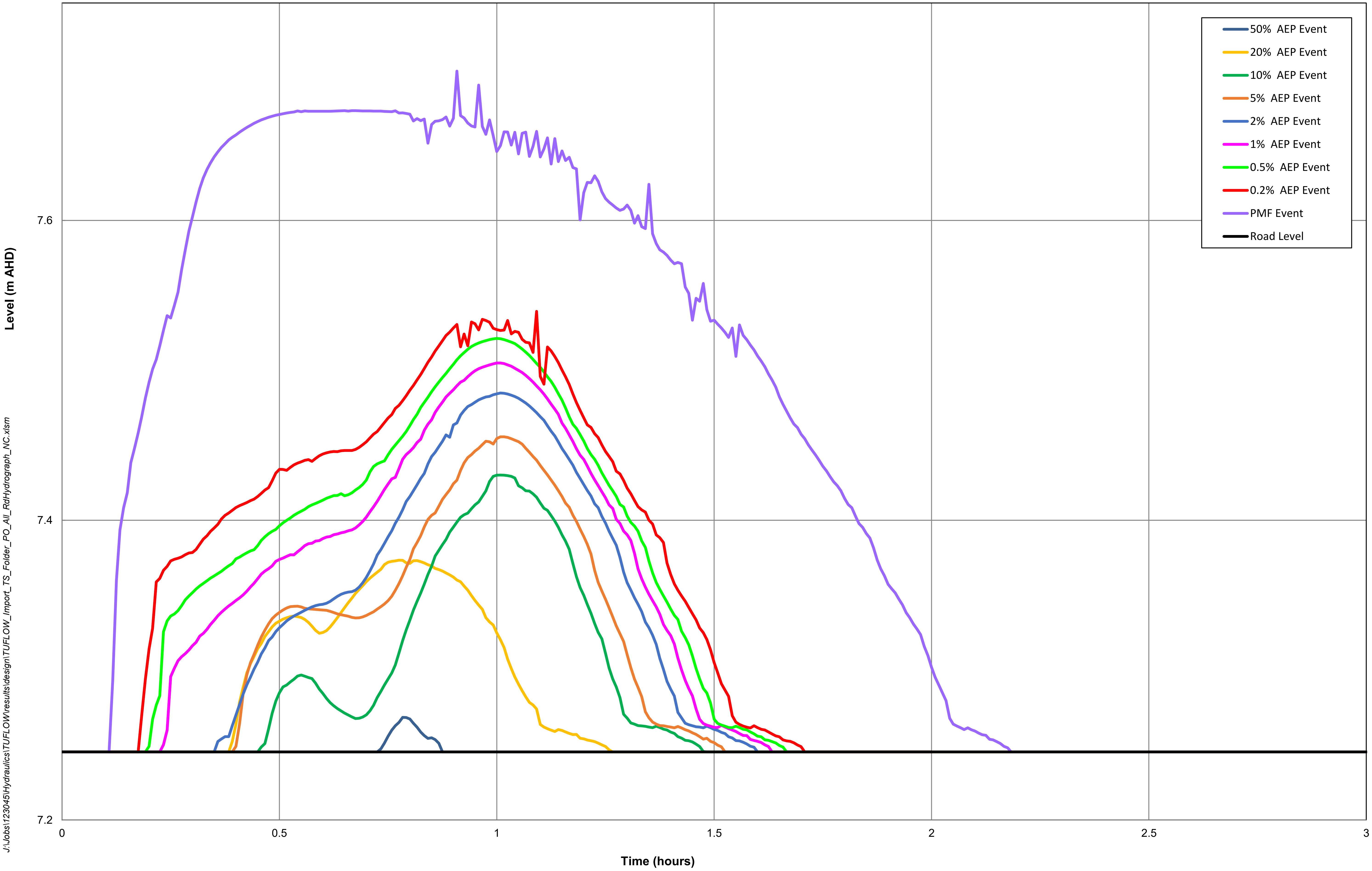

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FIGURE D8

STAGE HYDROGRAPH
S02: SEAMAN AVENUE AT SEAMAN AVENUE BRANCH
DESIGN STORMS

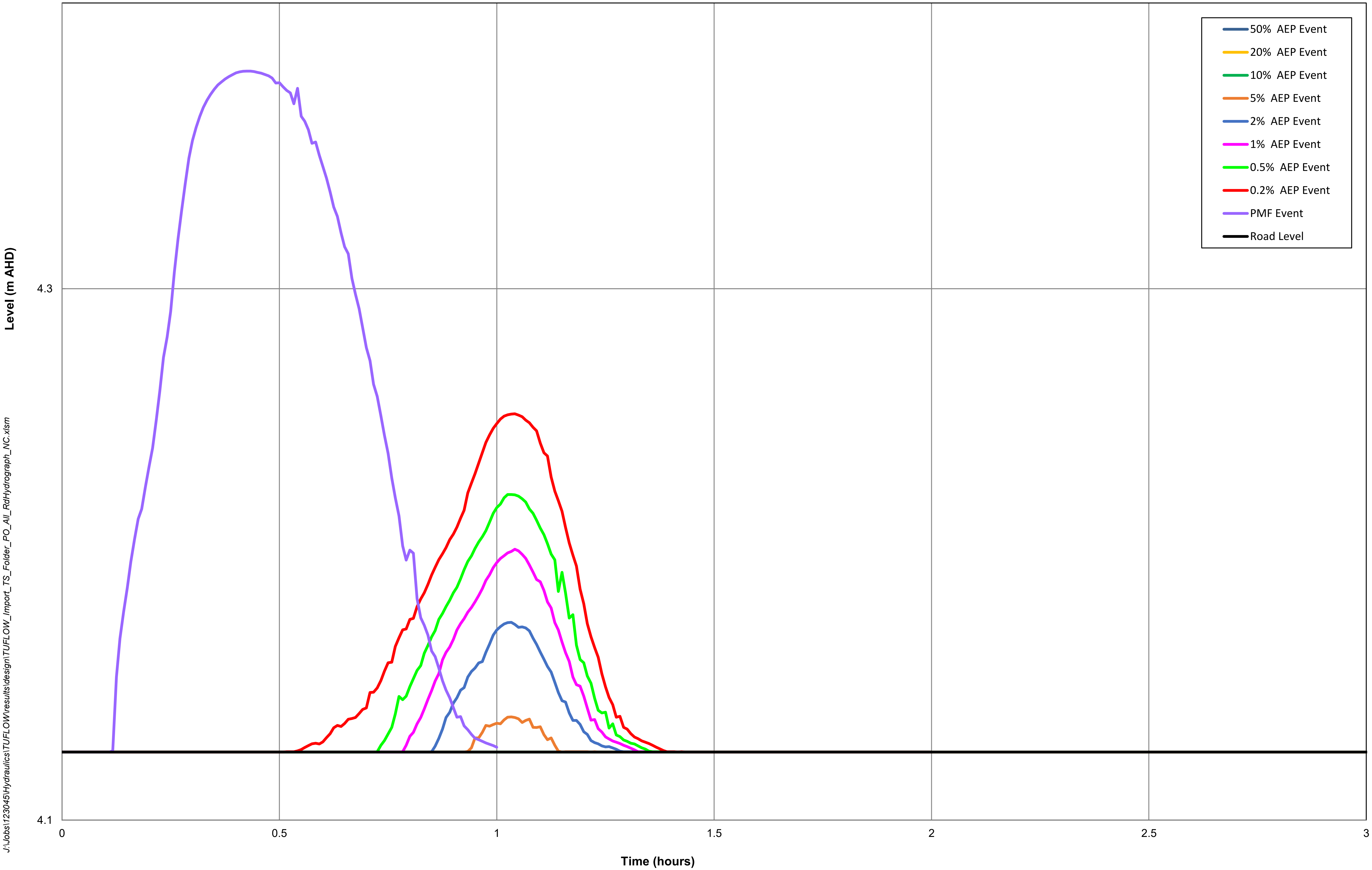

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FIGURE D9

STAGE HYDROGRAPH
W01: WINDROSS DRIVE AT WESTERN TRIBUTARY
DESIGN STORMS

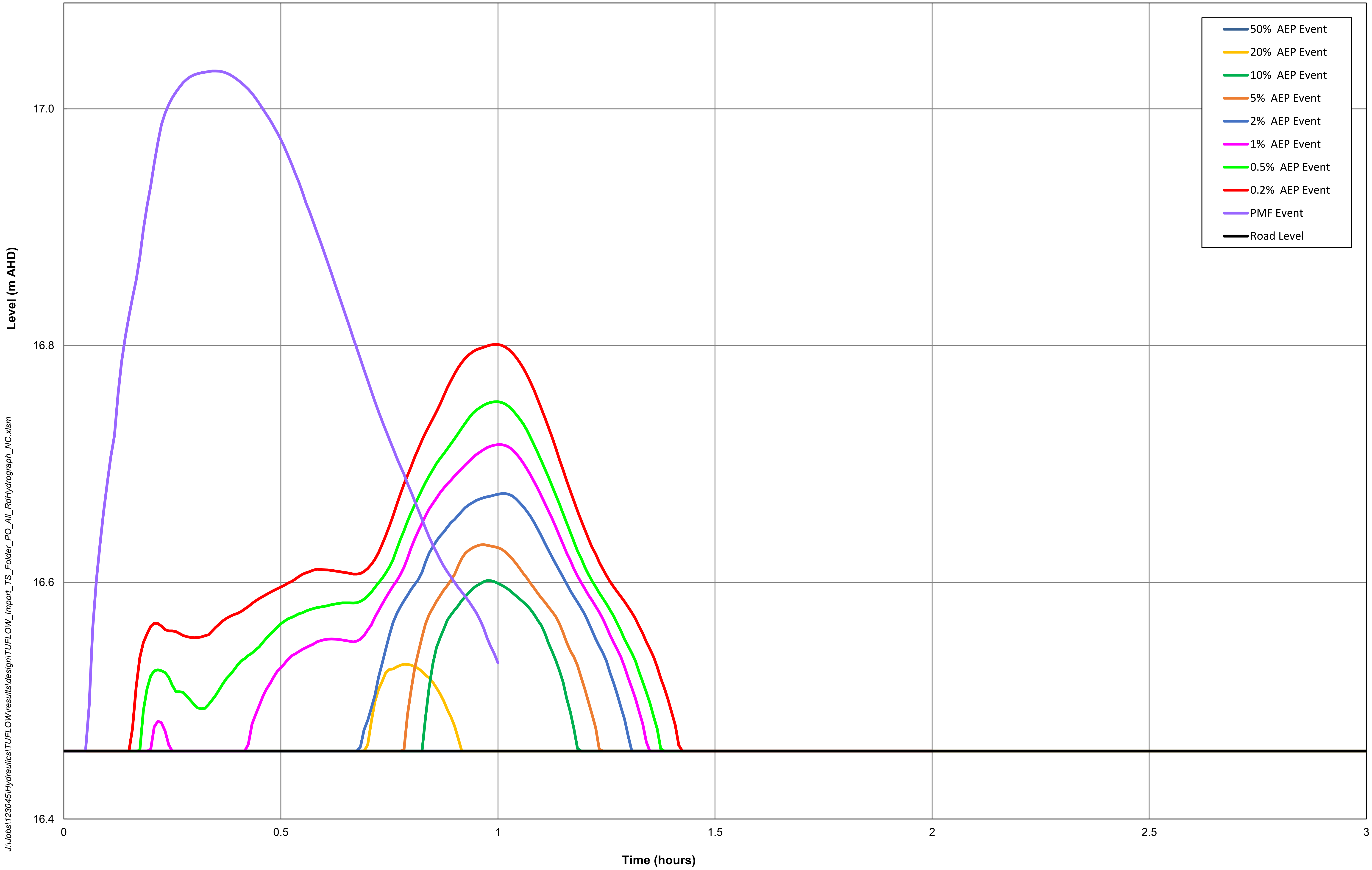

FIGURE D10

STAGE HYDROGRAPH
W02: MEDCALF STREET AT WESTERN TRIBUTARY
DESIGN STORMS

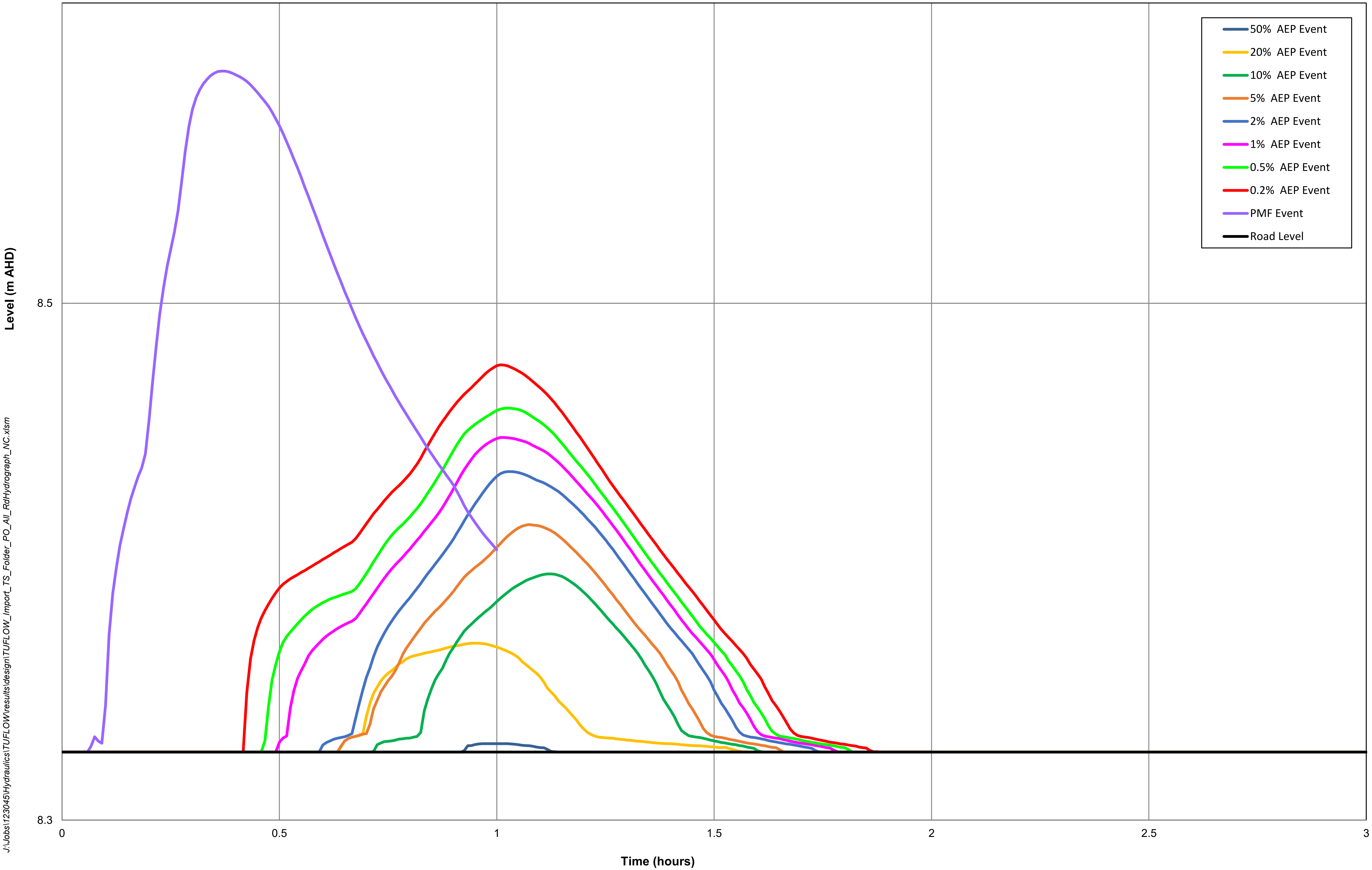

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FIGURE D11

STAGE HYDROGRAPH
L01: MEDCALF STREET AT LAKELANDS BRANCH
DESIGN STORMS

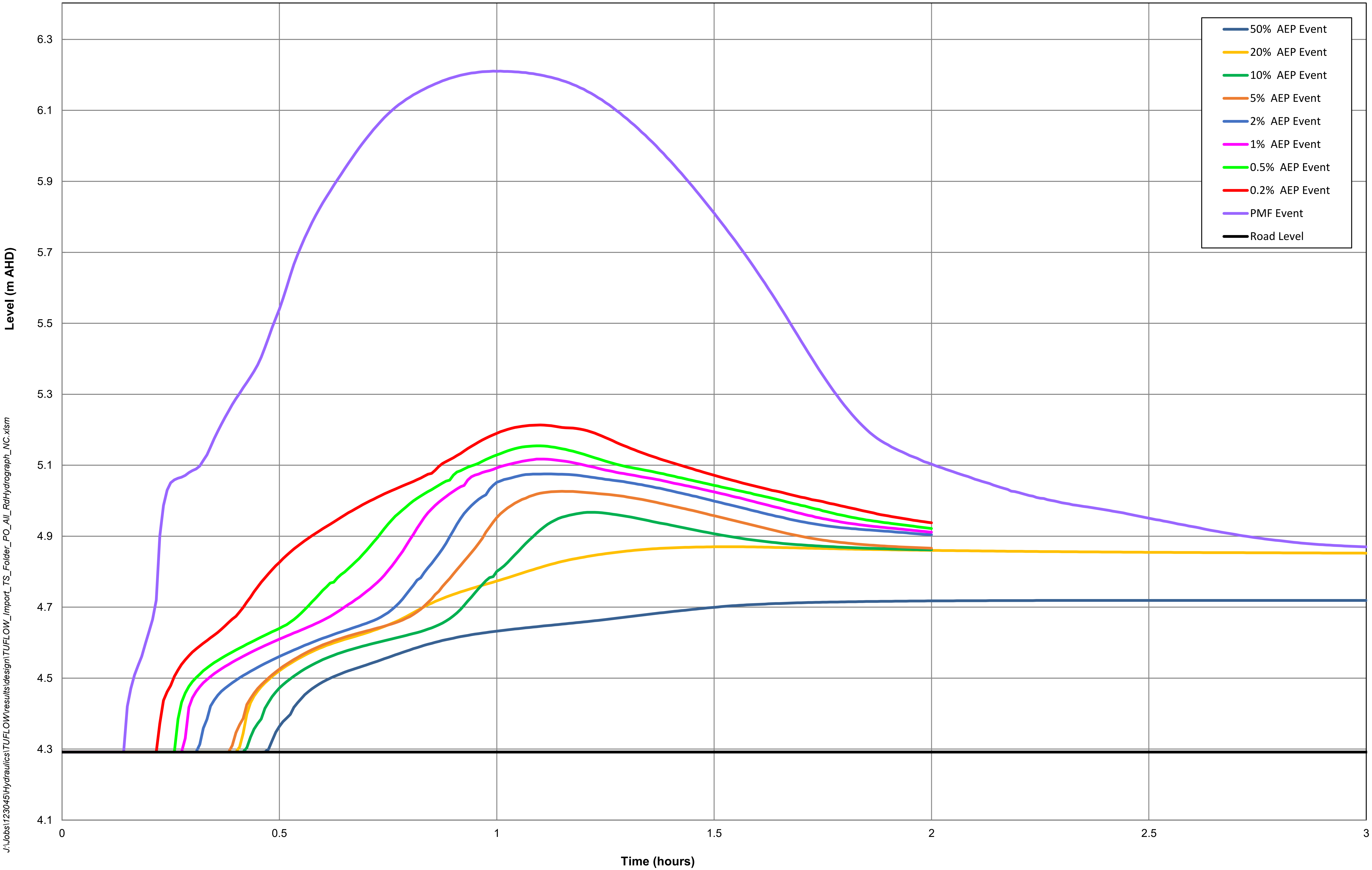

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FIGURE D12

STAGE HYDROGRAPH
V01: MYLES AVENUE AT VERMONT PLACE BRANCH
DESIGN STORMS

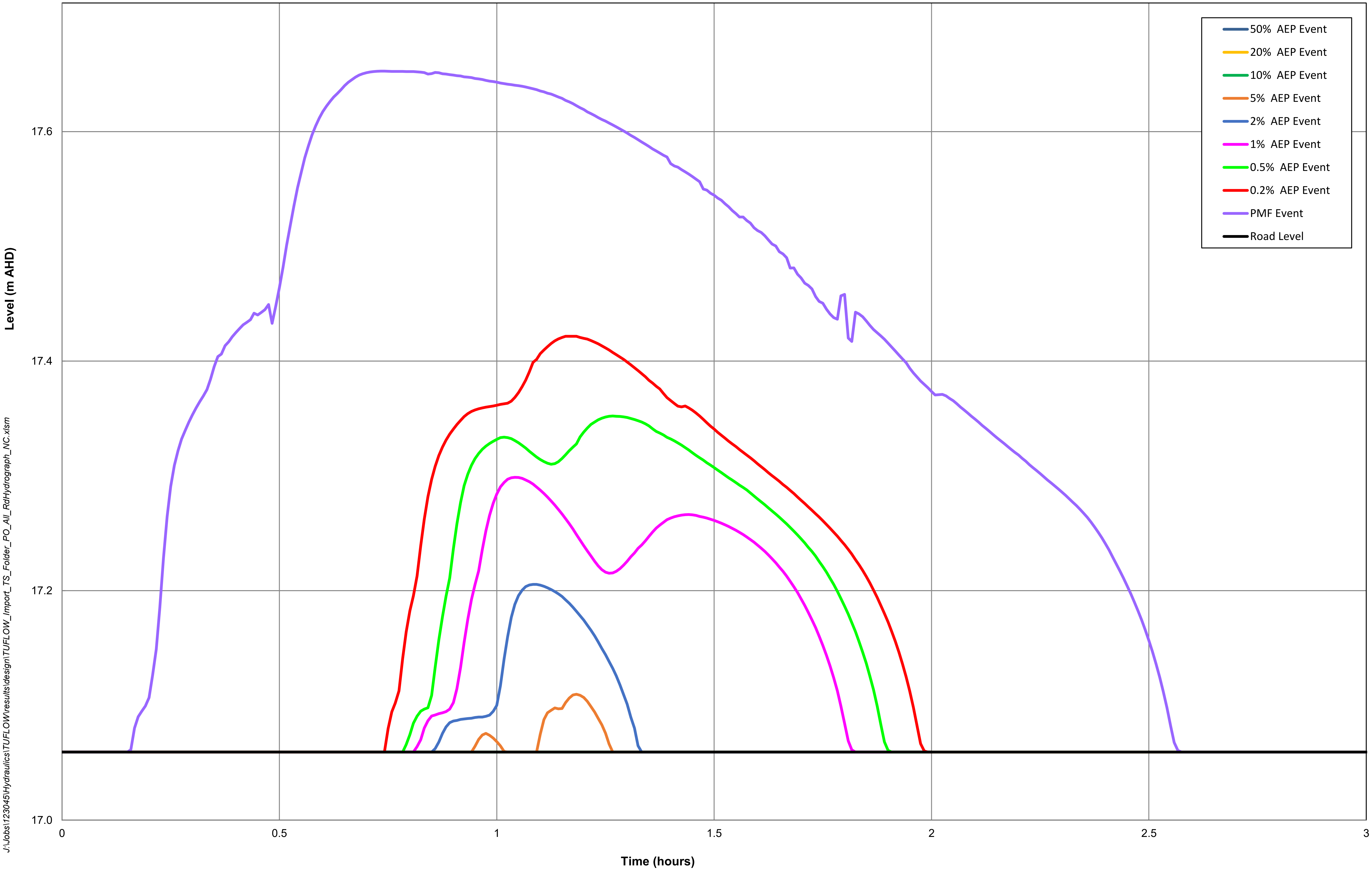

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FIGURE D13

STAGE HYDROGRAPH
K01: NOTT STREET AT KING STREET BRANCH
DESIGN STORMS

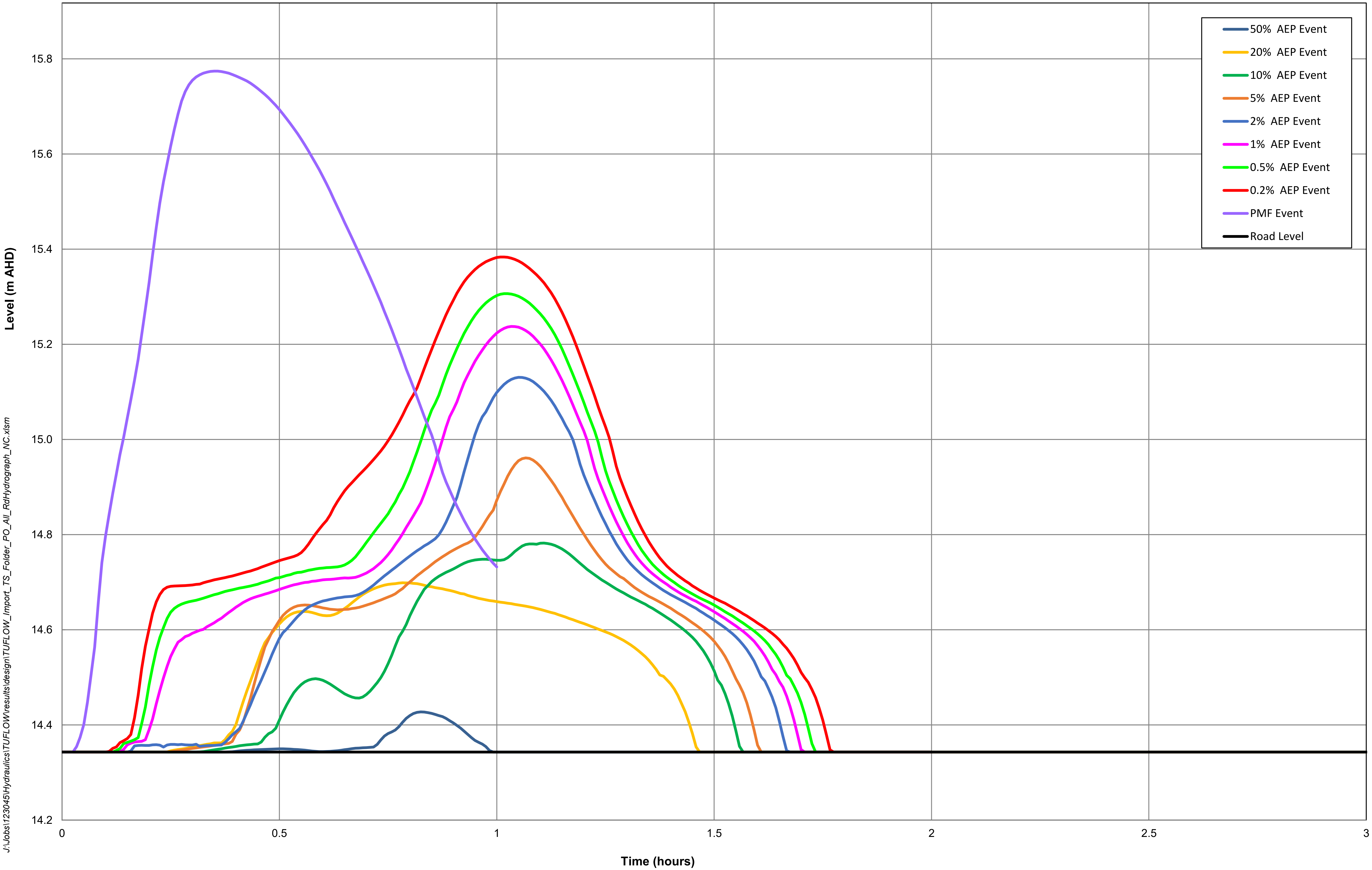

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FIGURE D14

STAGE HYDROGRAPH
K02: YORSTON STREET AT KING STREET BRANCH
DESIGN STORMS

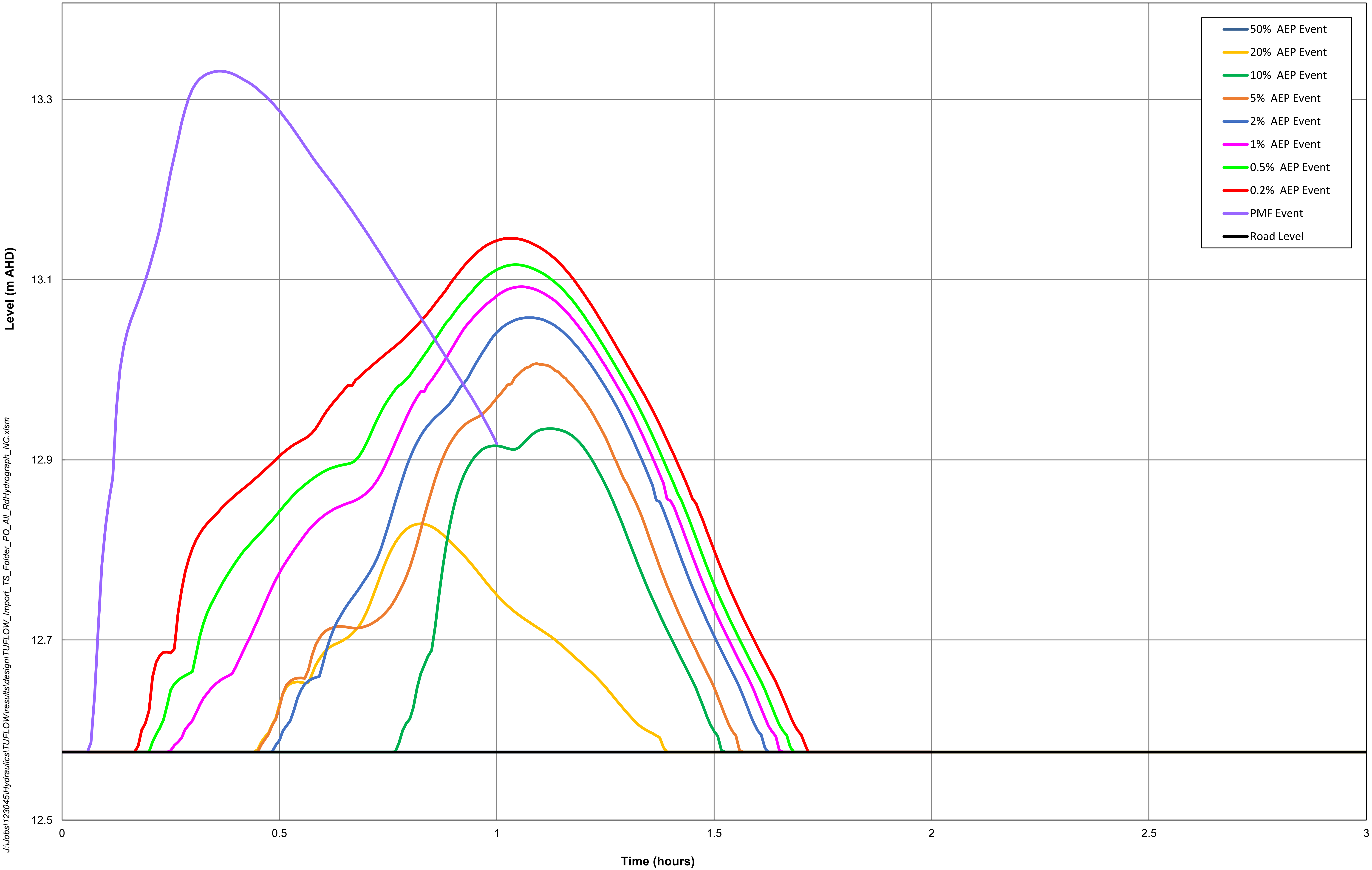

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FIGURE D15

STAGE HYDROGRAPH
K03: QUEEN STREET AT KING STREET BRANCH
DESIGN STORMS

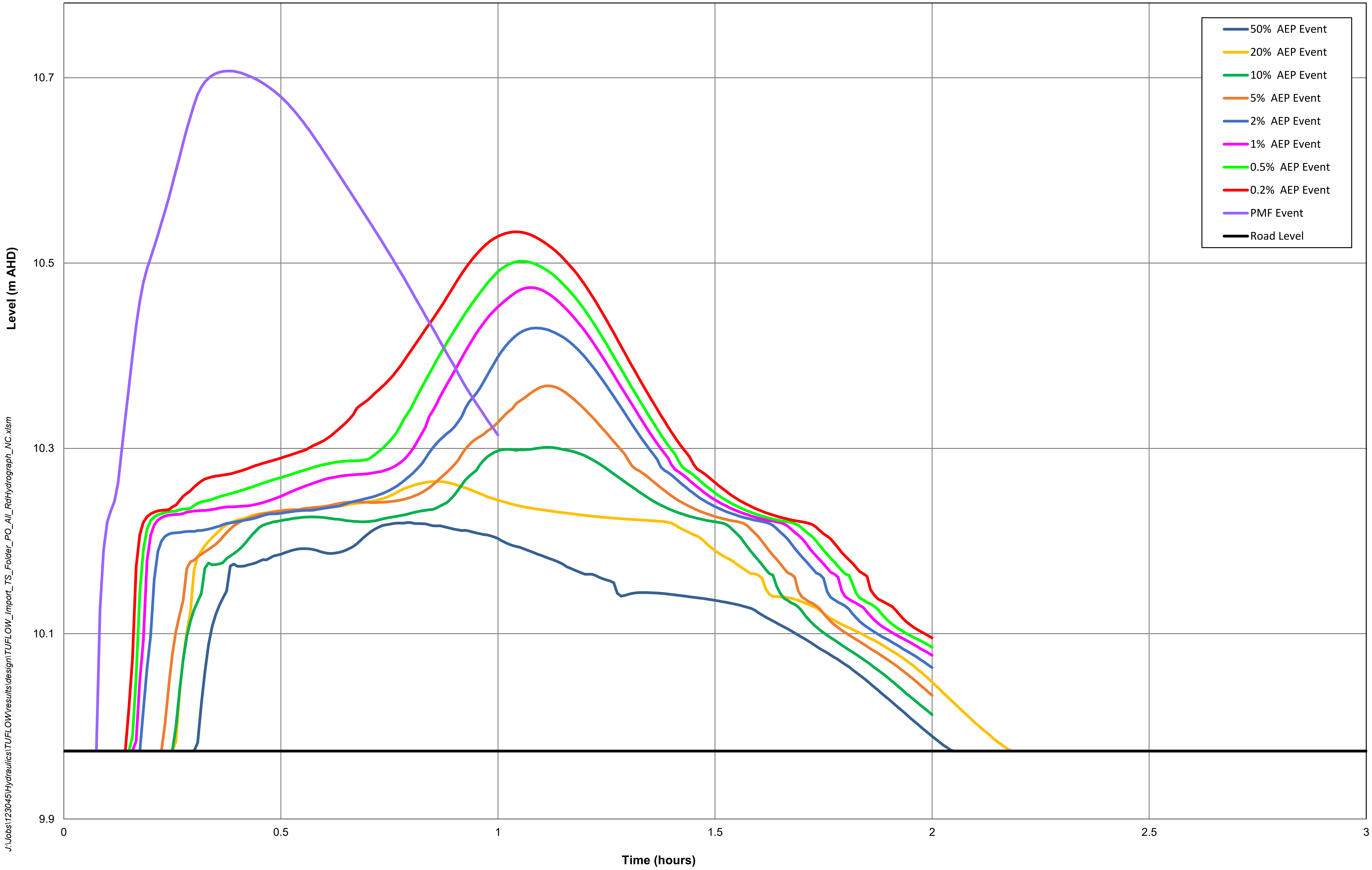

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FIGURE D16

STAGE HYDROGRAPH
K04: KING STREET AT KING STREET BRANCH
DESIGN STORMS

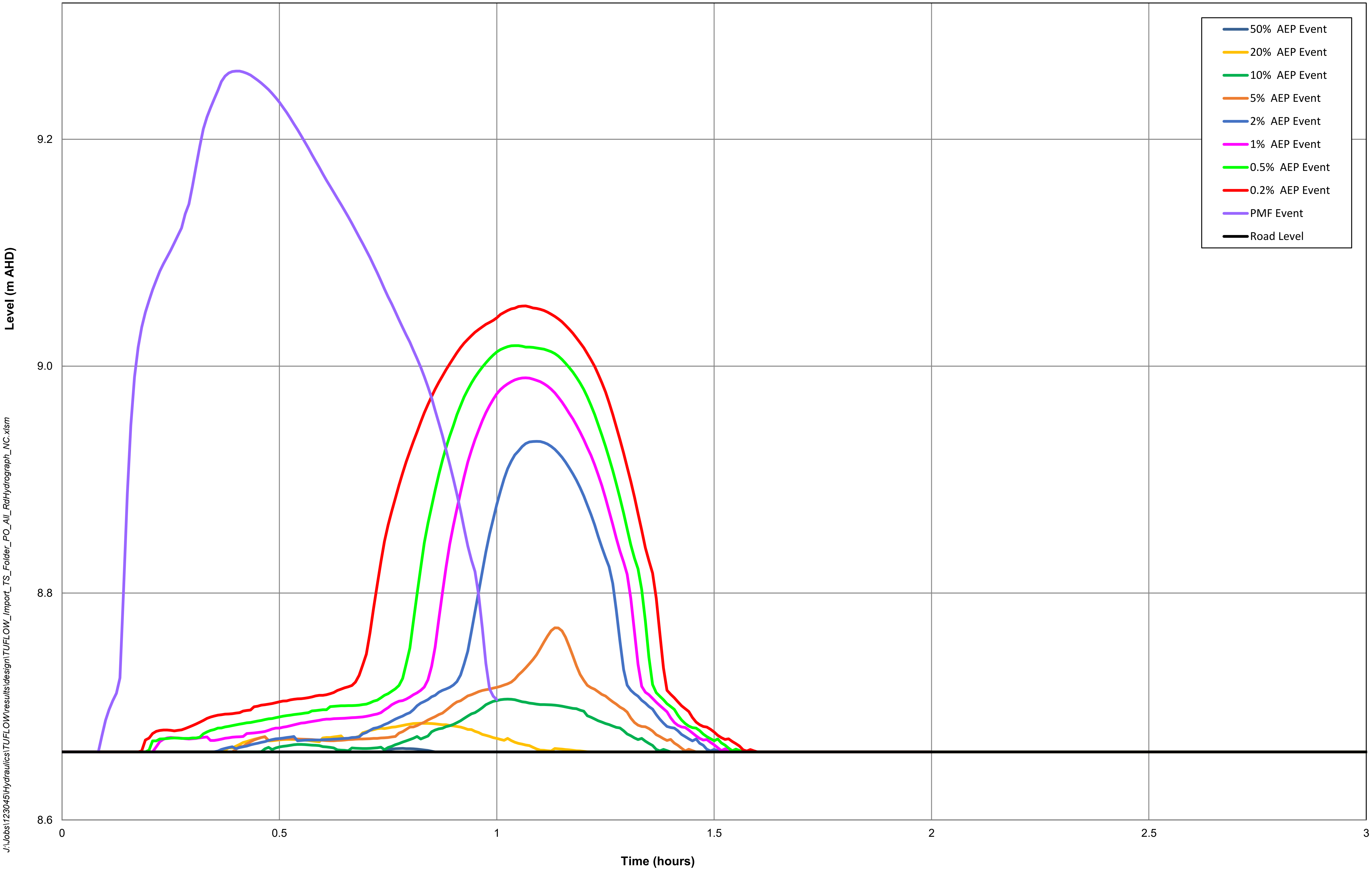

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FIGURE D17

STAGE HYDROGRAPH
N01: HILLSBOROUGH ROAD SERVICE ROAD AT NORTH CREEK
DESIGN STORMS

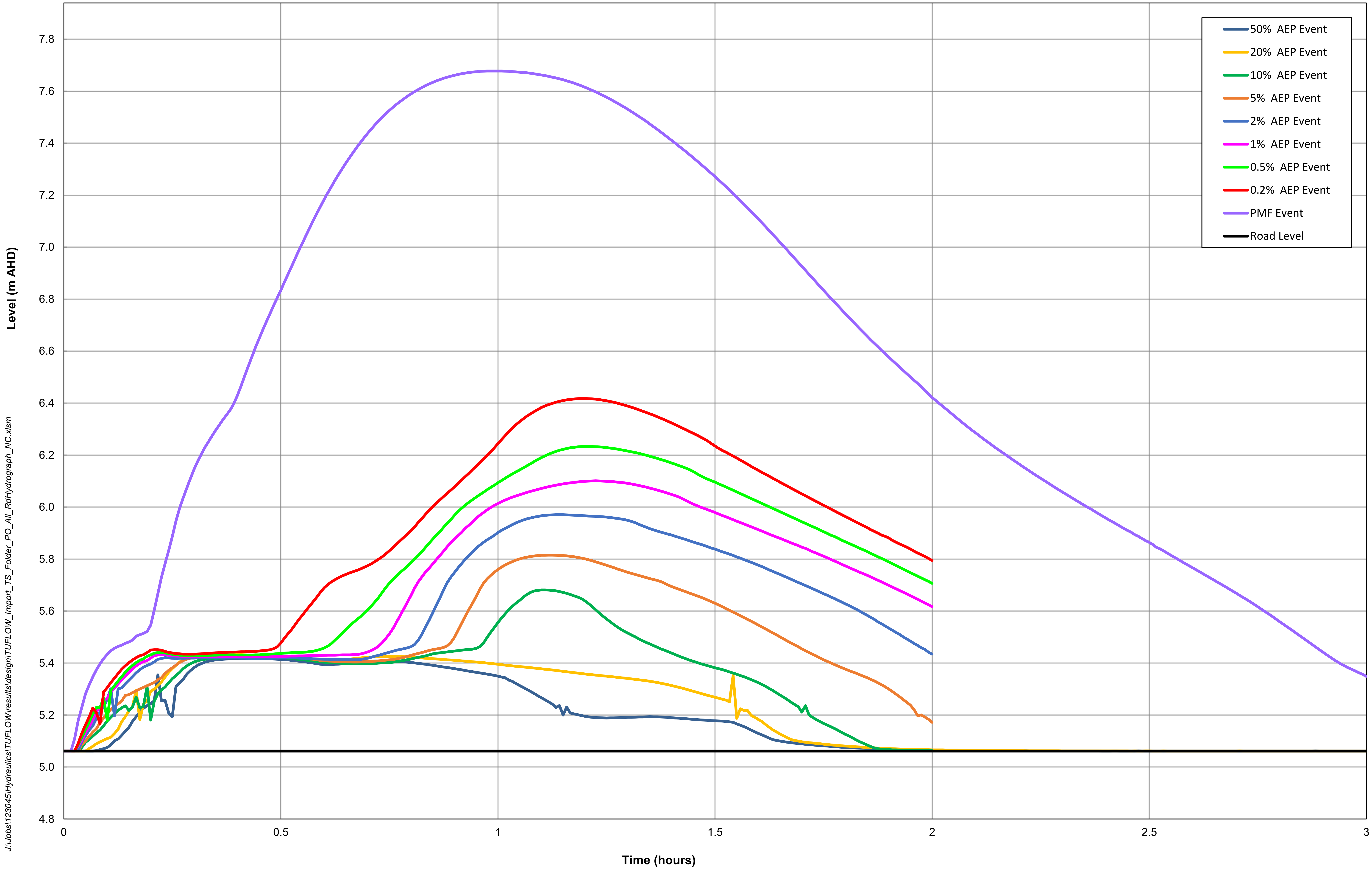

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FIGURE D18

STAGE HYDROGRAPH
N02: KING STREET AT NORTH CREEK
DESIGN STORMS

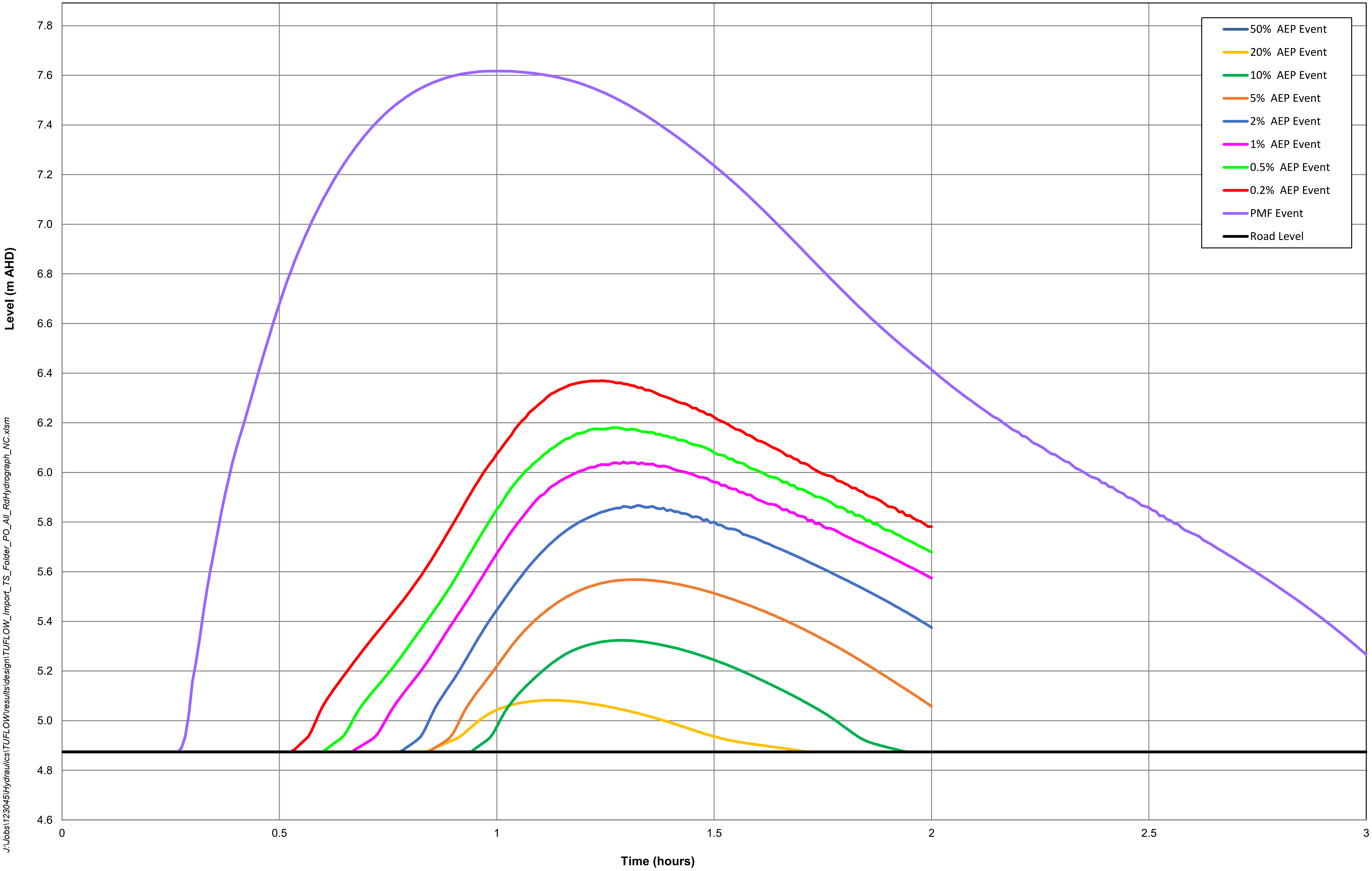

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FIGURE D19

STAGE HYDROGRAPH
N03: WALKER STREET AT NORTH CREEK
DESIGN STORMS

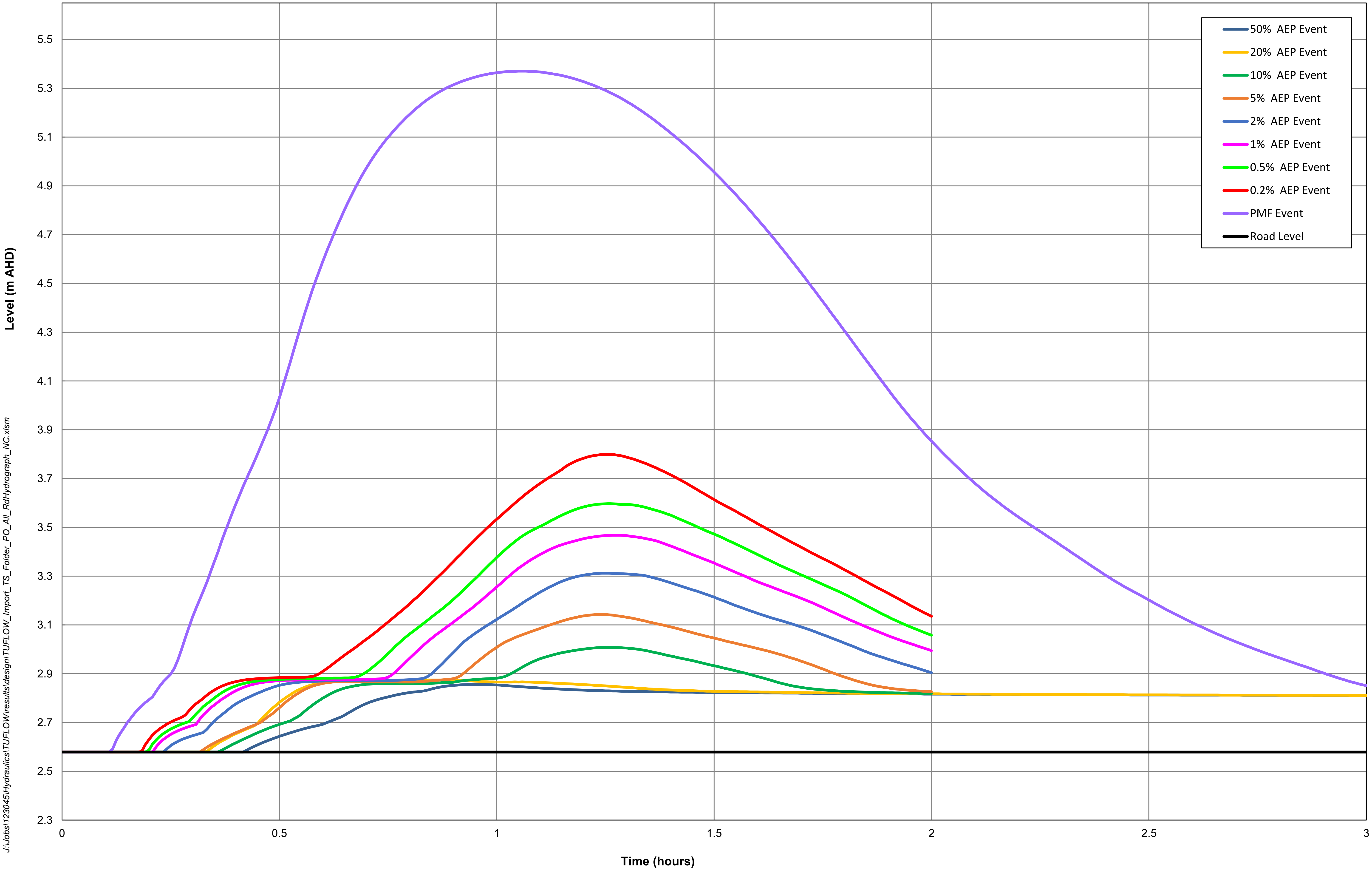

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FIGURE D20

STAGE HYDROGRAPH
N05: THE ESPLANADE AT NORTH CREEK
DESIGN STORMS

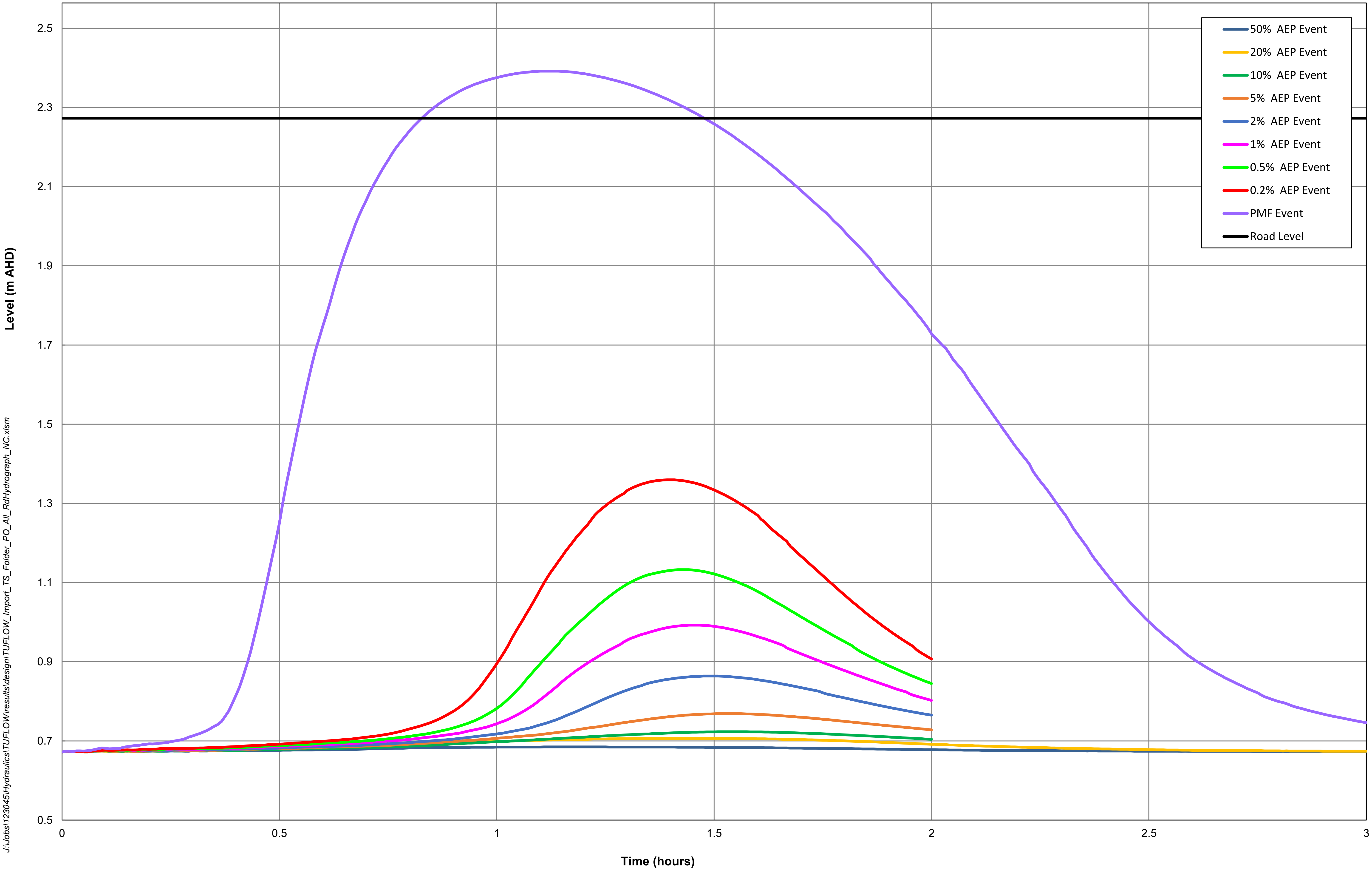

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FIGURE D21

STAGE HYDROGRAPH
R01: HUGHES AVENUE
DESIGN STORMS

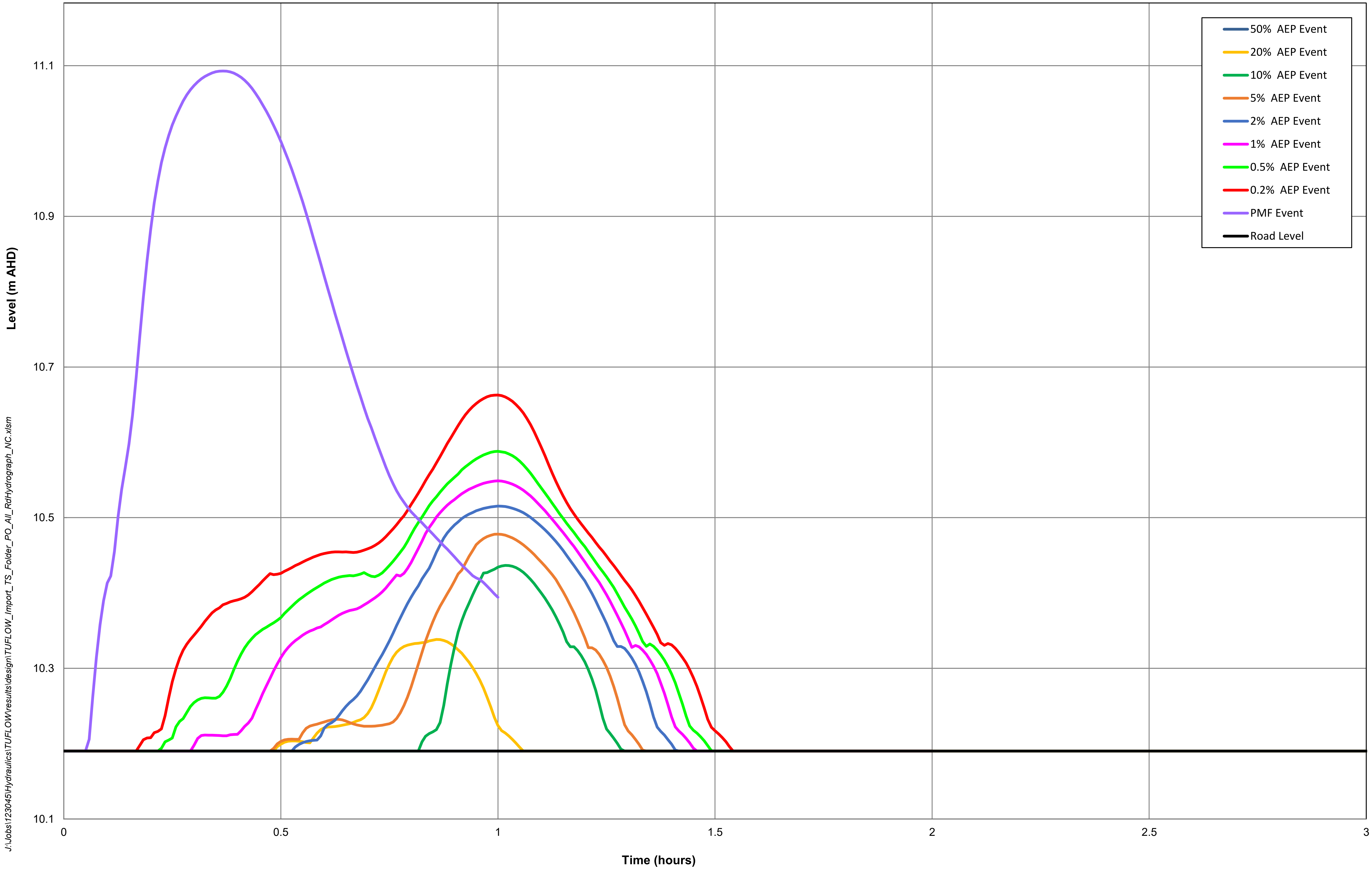

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FIGURE D22

STAGE HYDROGRAPH
R02: MACQUARIE ROAD
DESIGN STORMS

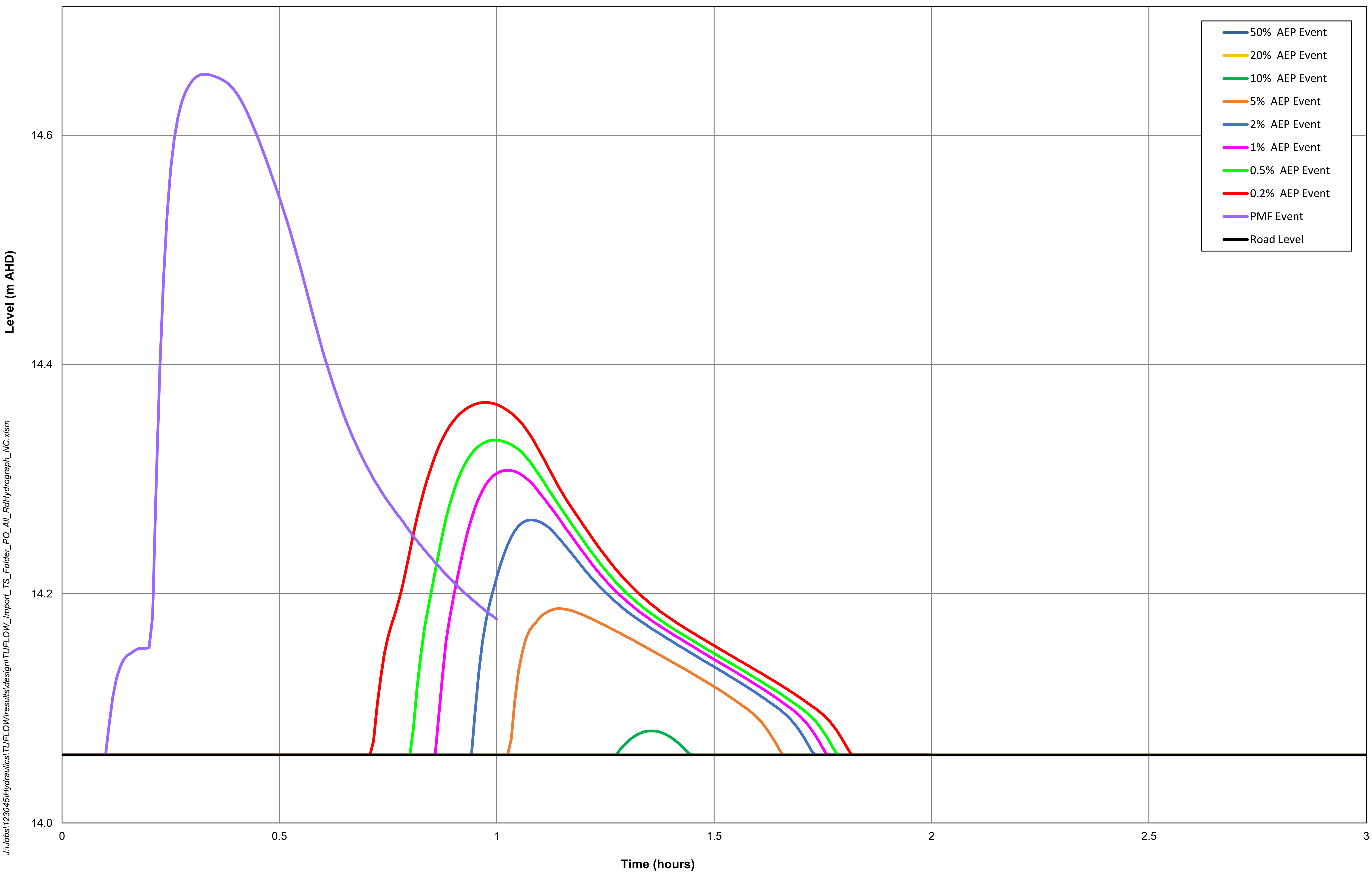

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FIGURE D23

STAGE HYDROGRAPH
R03: MYLES AVENUE (NORTH)
DESIGN STORMS

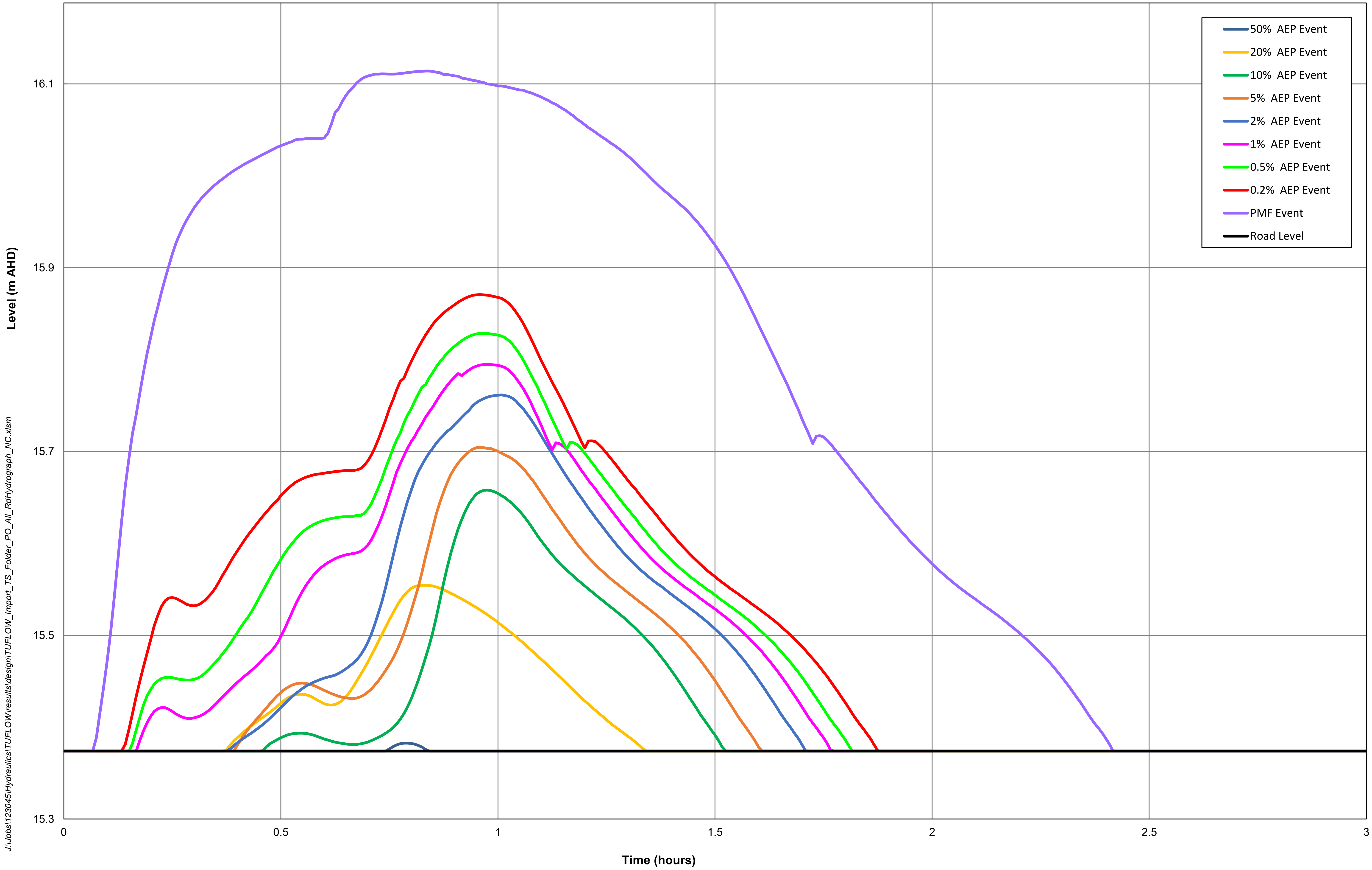

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FIGURE D24

STAGE HYDROGRAPH
R04: NEBRASKA CLOSE
DESIGN STORMS

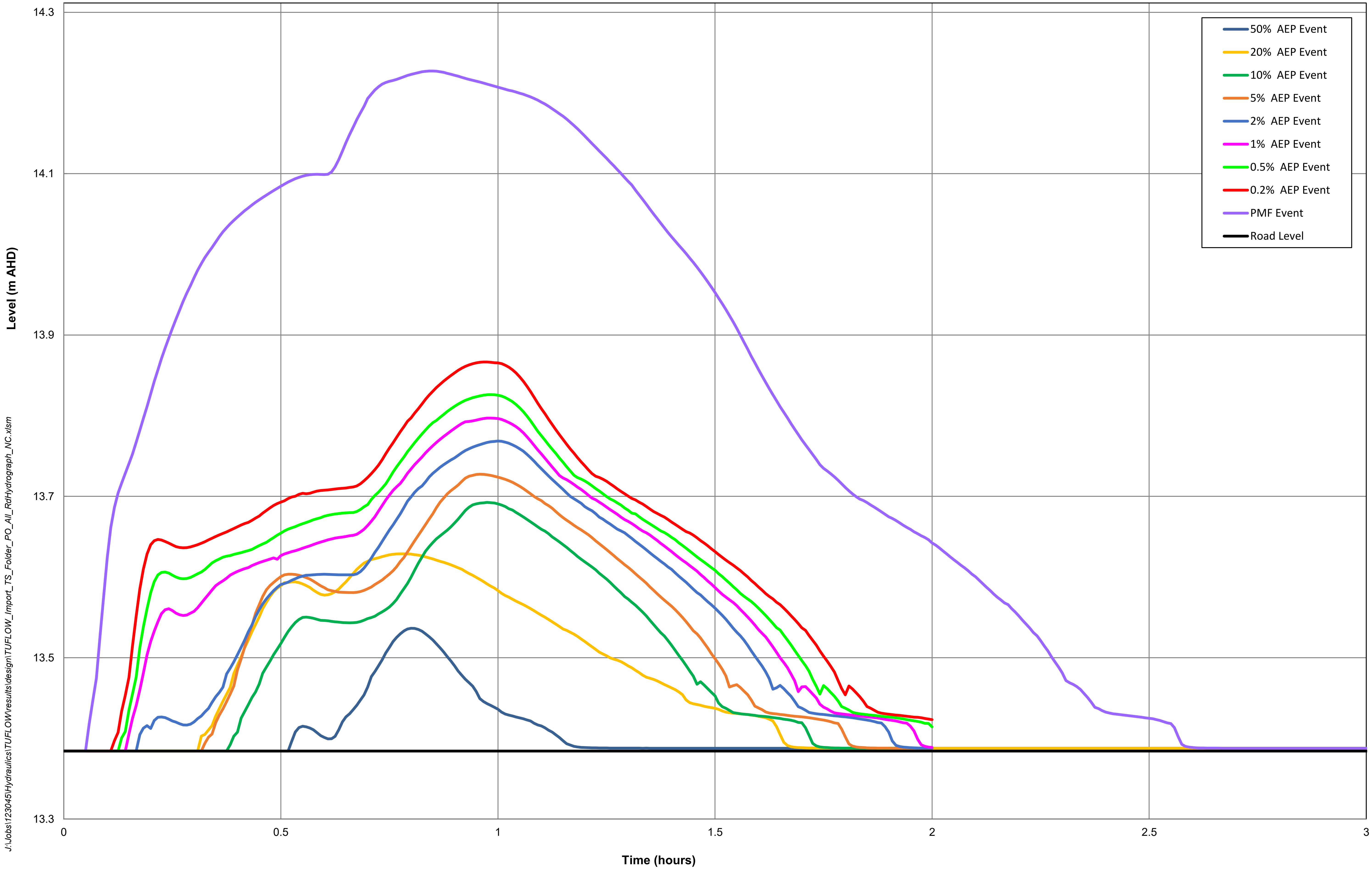

FIGURE D25

STAGE HYDROGRAPH
R05: COLORADO CLOSE
DESIGN STORMS

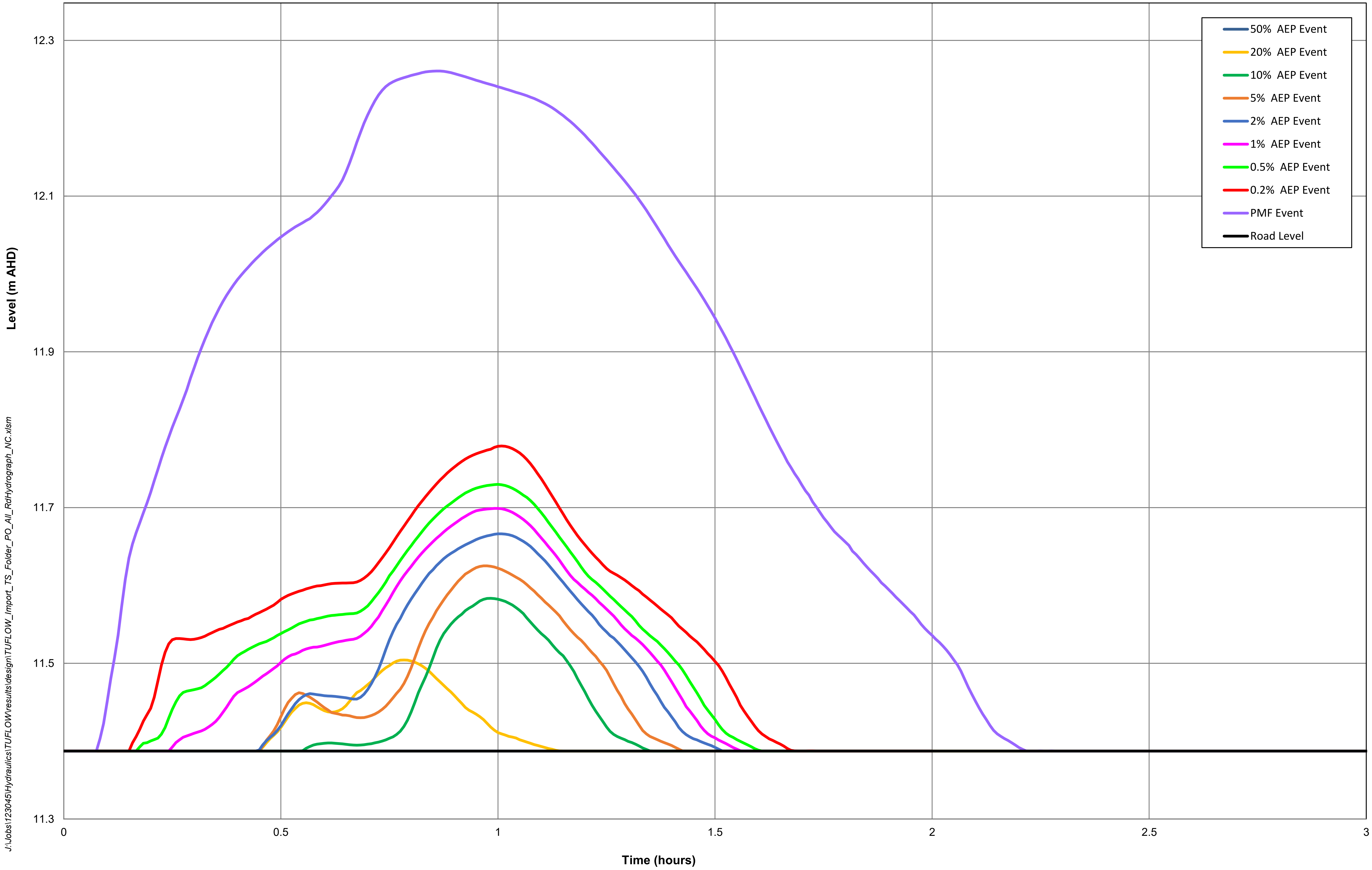

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FIGURE D26

STAGE HYDROGRAPH
R06: NEW YORK AVENUE
DESIGN STORMS

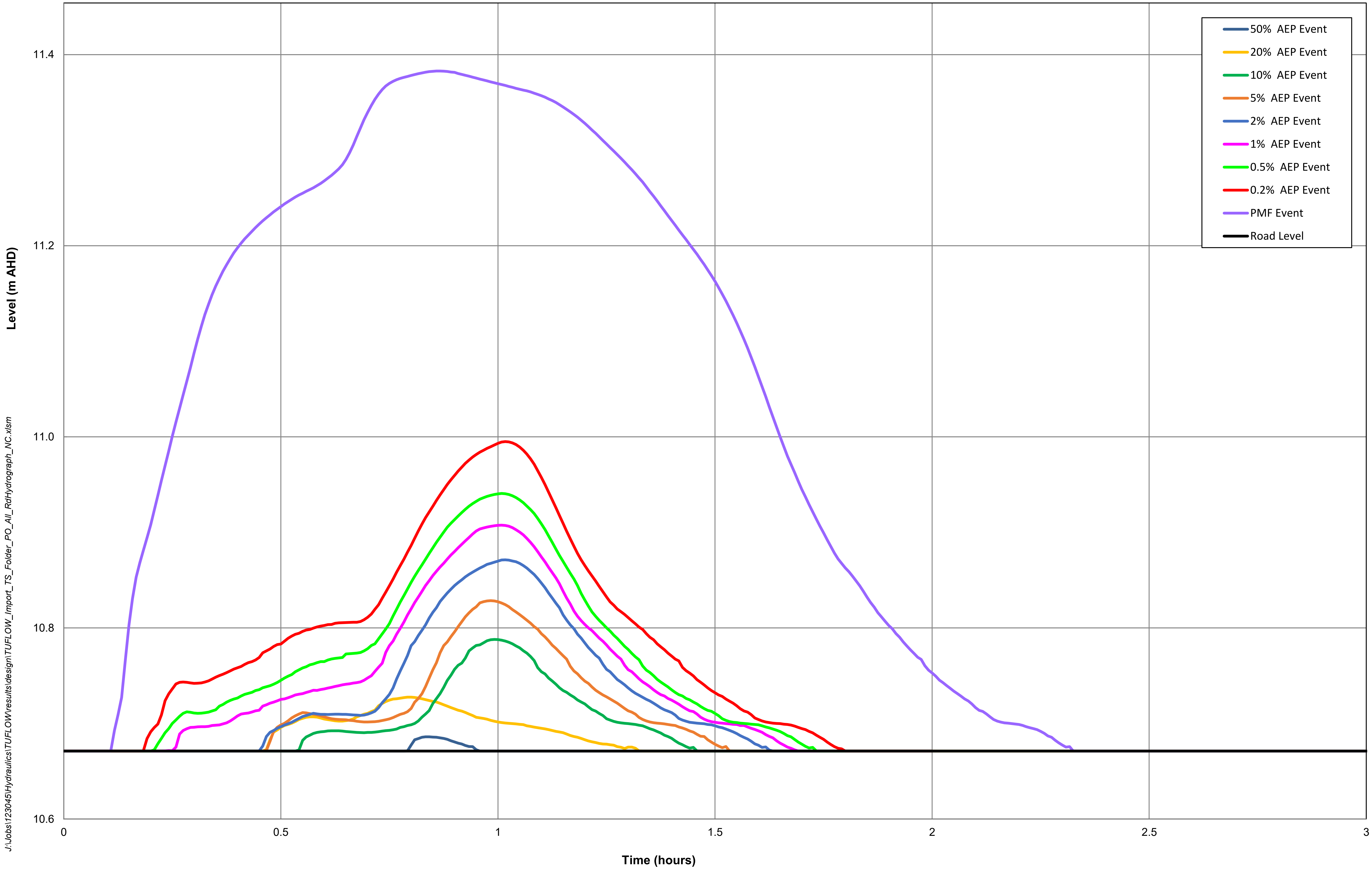

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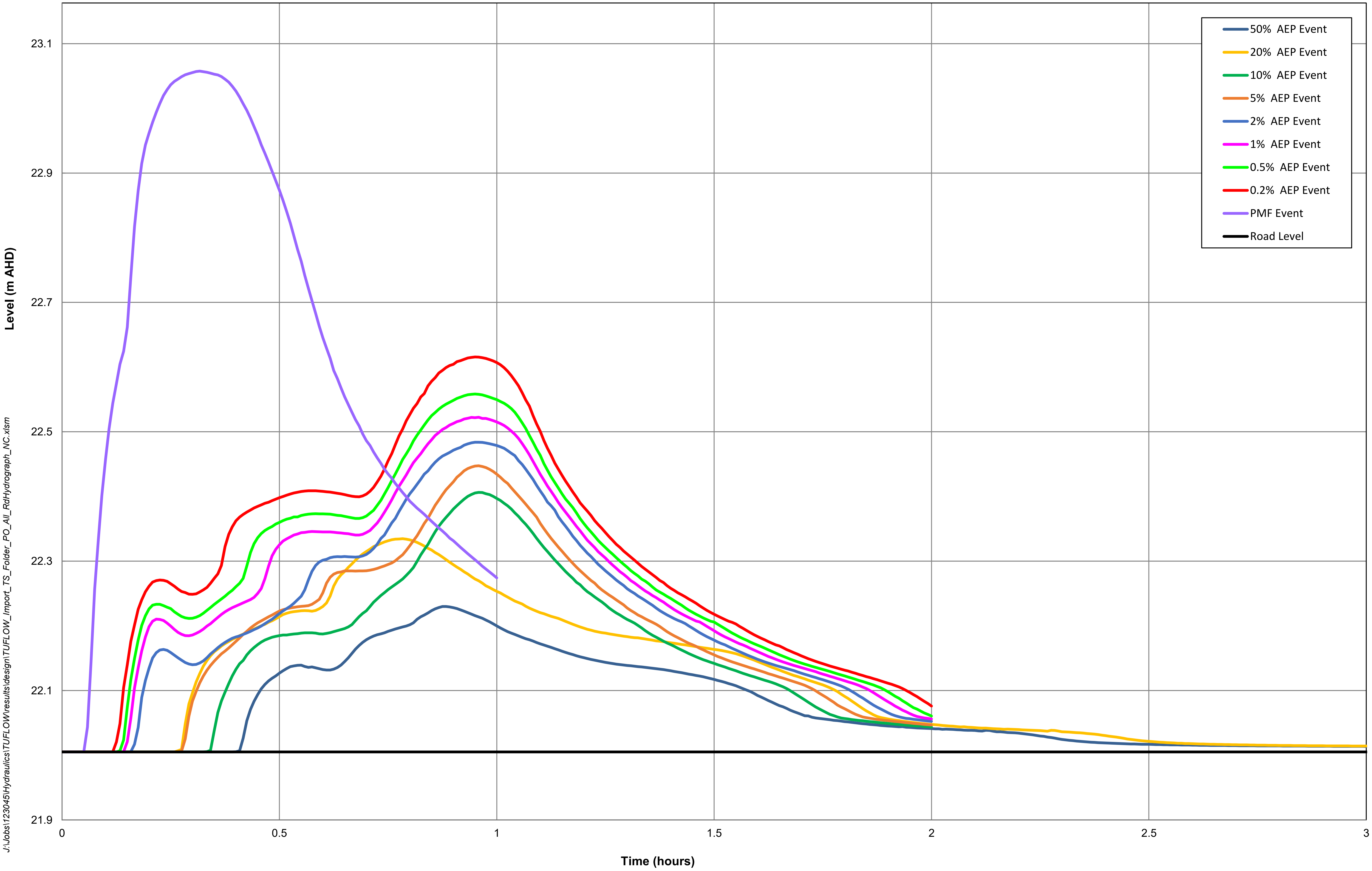
FIGURE D27

STAGE HYDROGRAPH
R07: WILTON CLOSE
DESIGN STORMS



J:\Jobs\123045\Hydraulics\TUFLOW\results\design\TUFLOW_Import_TS_Folder_PO_All_RdHydrograph_NC.xlsm

FIGURE D28

STAGE HYDROGRAPH
R08: MARGARET STREET
DESIGN STORMS

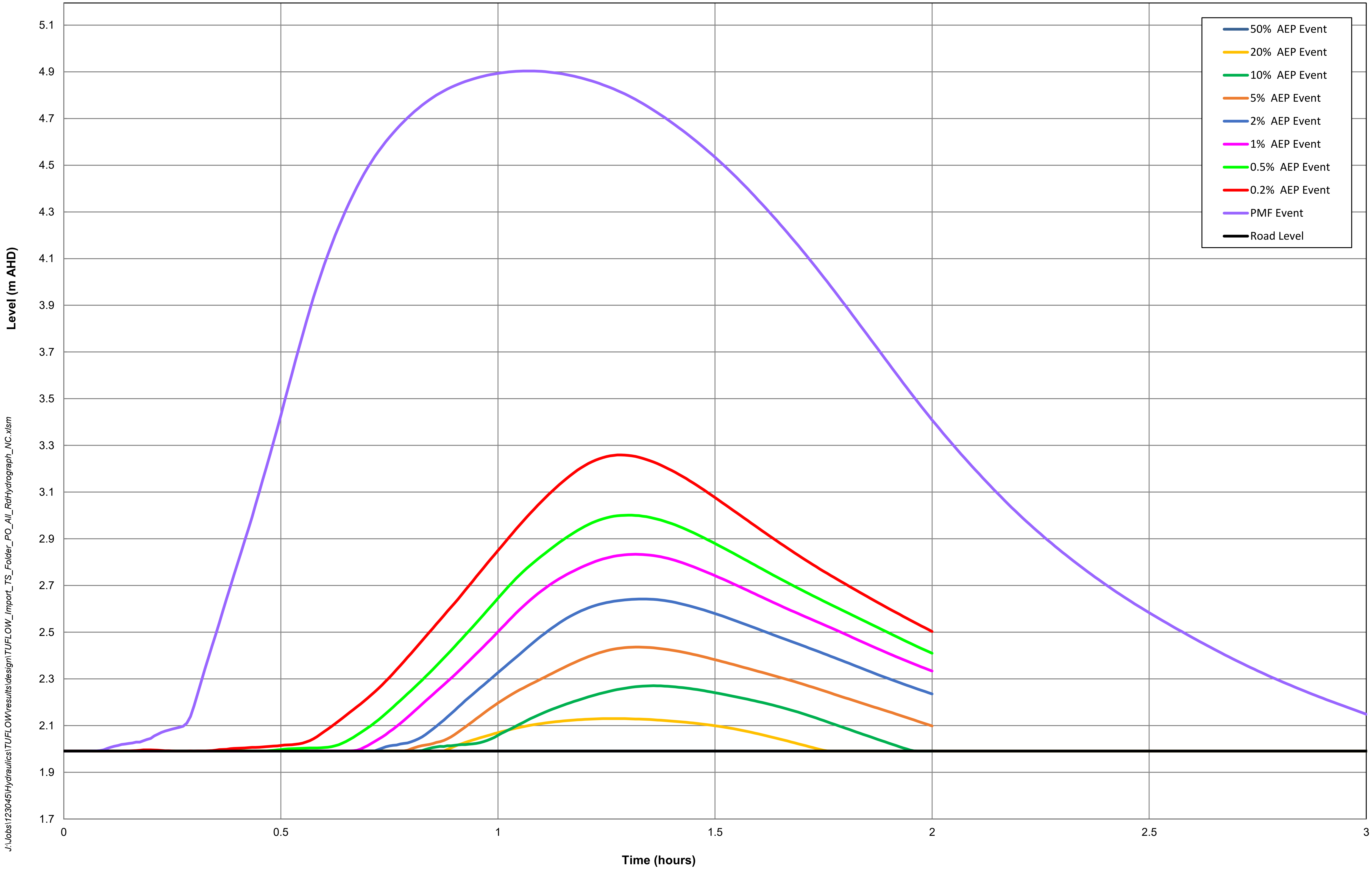


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FIGURE D29

STAGE HYDROGRAPH
R09: MARTIN STREET (EAST)
DESIGN STORMS

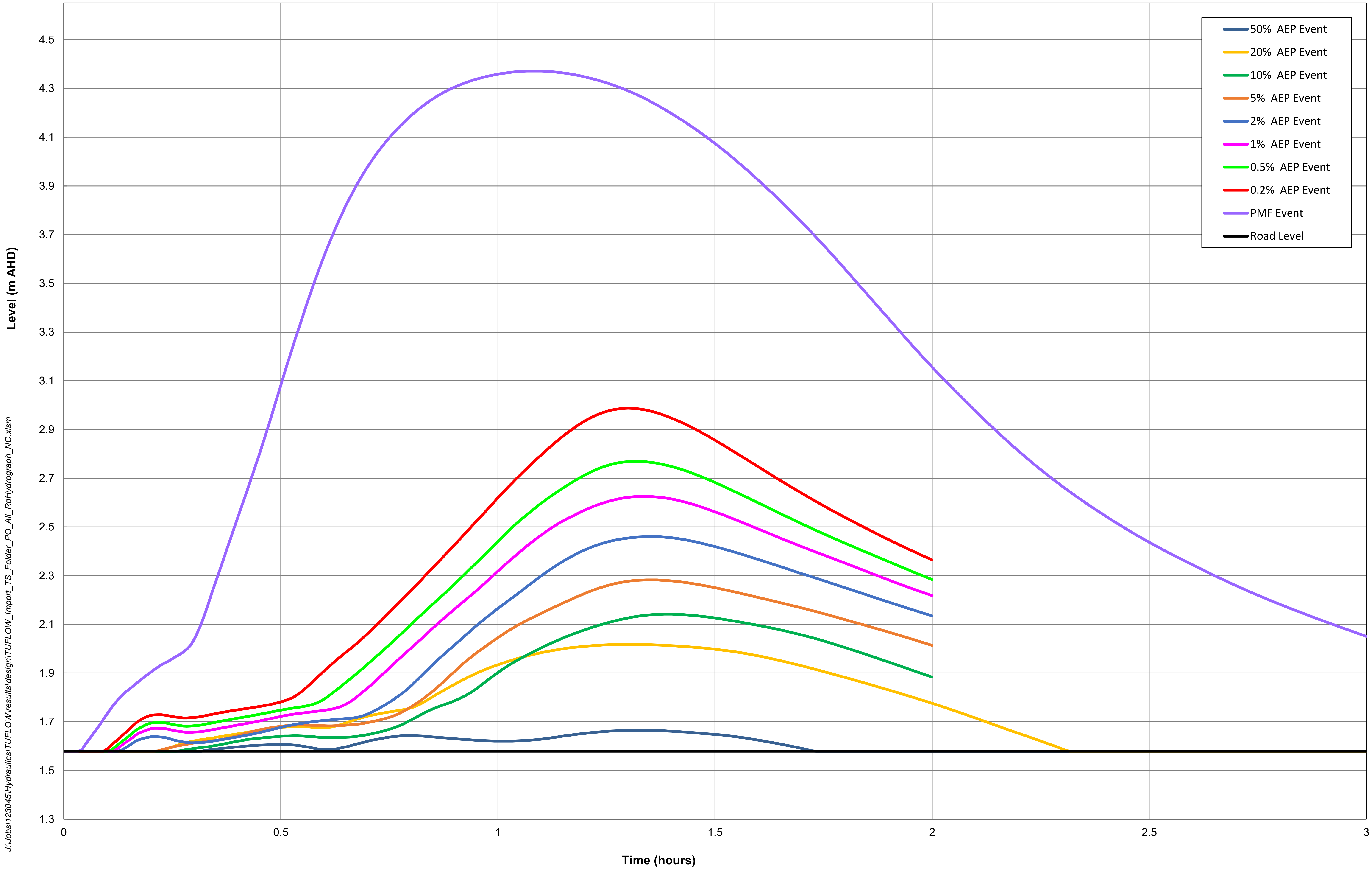

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FIGURE D30

STAGE HYDROGRAPH
R10: ALBERT STREET (WEST)
DESIGN STORMS

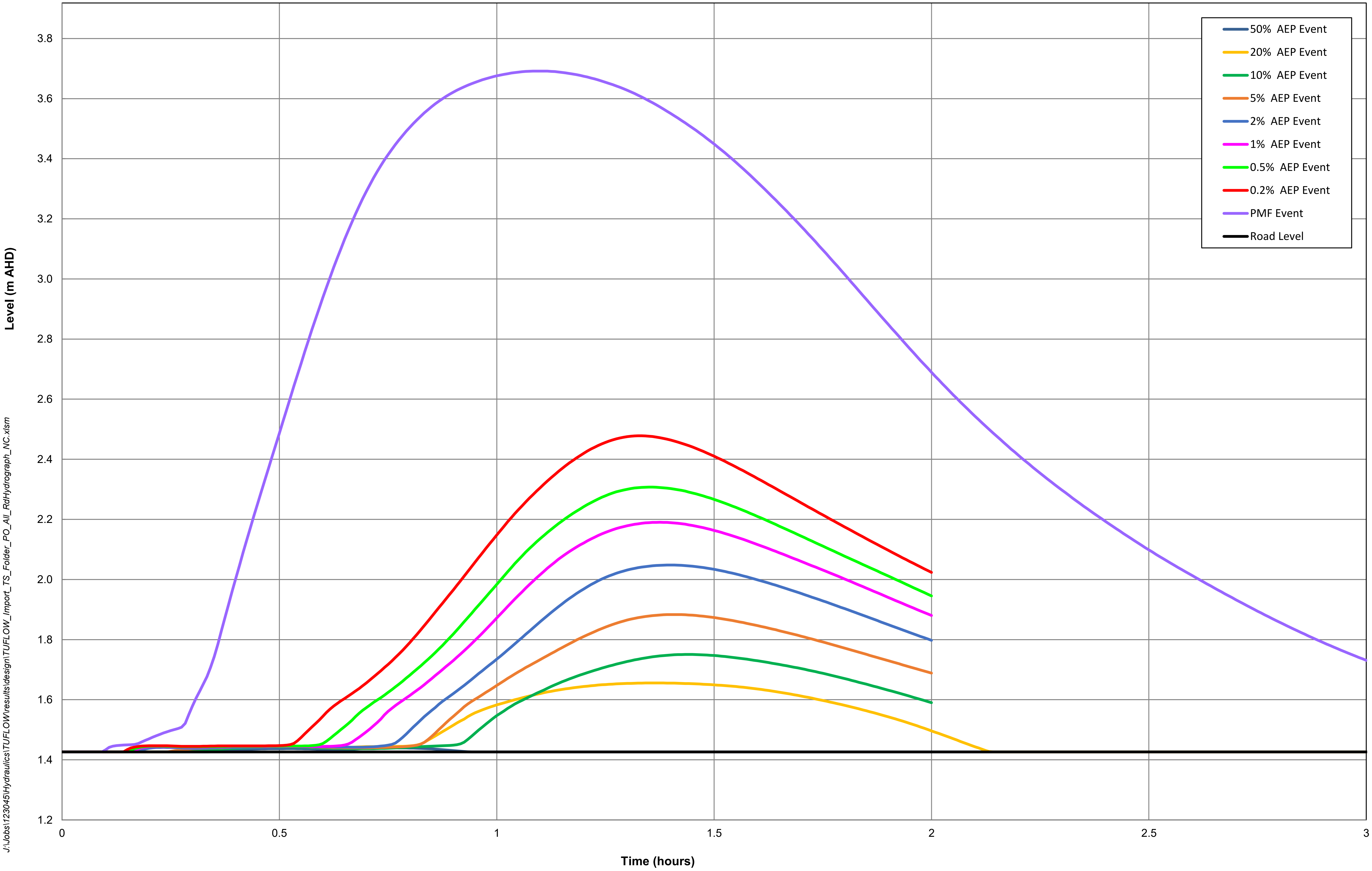

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FIGURE D31

STAGE HYDROGRAPH
R11: ALBERT STREET (EAST)
DESIGN STORMS

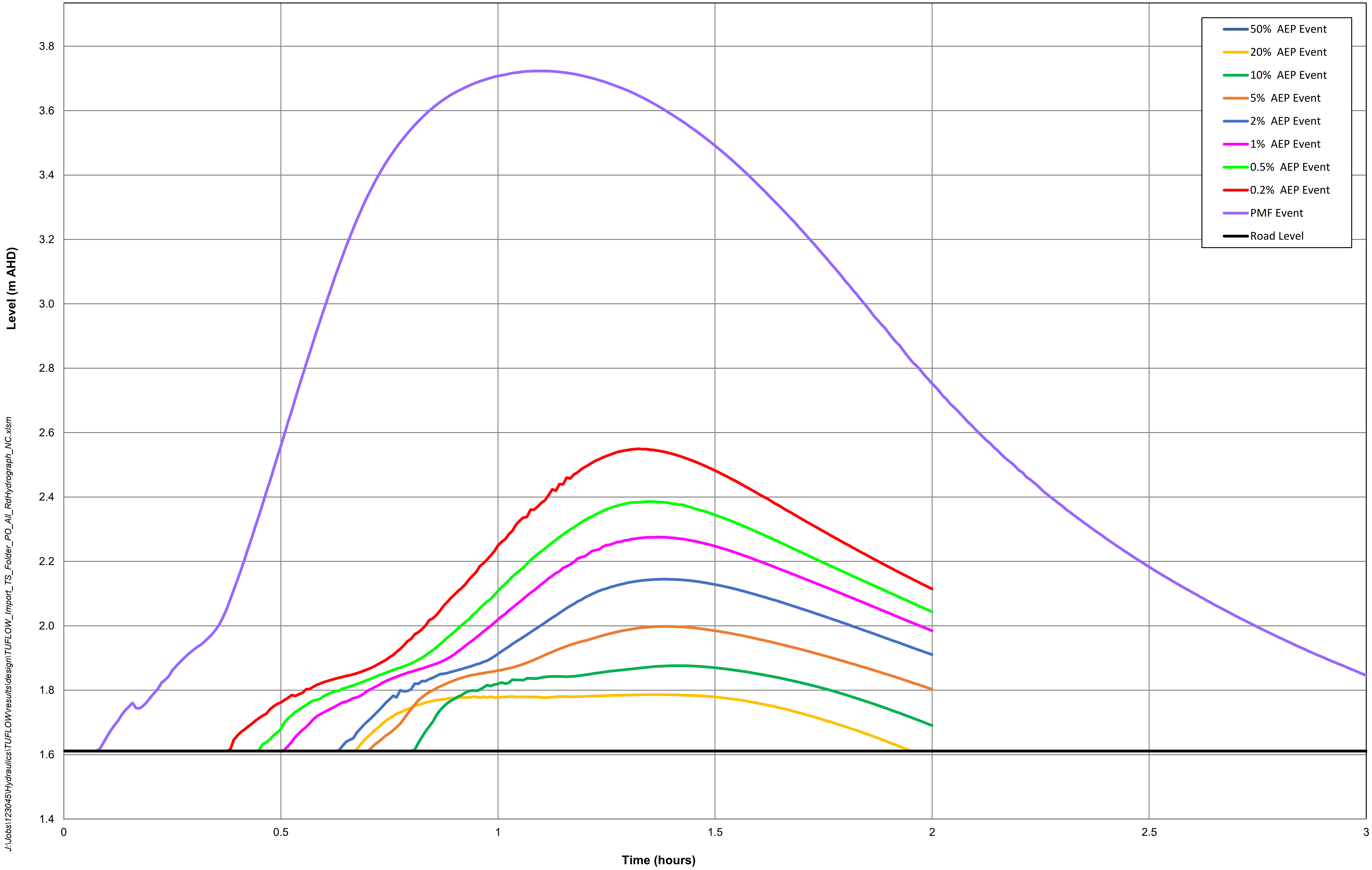

FIGURE D32

STAGE HYDROGRAPH
R12: CHARLES STREET (WEST)
DESIGN STORMS

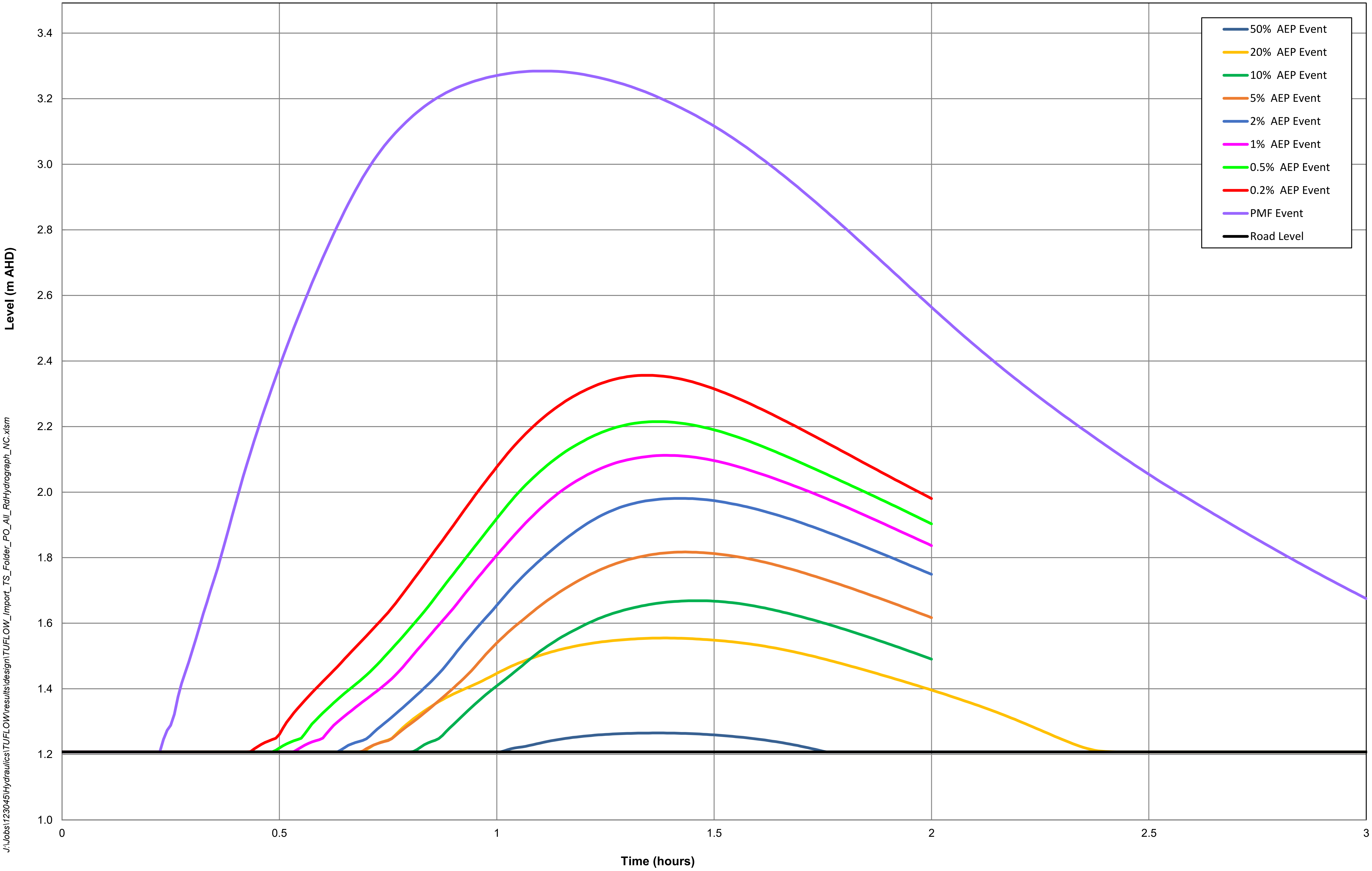

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FIGURE D33
STAGE HYDROGRAPH
R13: CHARLES STREET (EAST)
DESIGN STORMS

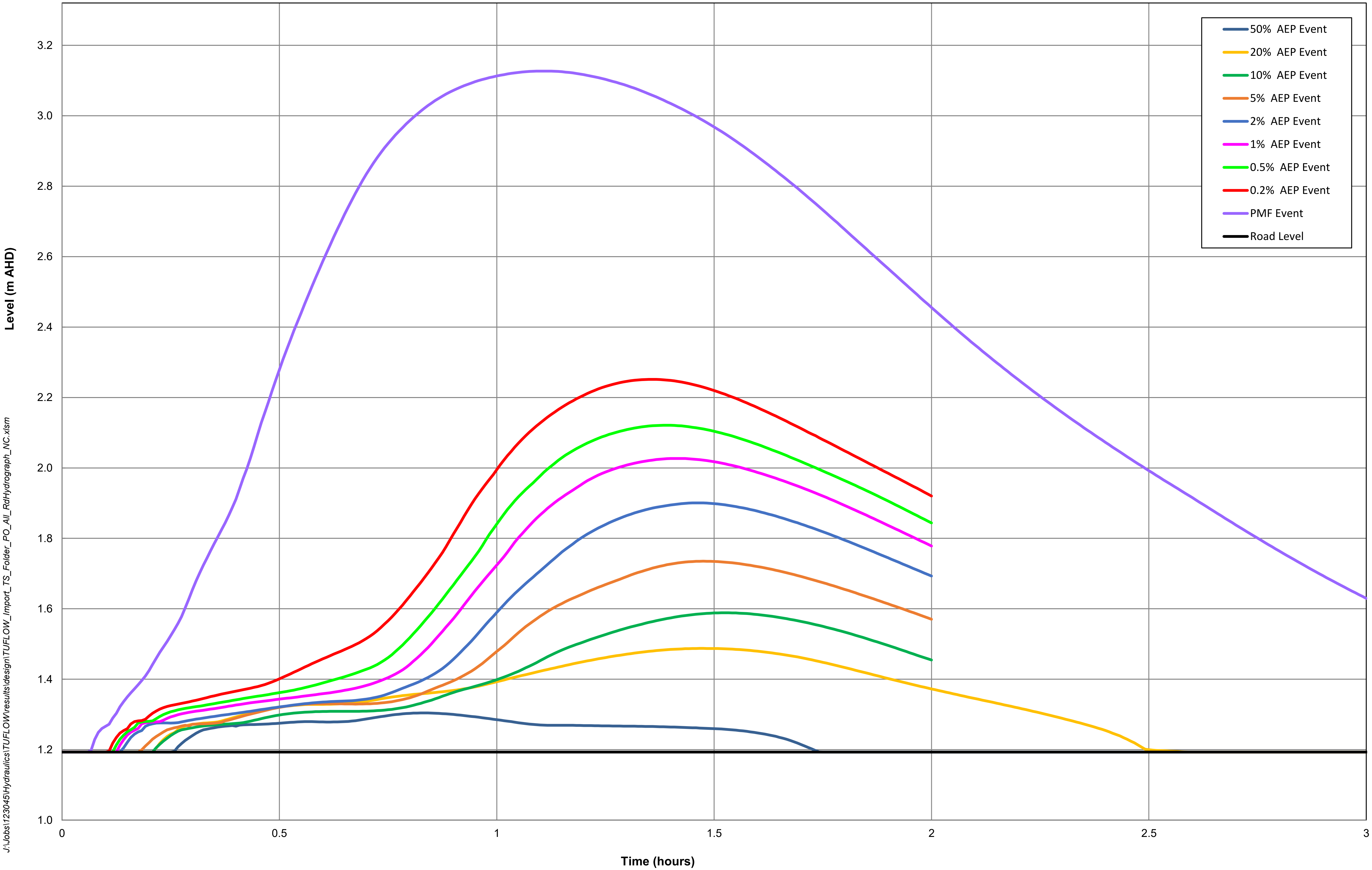

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FIGURE D34

STAGE HYDROGRAPH
R14: JOHN STREET
DESIGN STORMS

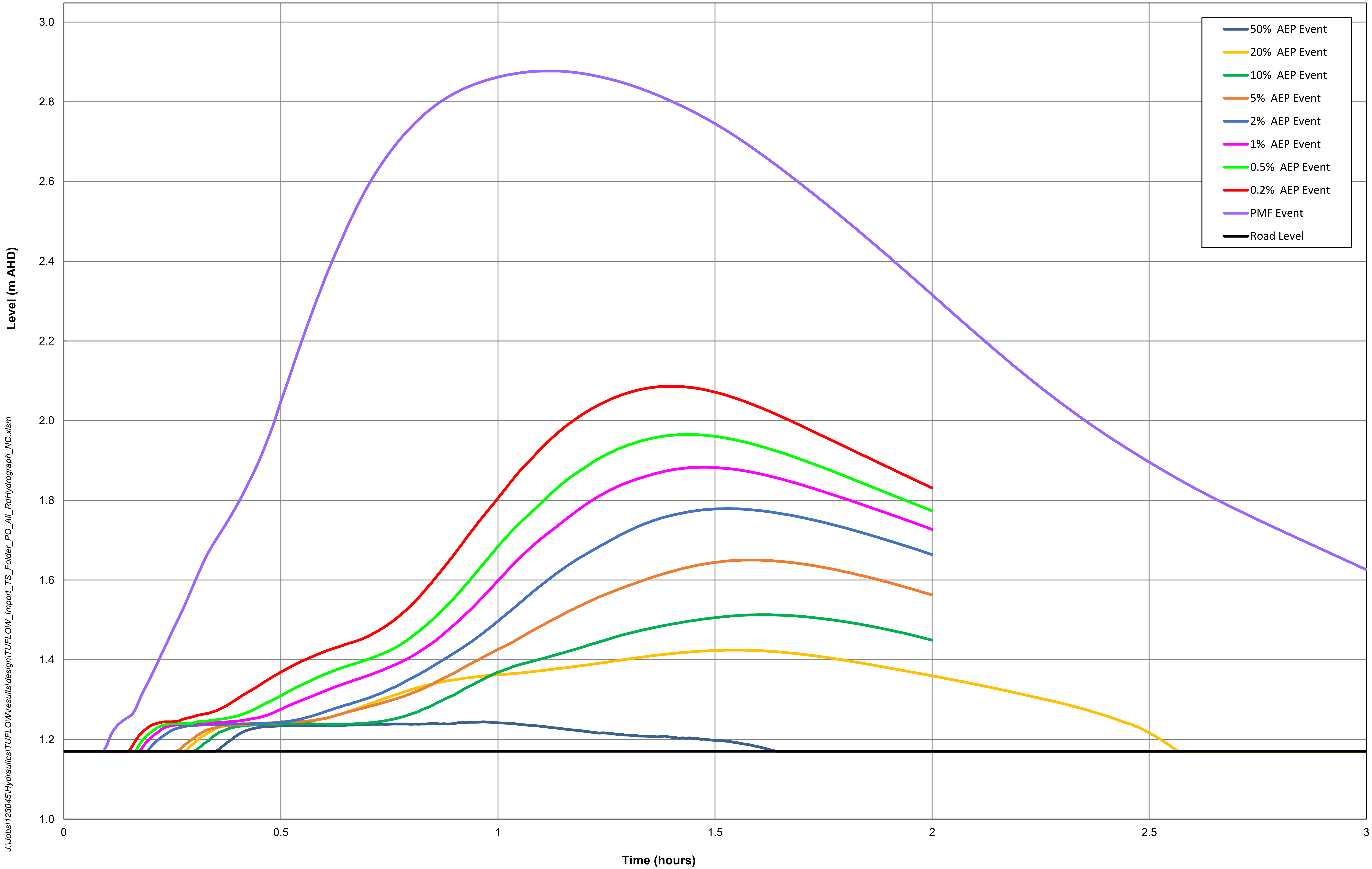

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FIGURE D35

STAGE HYDROGRAPH
R15: THE ESPLANADE (WEST)
DESIGN STORMS

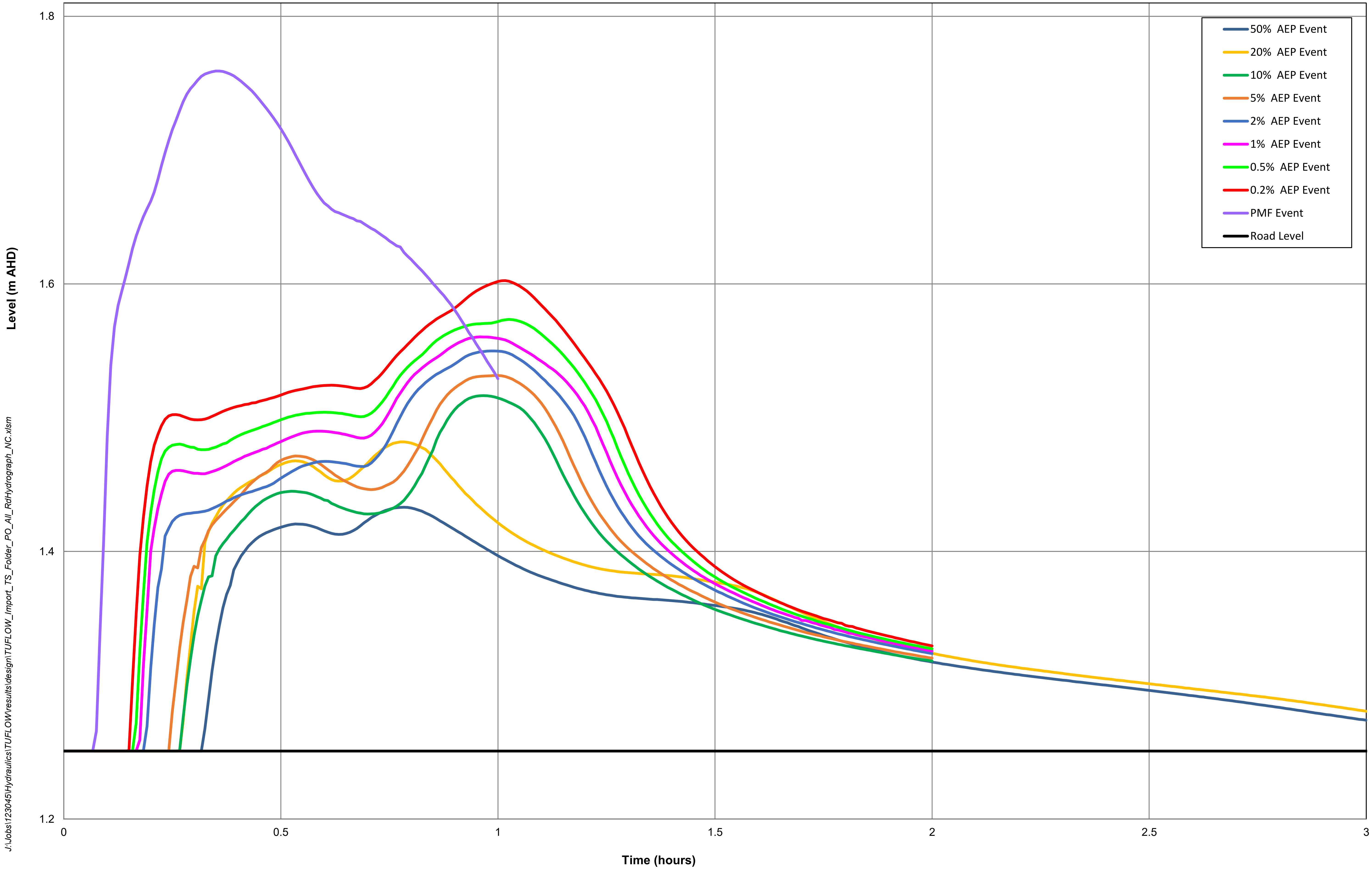

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FIGURE D36

STAGE HYDROGRAPH
R16: THE ESPLANADE (EAST)
DESIGN STORMS

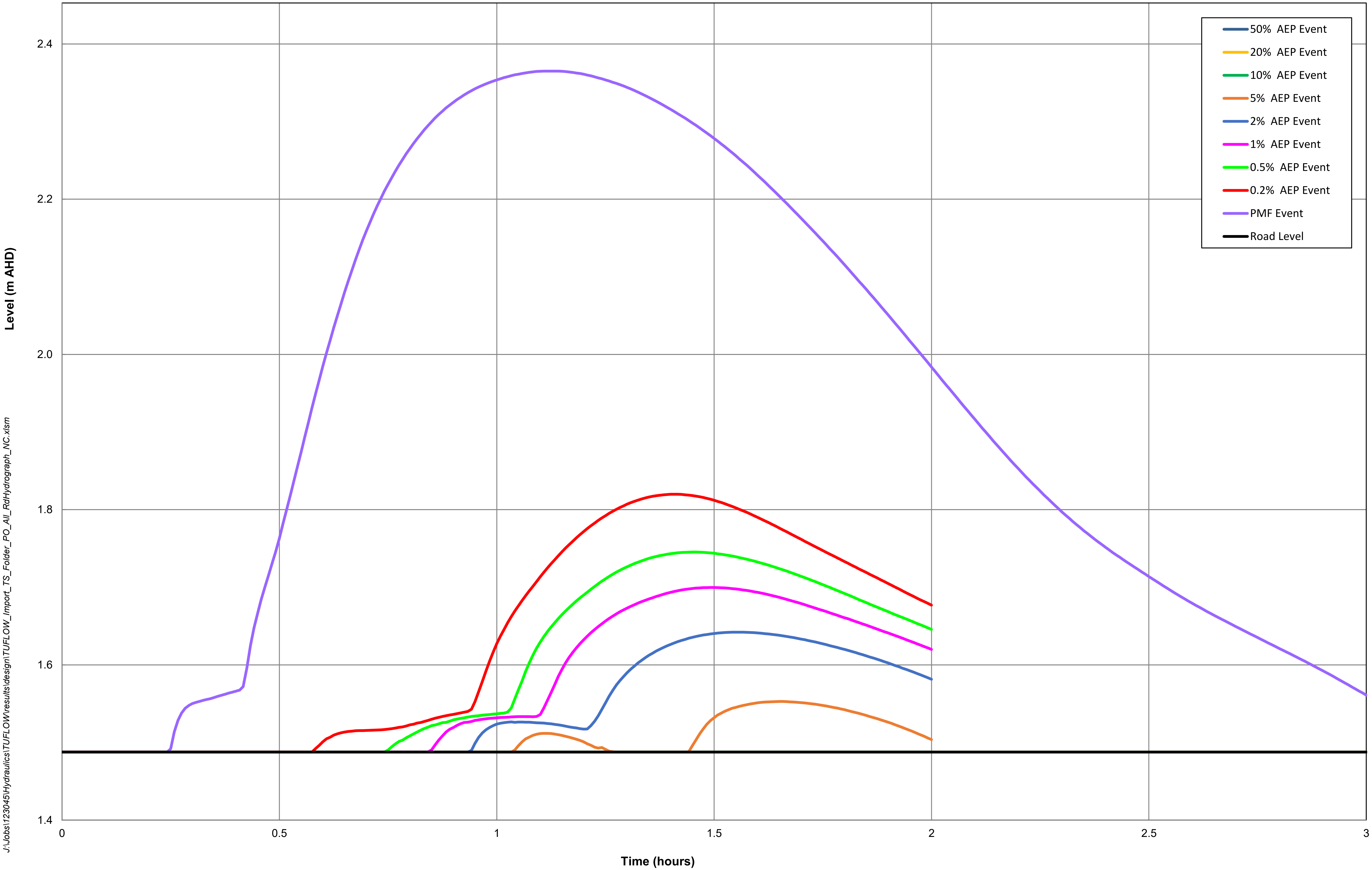

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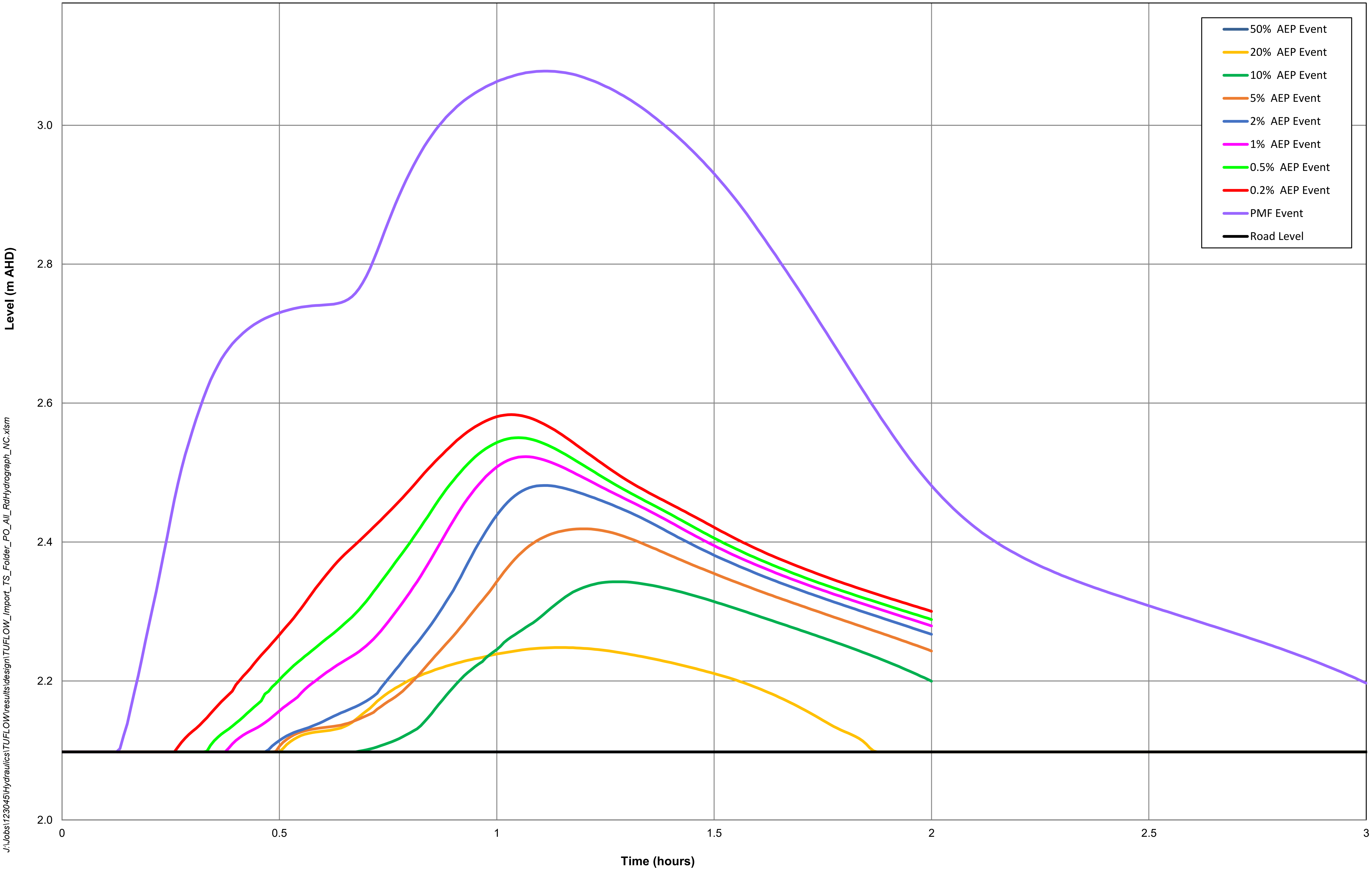
FIGURE D37

STAGE HYDROGRAPH
R17: SEAMAN AVENUE SOUTH
DESIGN STORMS



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APPENDIX E. SENSITIVITY FIGURES

Appendix E

J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E01_020Y_060m_4573_IMPACT_RAINFALL_LOSS_INCREASED_BY_20PC.mxd

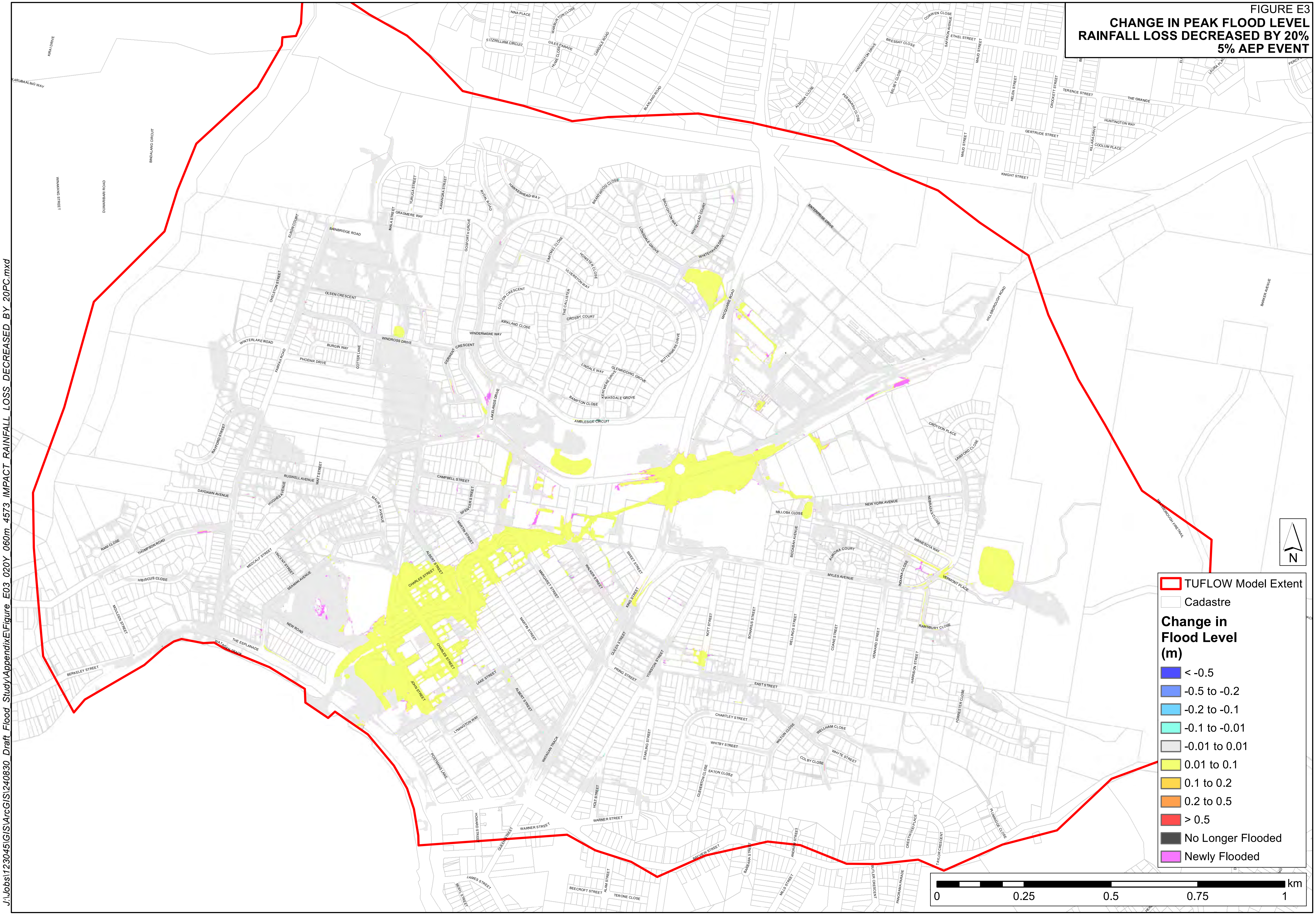
FIGURE E2

CHANGE IN PEAK FLOOD LEVEL
RAINFALL LOSS INCREASED BY 20%
1% AEP EVENT

J:\Jobs\123045\GIS\ArcGIS\240830 Draft Flood Study\AppendixE\Figure E02 100Y 060m 4463 IMPACT RAINFALL LOSS INCREASED BY 20PC.mxd

FIGURE E3

CHANGE IN PEAK FLOOD LEVEL
RAINFALL LOSS DECREASED BY 20%
5% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E03_020Y_060m_4573_IMPACT_RAINFALL_LOSS_DECREASED_BY_20PC.mxd

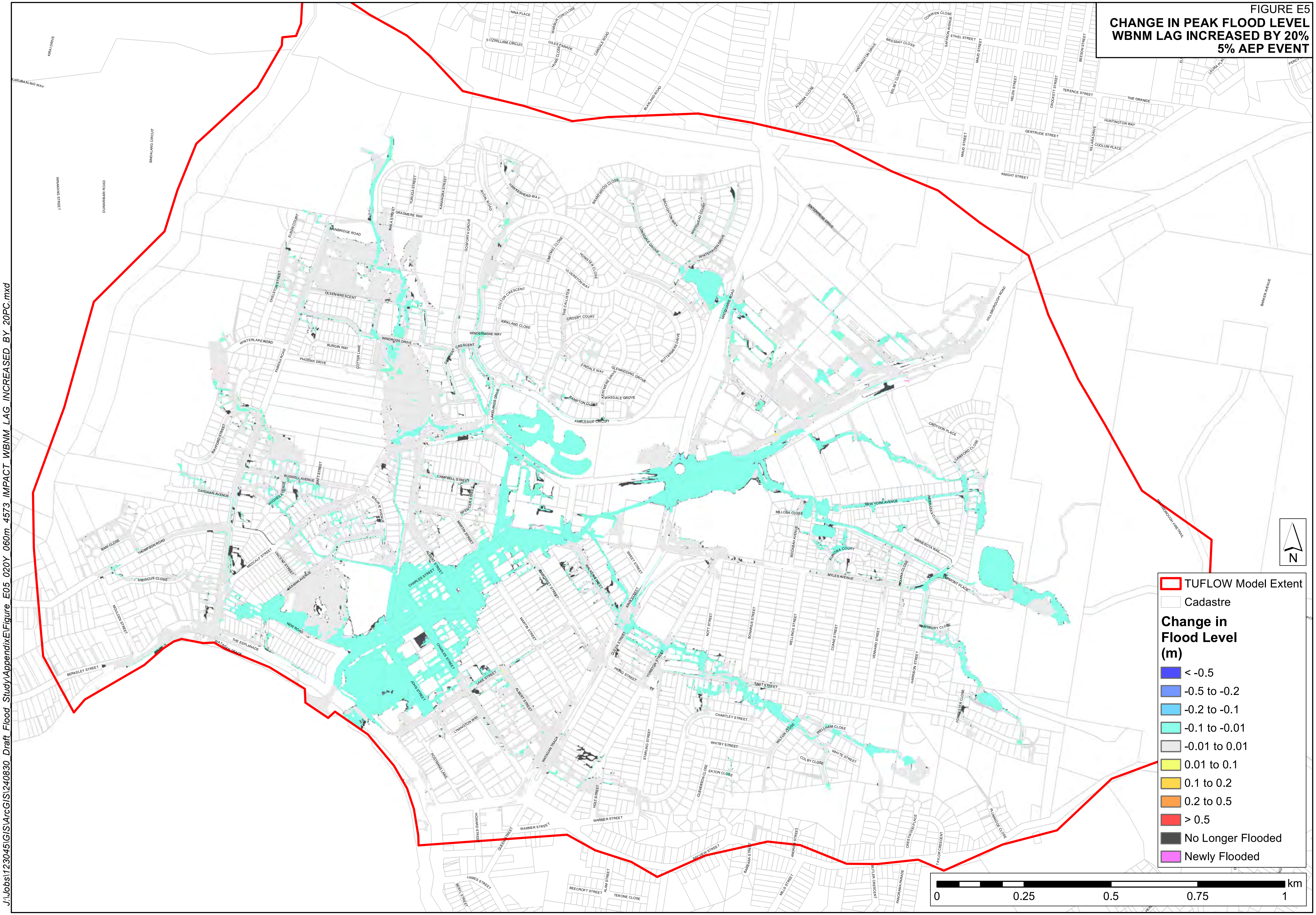
FIGURE E4

CHANGE IN PEAK FLOOD LEVEL
RAINFALL LOSS DECREASED BY 20%
1% AEP EVENT

J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E04_100Y_060m_4463_IMPACT_RAINFALL_LOSS_DECREASED_BY_20PC.mxd

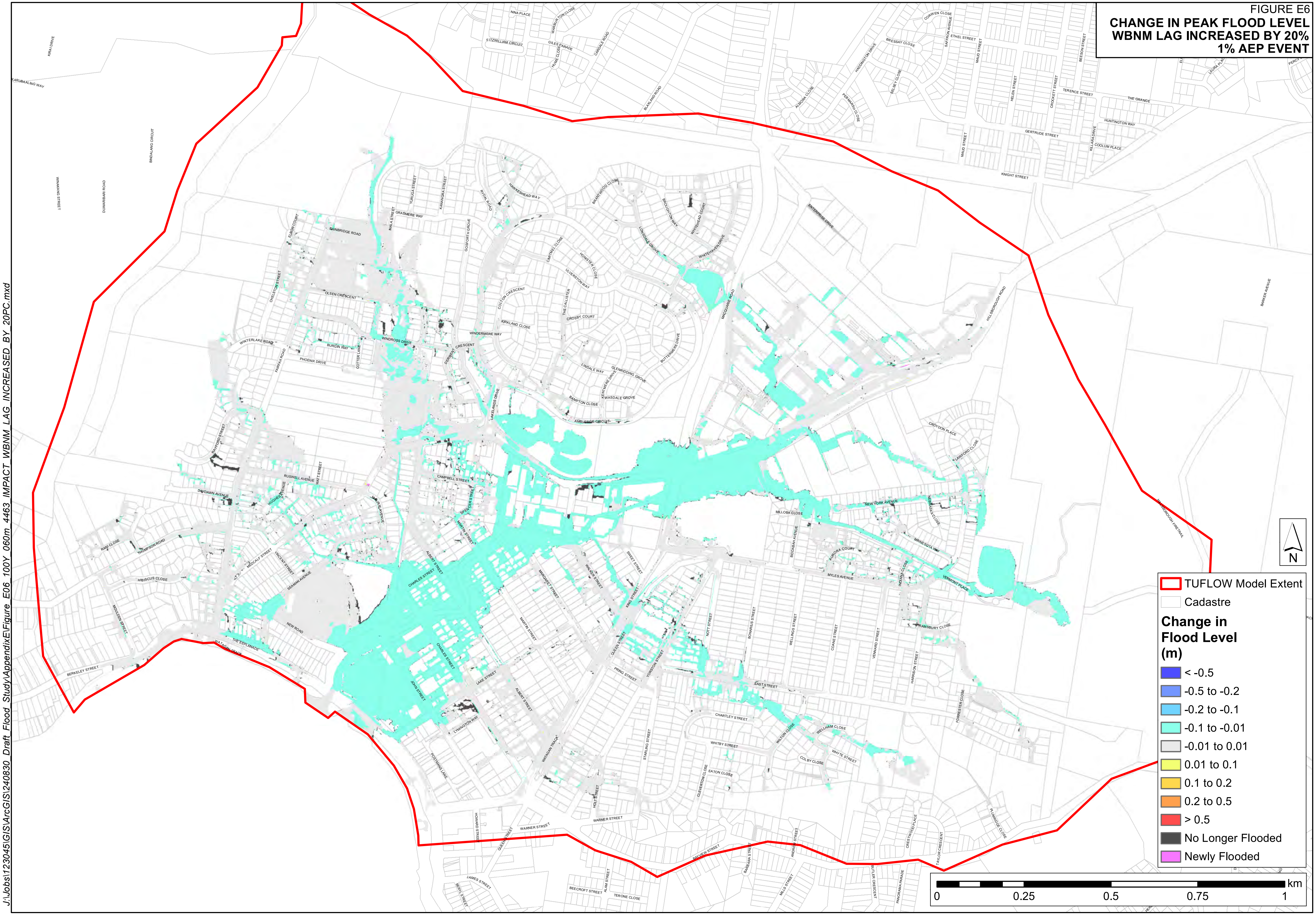
FIGURE E5

CHANGE IN PEAK FLOOD LEVEL
WBNM LAG INCREASED BY 20%
5% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E05_020Y_060m_4573_IMPACT_WBNM_LAG_INCREASED_BY_20PC.mxd

FIGURE E6
CHANGE IN PEAK FLOOD LEVEL
WBNM LAG INCREASED BY 20%
1% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E06_100Y_060m_4463_IMPACT_WBNM_LAG_INCREASED_BY_20PC.mxd

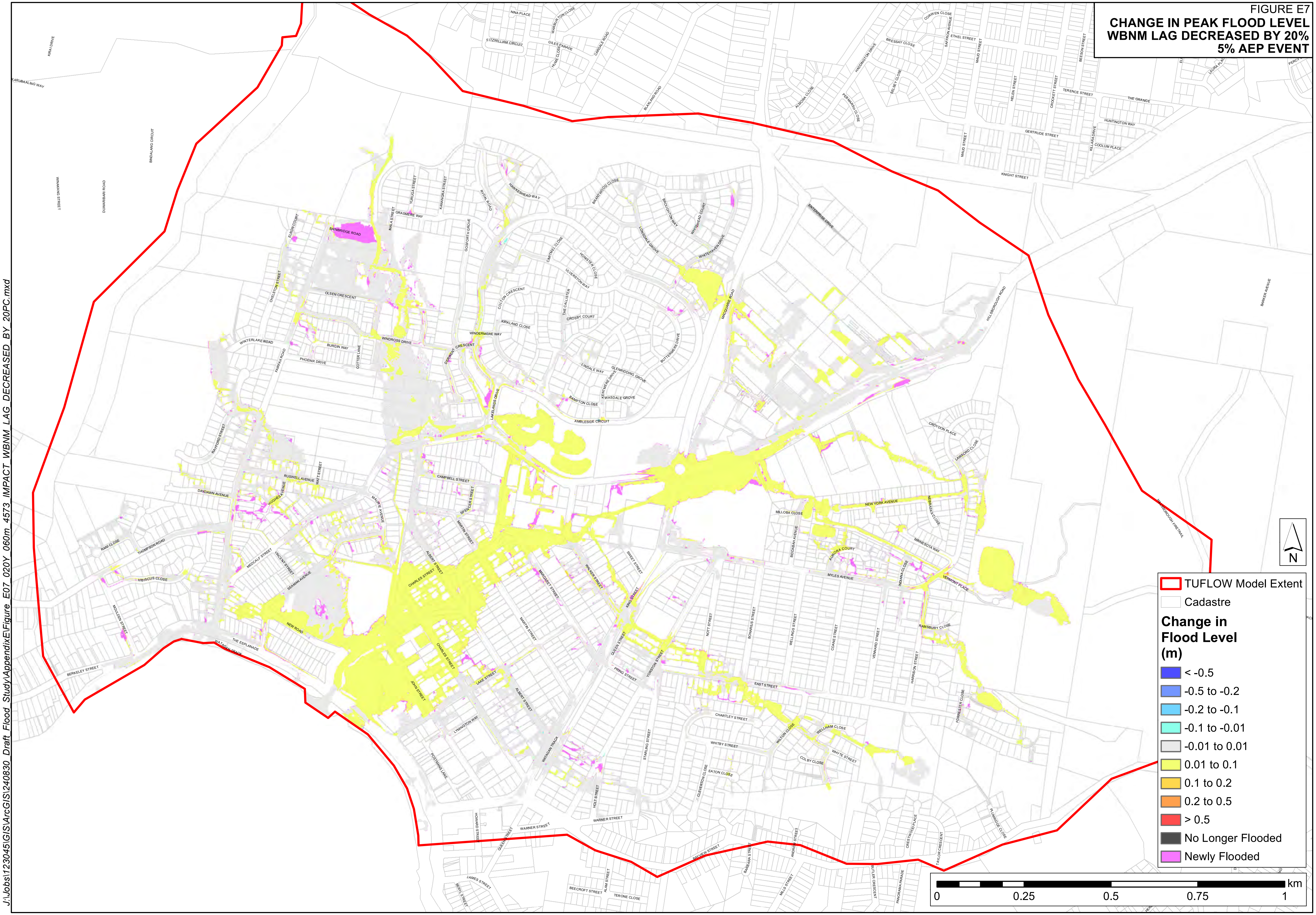
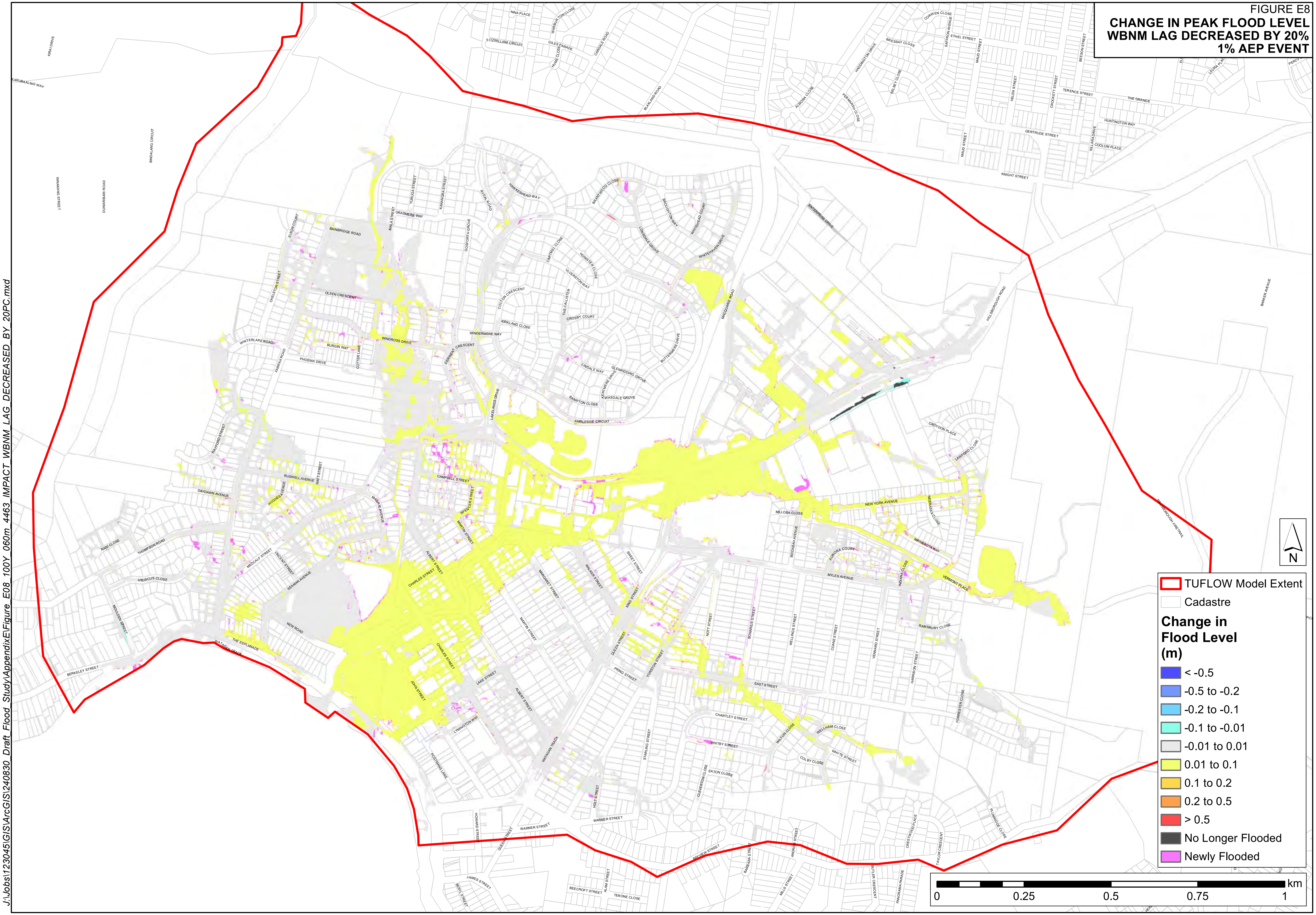


FIGURE E7
CHANGE IN PEAK FLOOD LEVEL
WBNM LAG DECREASED BY 20%
5% AEP EVENT

J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E07_020Y_060m_4573_IMPACT_WBNM_LAG_DECREASED_BY_20PC.mxd

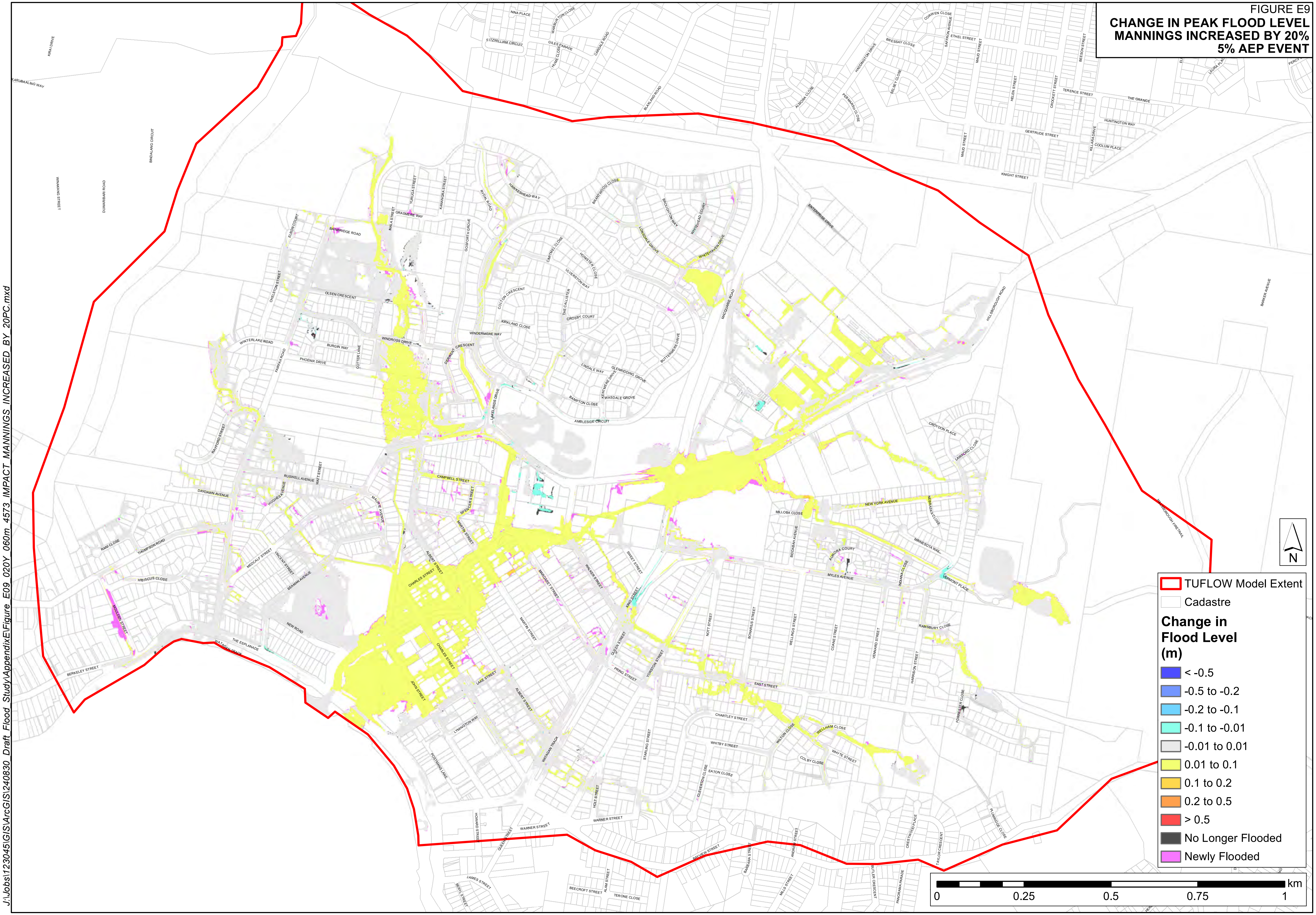
FIGURE E8
CHANGE IN PEAK FLOOD LEVEL
WBNM LAG DECREASED BY 20%
1% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830 Draft Flood Study\AppendixE\Figure E08 100Y 060m 4463 IMPACT WBNM LAG DECREASED BY 20PC.mxd

FIGURE E9

CHANGE IN PEAK FLOOD LEVEL
MANNINGS INCREASED BY 20%
5% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830 Draft Flood Study\AppendixE\Figure E09 020Y 060m 4573 IMPACT MANNINGS INCREASED BY 20PC.mxd

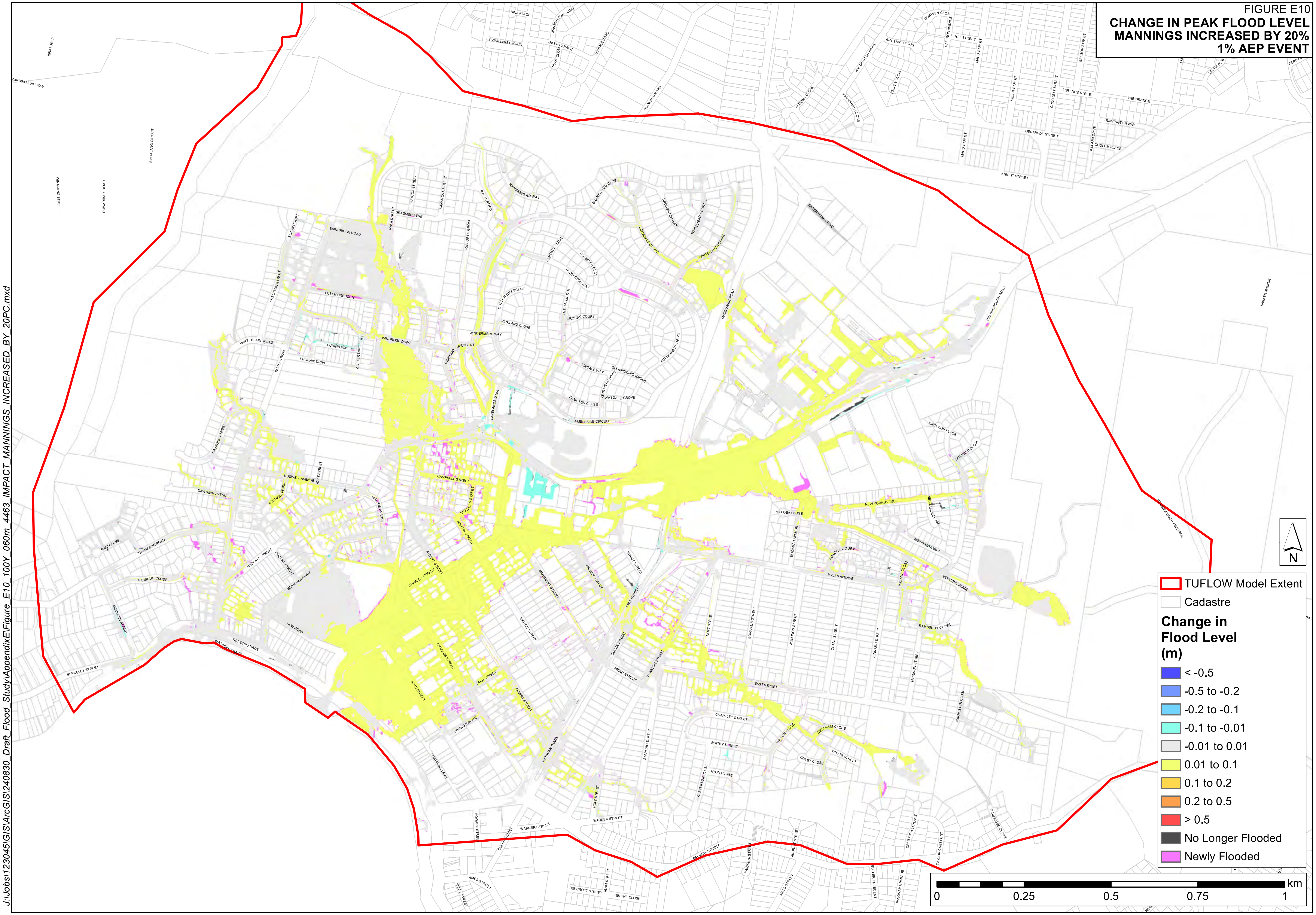
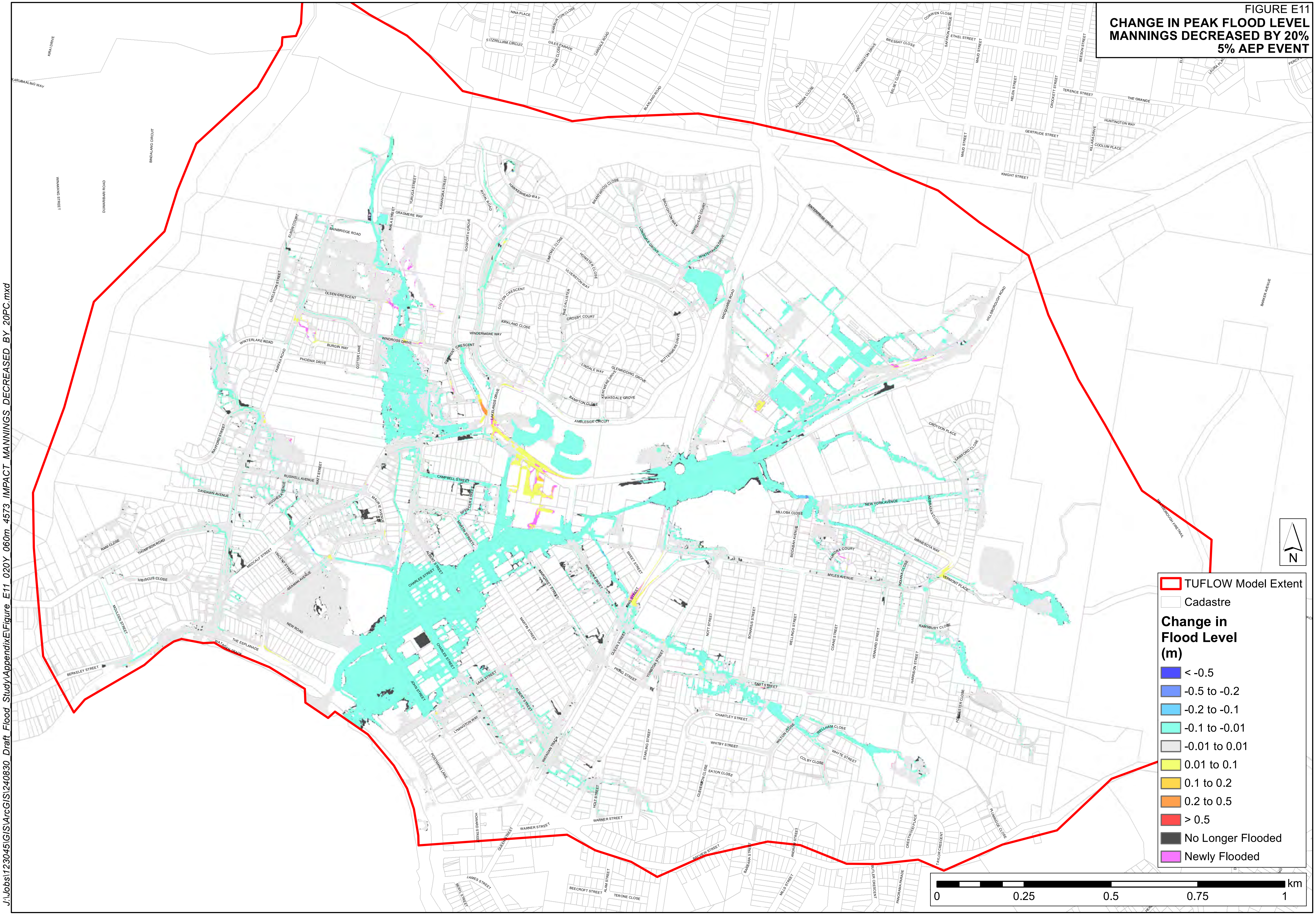


FIGURE E10
CHANGE IN PEAK FLOOD LEVEL
MANNINGS INCREASED BY 20%
1% AEP EVENT
TUFLOW Model Extent
Cadastre
Change in Flood Level (m)
< -0.5
-0.5 to -0.2
-0.2 to -0.1
-0.1 to -0.01
-0.01 to 0.01
0.01 to 0.1
0.1 to 0.2
0.2 to 0.5
> 0.5
No Longer Flooded
Newly Flooded
N
0
0.25
0.5
0.75
1
km
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E10_100Y_060m_4463_IMPACT_MANNINGS_INCREASED_BY_20PC.mxd



J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E11_020Y_060m_4573_IMPACT_MANNINGS_DECREASED_BY_20PC.mxd

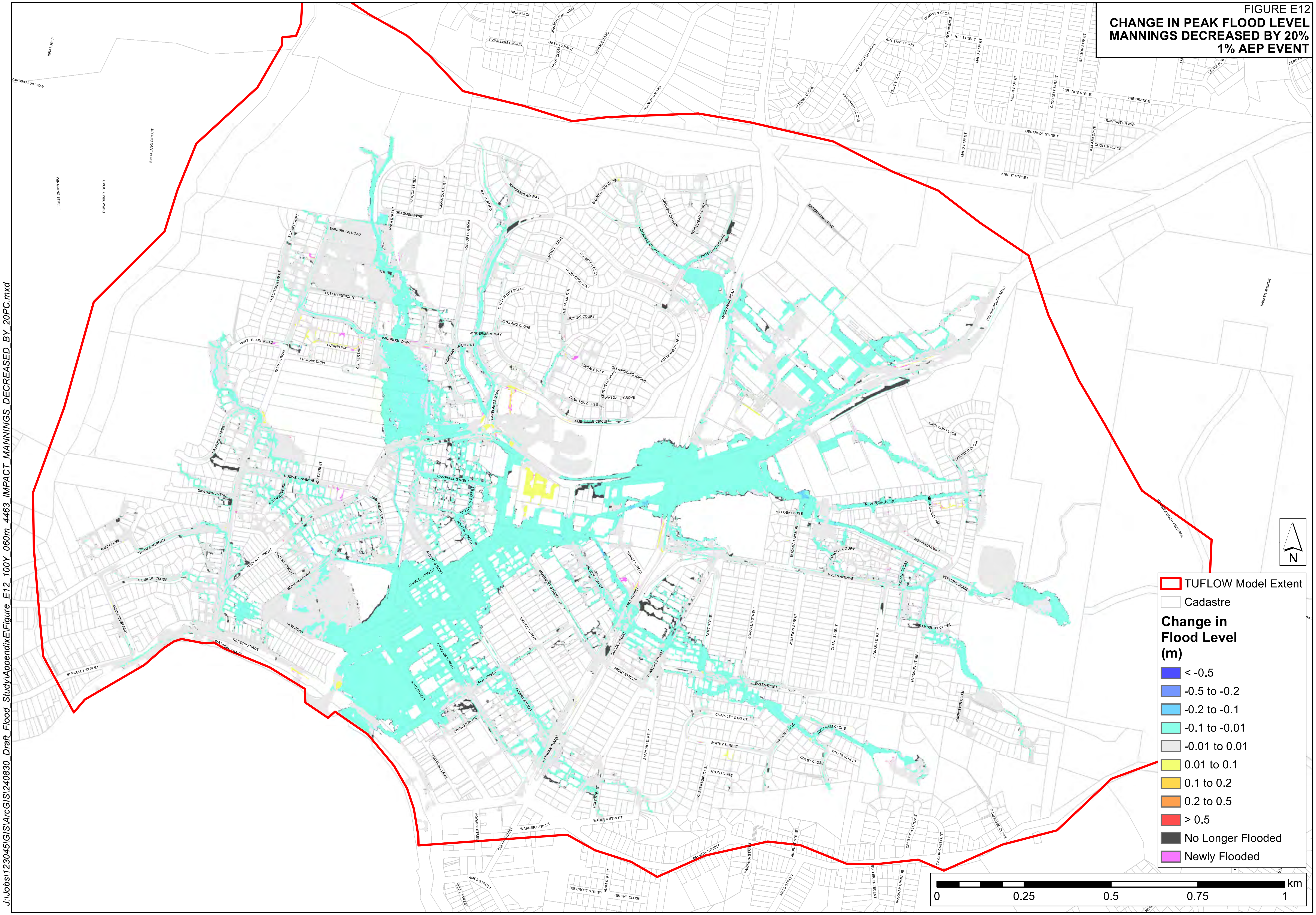
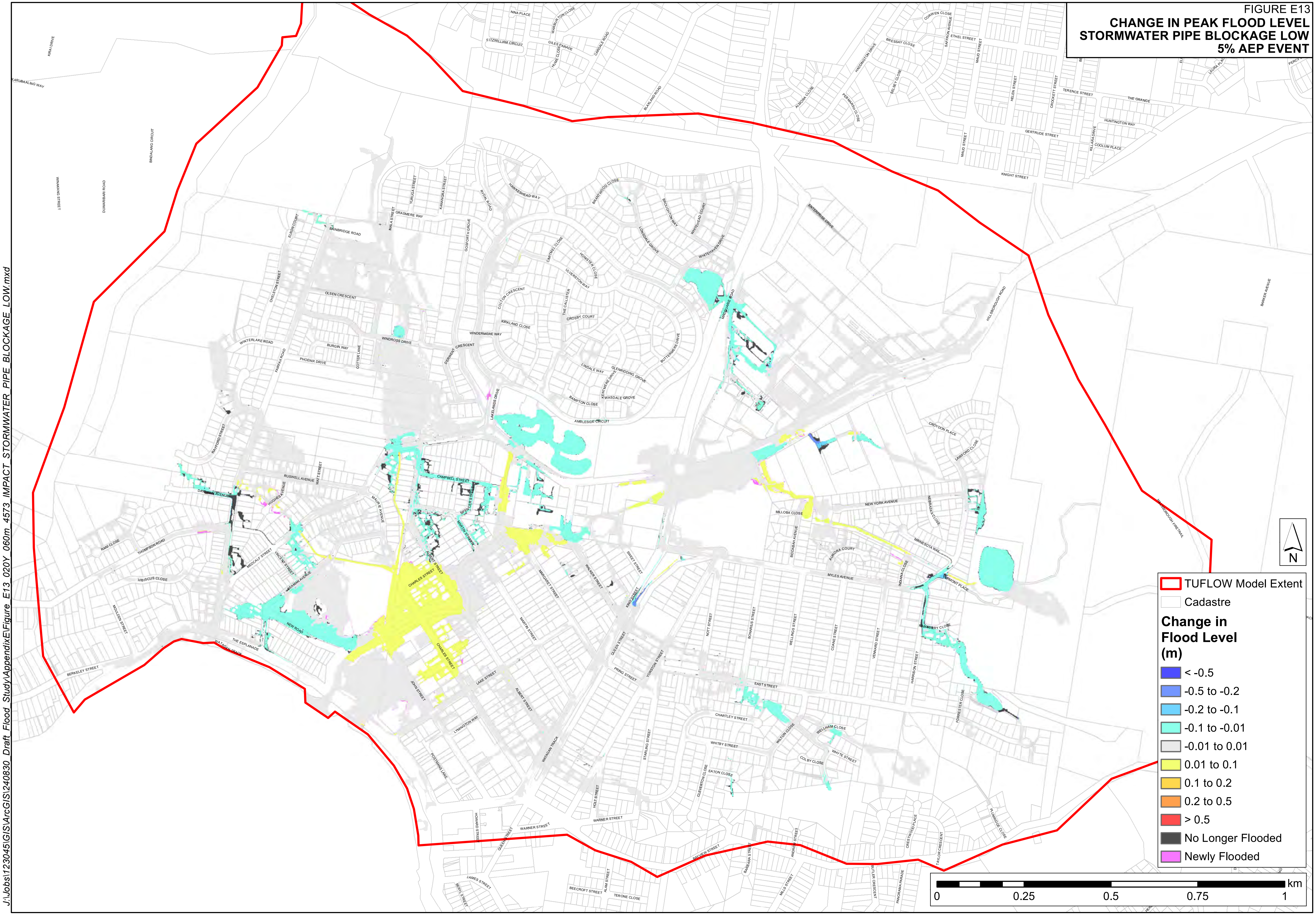


FIGURE E12
CHANGE IN PEAK FLOOD LEVEL MANNINGS DECREASED BY 20% 1% AEP EVENT

FIGURE E13
CHANGE IN PEAK FLOOD LEVEL STORMWATER PIPE BLOCKAGE LOW 5% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E13_020Y_060m_4573_IMPACT_STORMWATER_PIPE_BLOCKAGE_LOW.mxd

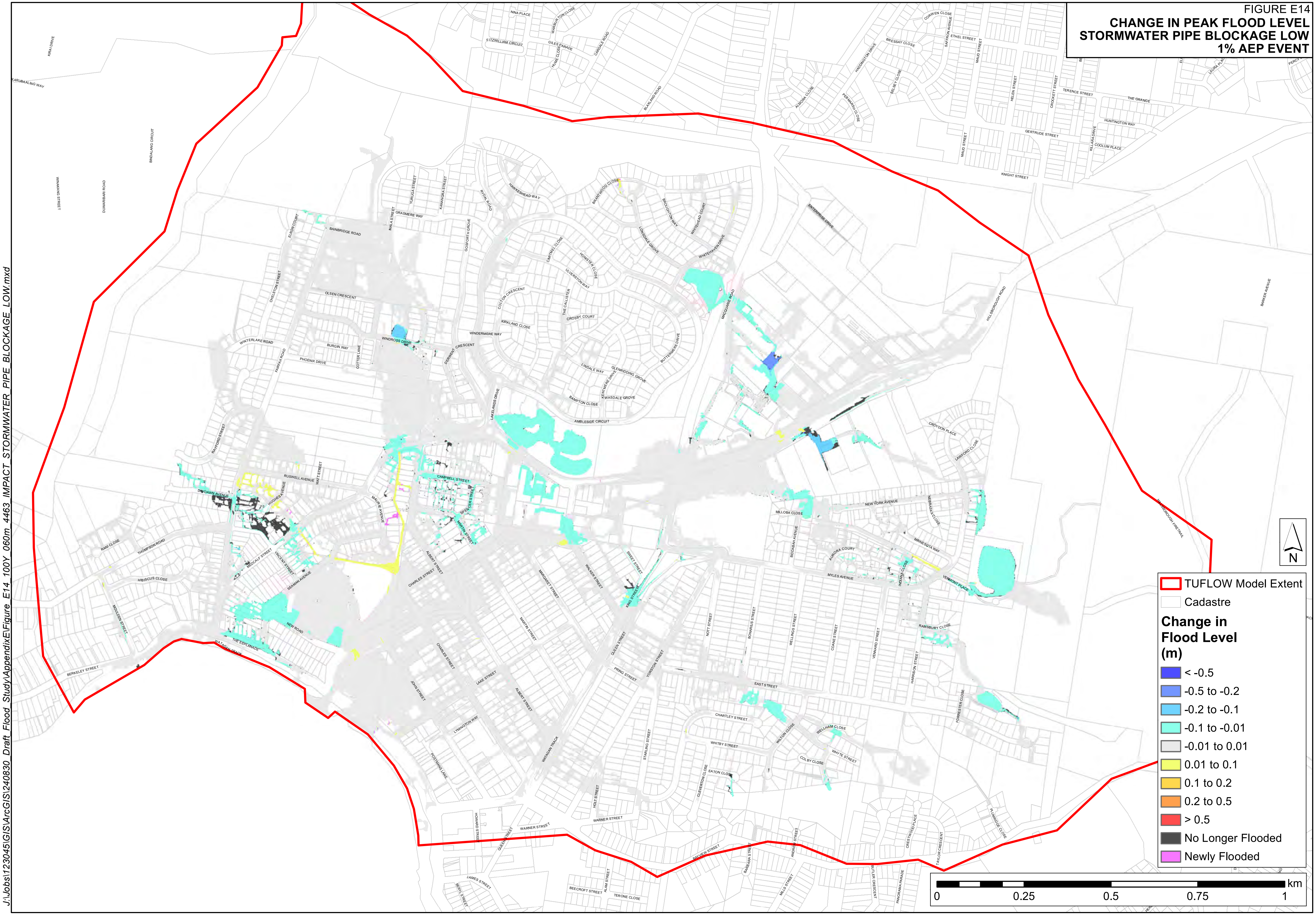
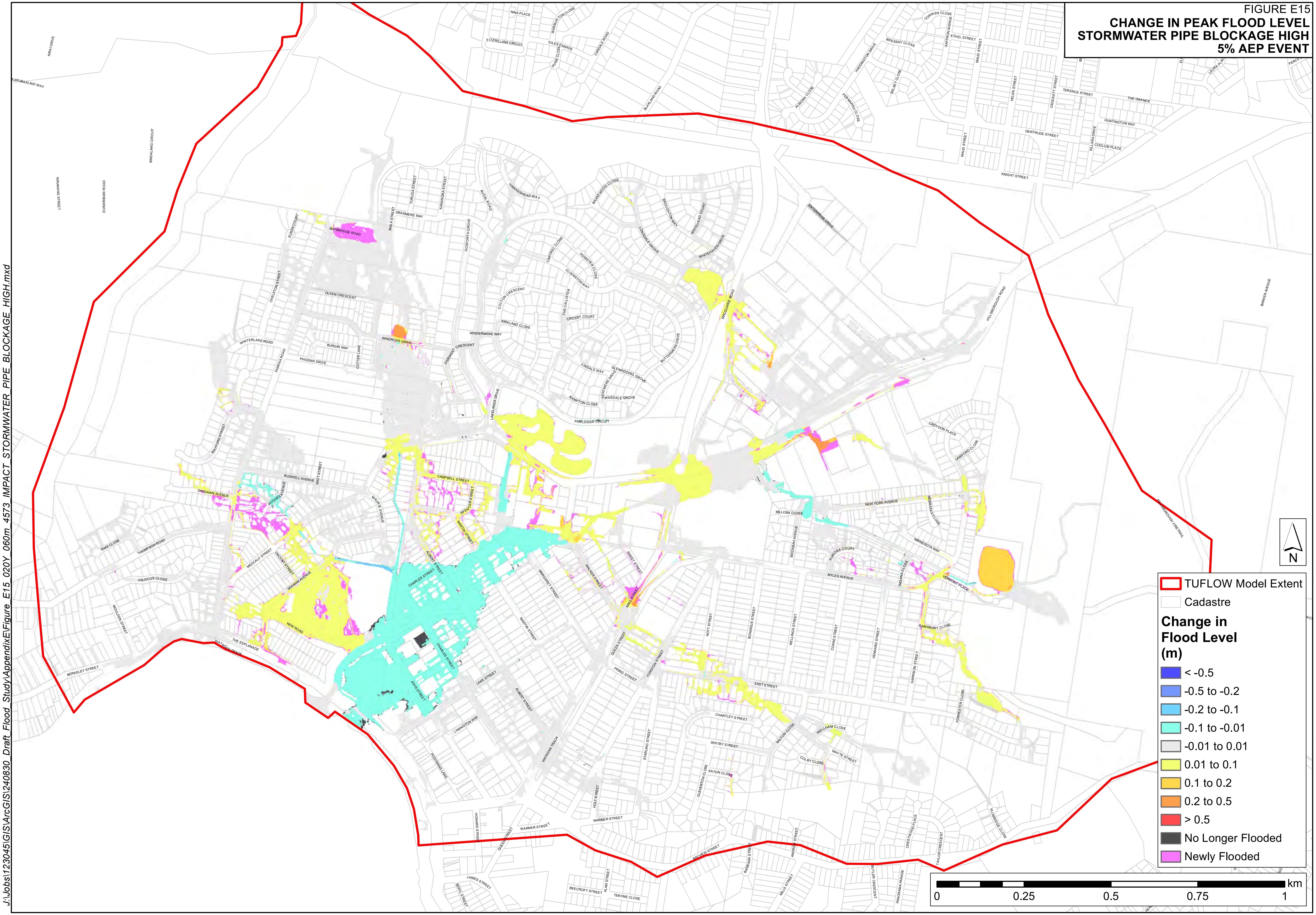


FIGURE E14
CHANGE IN PEAK FLOOD LEVEL
STORMWATER PIPE BLOCKAGE LOW
1% AEP EVENT
N
TUFLOW Model Extent
Cadastre
Change in Flood Level (m)
< -0.5
-0.5 to -0.2
-0.2 to -0.1
-0.1 to -0.01
-0.01 to 0.01
0.01 to 0.1
0.1 to 0.2
0.2 to 0.5
> 0.5
No Longer Flooded
Newly Flooded
0
0.25
0.5
0.75
1
km
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E14_100Y_060m_4463_IMPACT_STORMWATER_PIPE_BLOCKAGE_LOW.mxd

FIGURE E15
CHANGE IN PEAK FLOOD LEVEL STORMWATER PIPE BLOCKAGE HIGH 5% AEP EVENT



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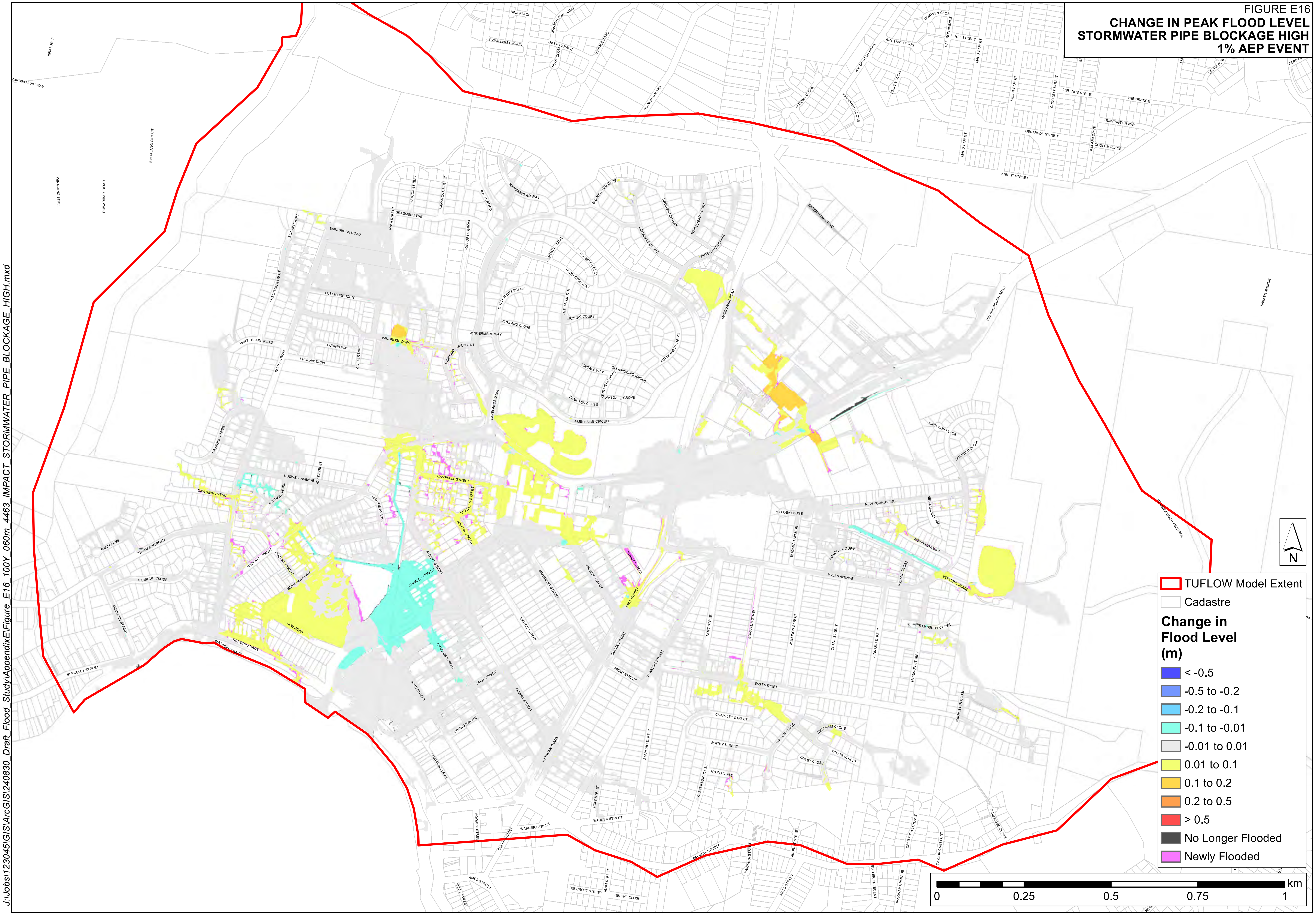

FIGURE E16
CHANGE IN PEAK FLOOD LEVEL STORMWATER PIPE BLOCKAGE HIGH 1% AEP EVENT

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FIGURE E17
**CHANGE IN PEAK FLOOD LEVEL
STORMWATER PIT BLOCKAGE LOW
5% AEP EVENT**

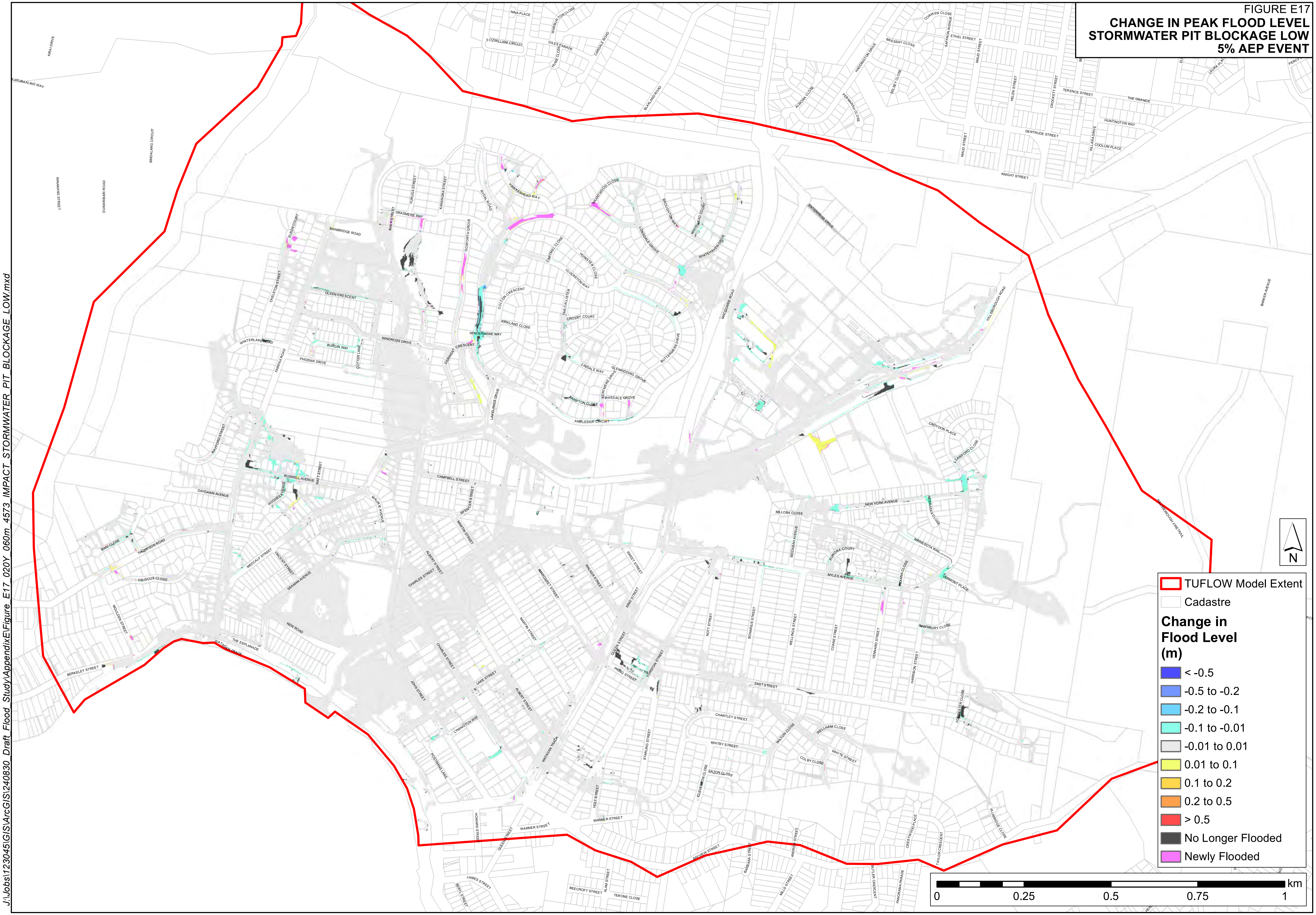

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FIGURE E18

CHANGE IN PEAK FLOOD LEVEL
STORMWATER PIT BLOCKAGE LOW
1% AEP EVENT

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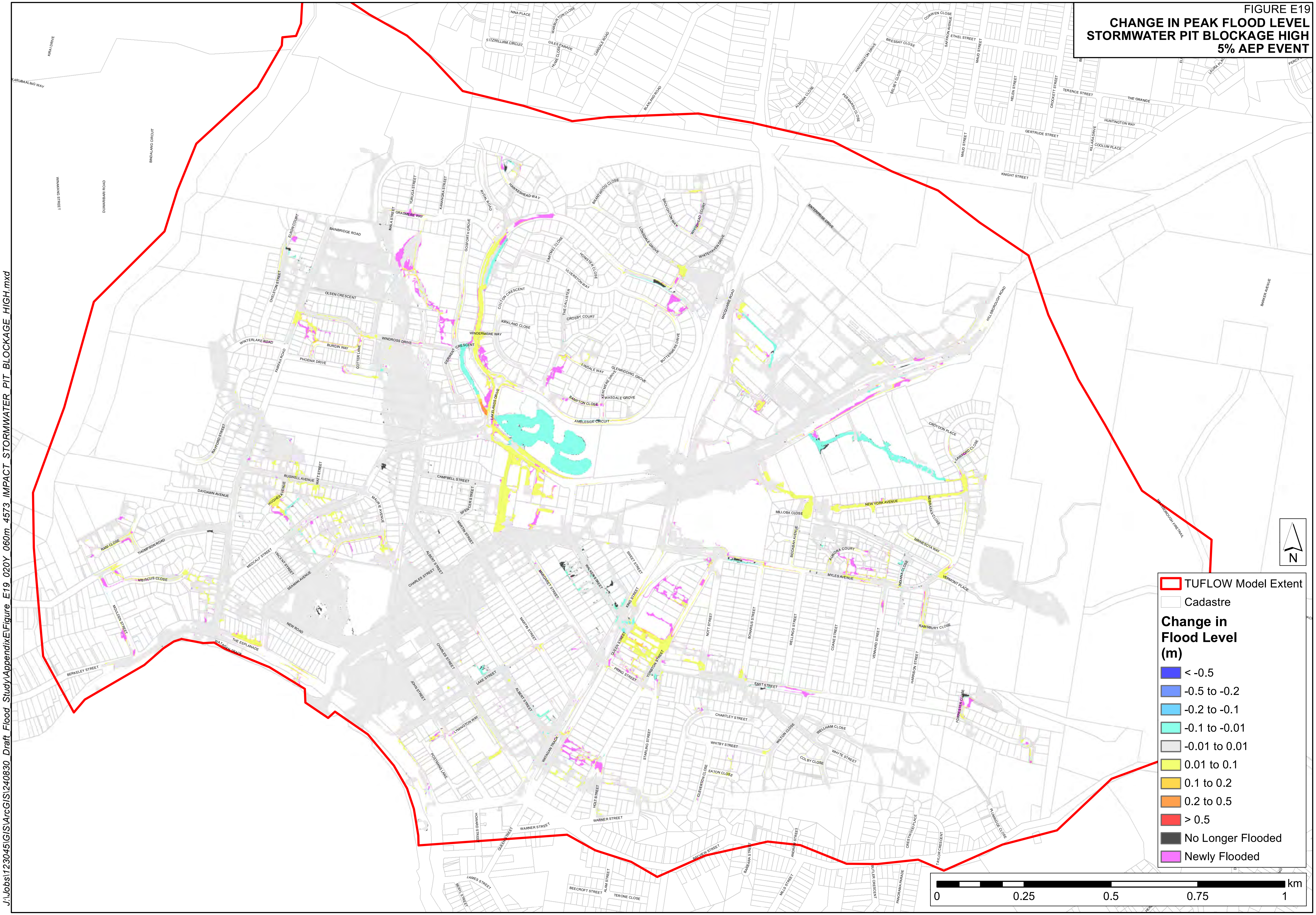

FIGURE E19
CHANGE IN PEAK FLOOD LEVEL
STORMWATER PIT BLOCKAGE HIGH
5% AEP EVENT

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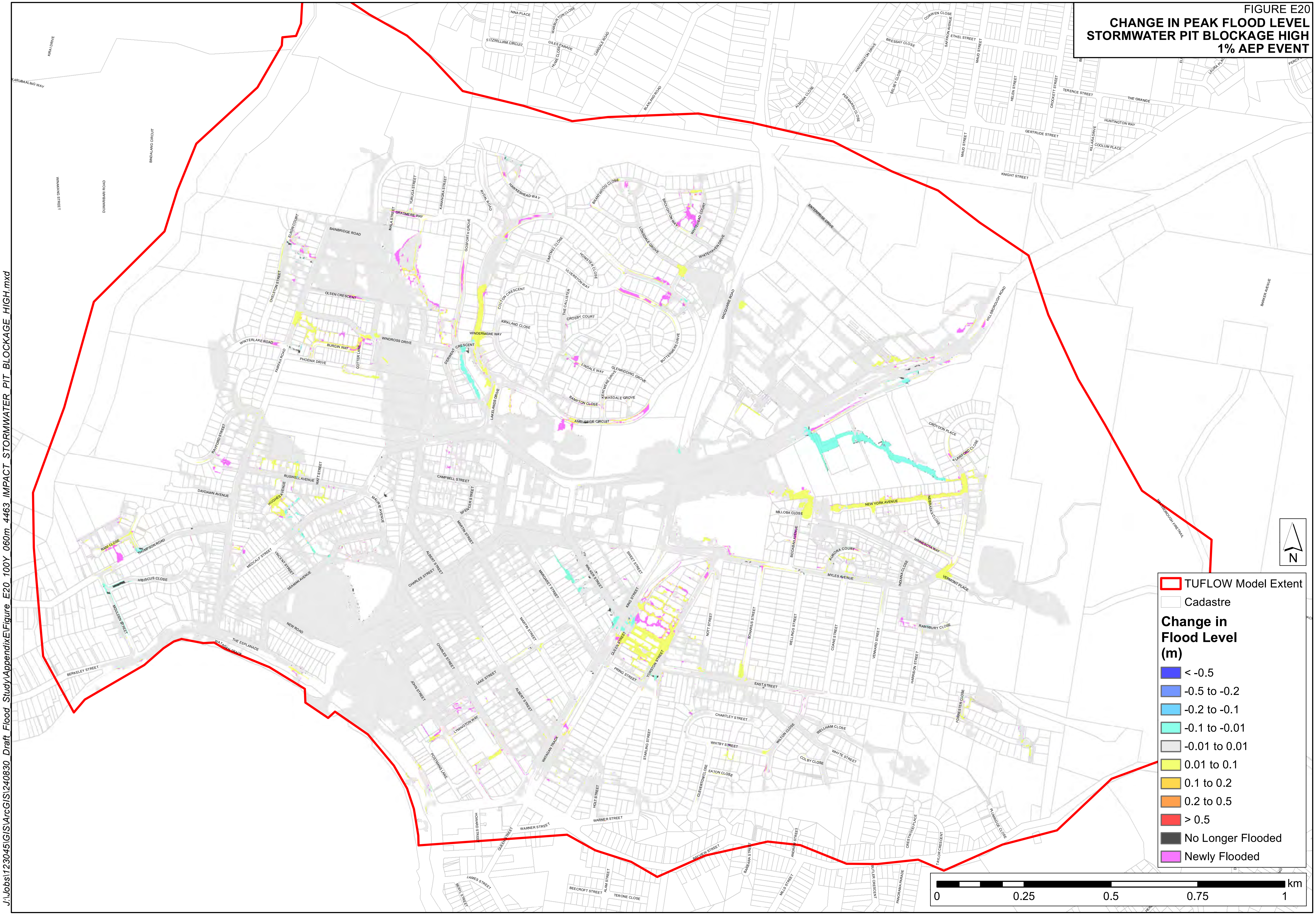

FIGURE E20
CHANGE IN PEAK FLOOD LEVEL
STORMWATER PIT BLOCKAGE HIGH
1% AEP EVENT
N
TUFLOW Model Extent
Cadastre
Change in Flood Level (m)
< -0.5
-0.5 to -0.2
-0.2 to -0.1
-0.1 to -0.01
-0.01 to 0.01
0.01 to 0.1
0.1 to 0.2
0.2 to 0.5
> 0.5
No Longer Flooded
Newly Flooded
km
0
0.25
0.5
0.75
1
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E20_100Y_060m_4463_IMPACT_STORMWATER_PIT_BLOCKAGE_HIGH.mxd

FIGURE E21
CHANGE IN PEAK FLOOD LEVEL
LAKE LEVEL REDUCED 250MM
5% AEP EVENT
TUFLOW Model Extent
Cadastre
Change in Flood Level (m)
< -0.5
-0.5 to -0.2
-0.2 to -0.1
-0.1 to -0.01
-0.01 to 0.01
0.01 to 0.1
0.1 to 0.2
0.2 to 0.5
> 0.5
No Longer Flooded
Newly Flooded
N
0
0.25
0.5
0.75
1
km
J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E21_020Y_060m_4573_IMPACT_LAKE_LEVEL_REDUCED_250MM.mxd

FIGURE E22

CHANGE IN PEAK FLOOD LEVEL
LAKE LEVEL REDUCED 250MM
1% AEP EVENT

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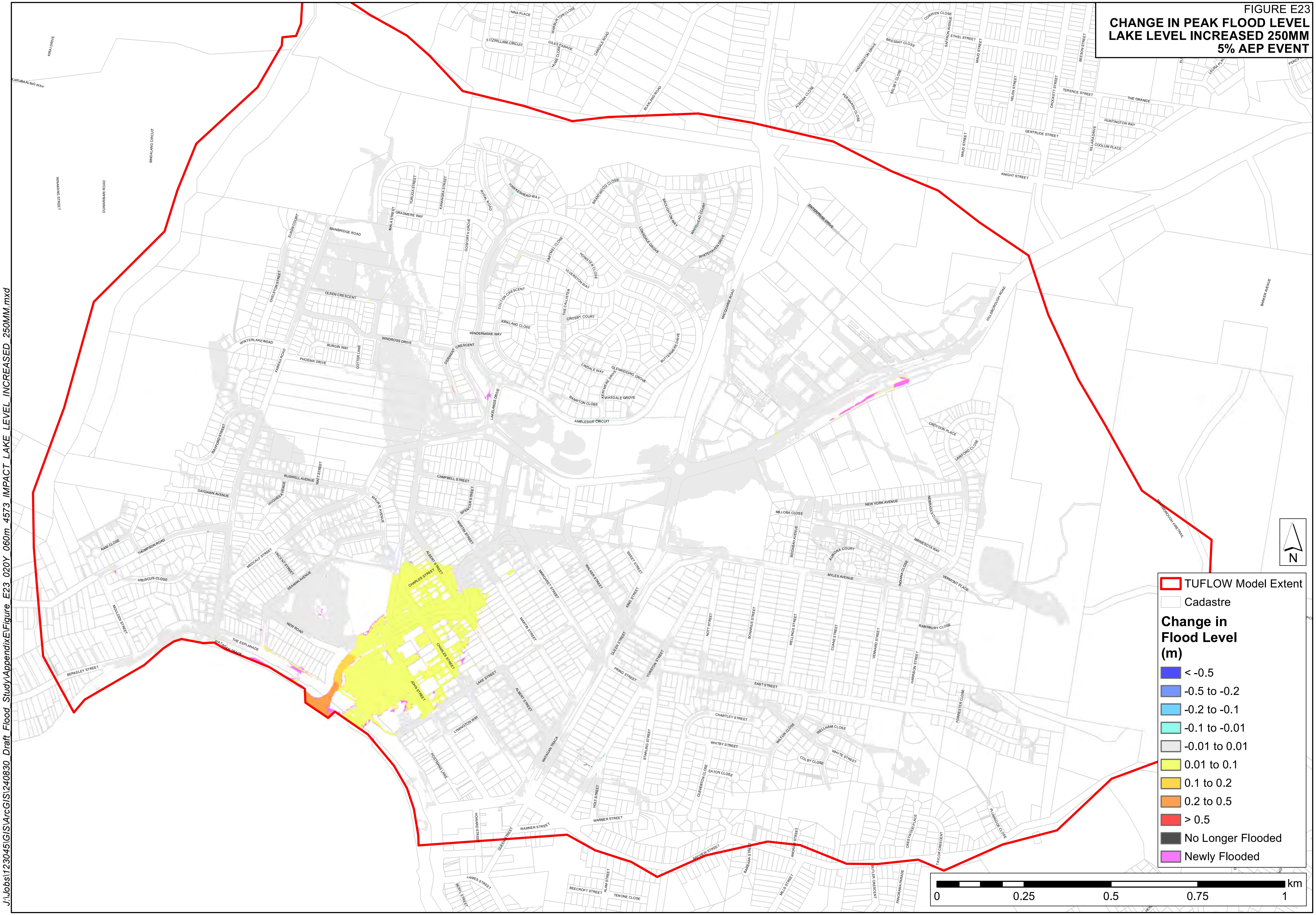
FIGURE E23

CHANGE IN PEAK FLOOD LEVEL
LAKE LEVEL INCREASED 250MM
5% AEP EVENT



TUFLOW Model Extent
Cadastre

Change in Flood Level (m)

- < -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to -0.01
- -0.01 to 0.01
- 0.01 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- > 0.5
- No Longer Flooded
- Newly Flooded

0 0.25 0.5 0.75 1 km

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FIGURE E24

CHANGE IN PEAK FLOOD LEVEL
LAKE LEVEL INCREASED 250MM
1% AEP EVENT

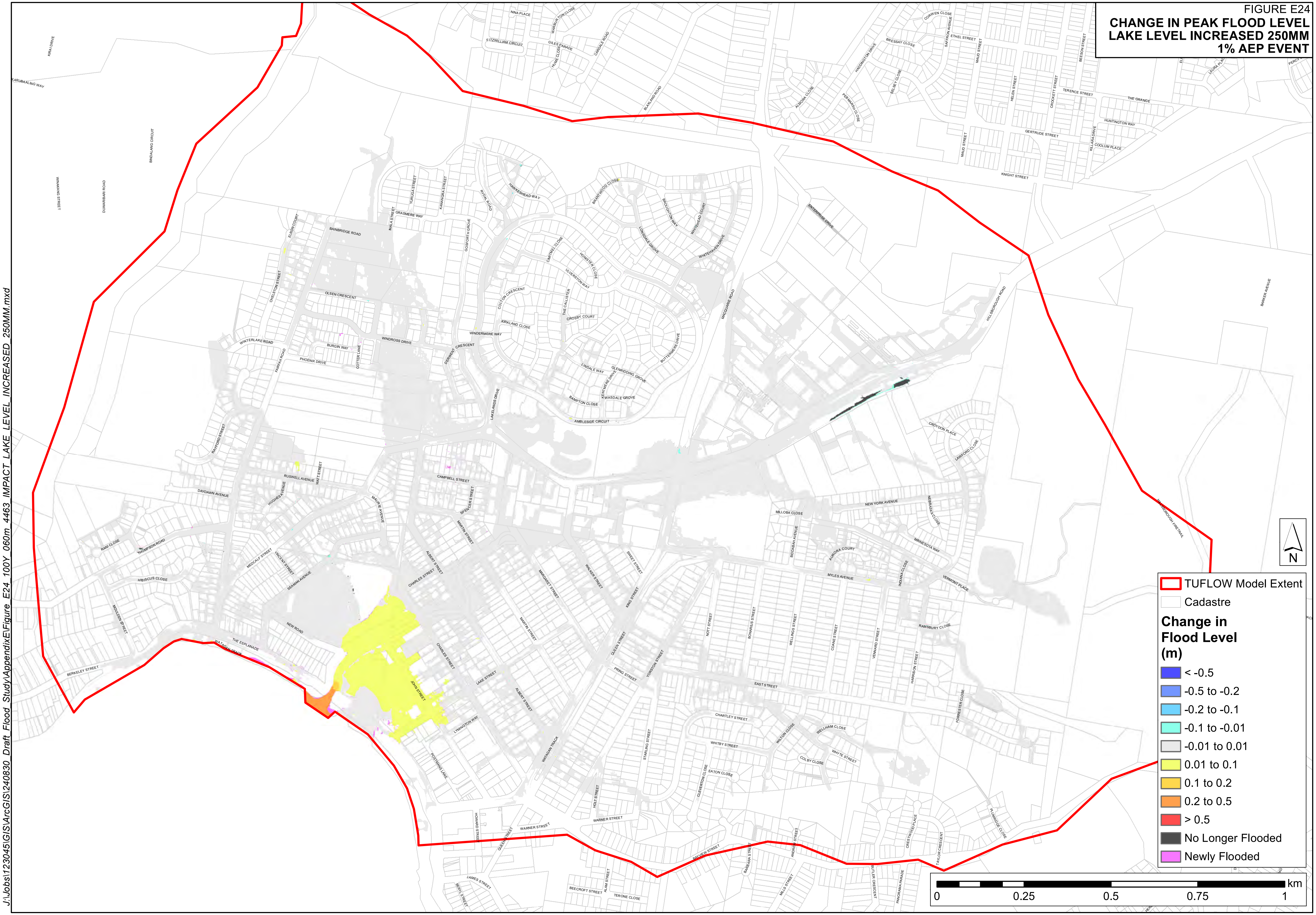

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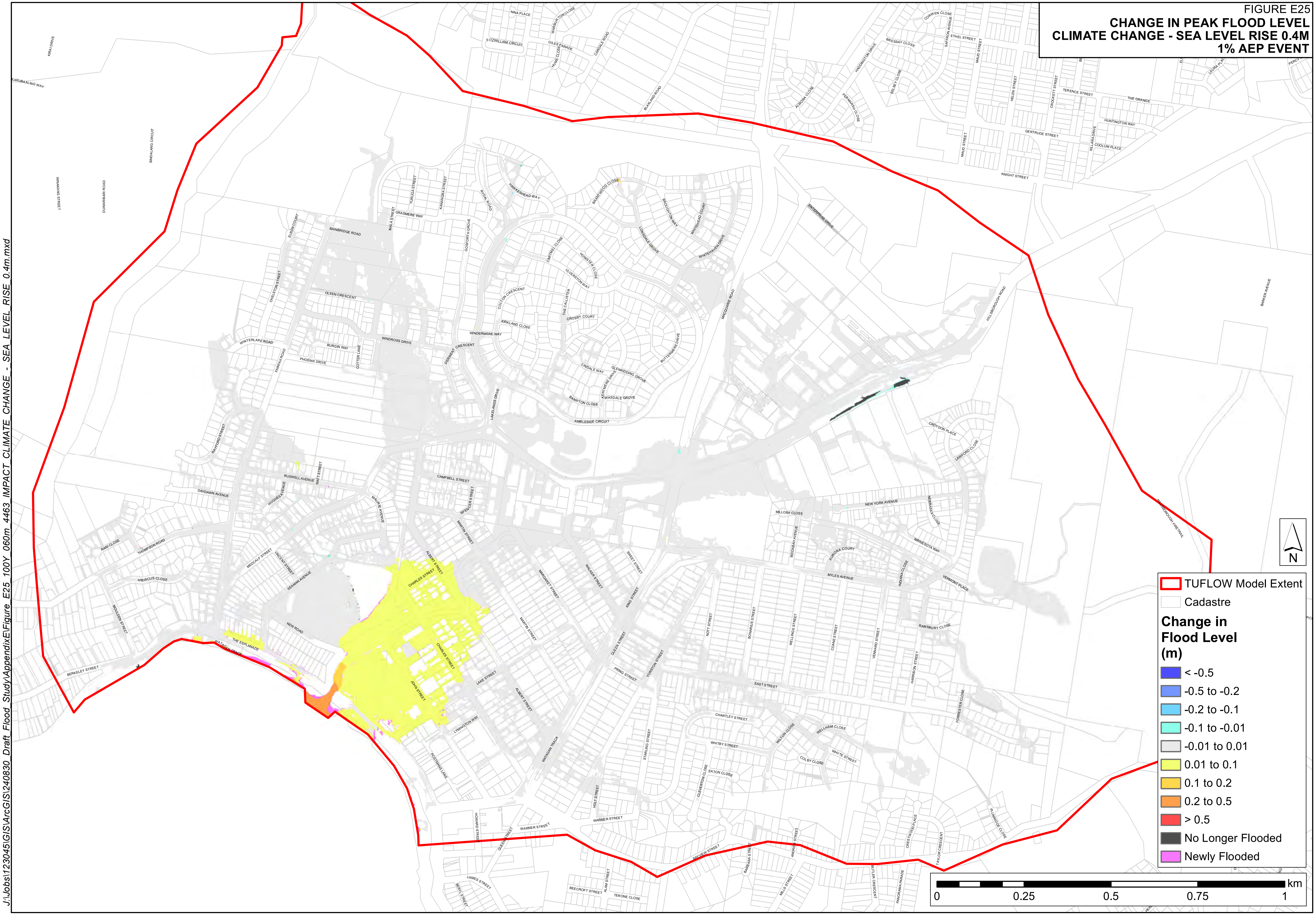
FIGURE E25

CHANGE IN PEAK FLOOD LEVEL
CLIMATE CHANGE - SEA LEVEL RISE 0.4M
1% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E25_100Y_060m_4463_IMPACT_CLIMATE_CHANGE_-_SEA_LEVEL_RISE_0.4m.mxd

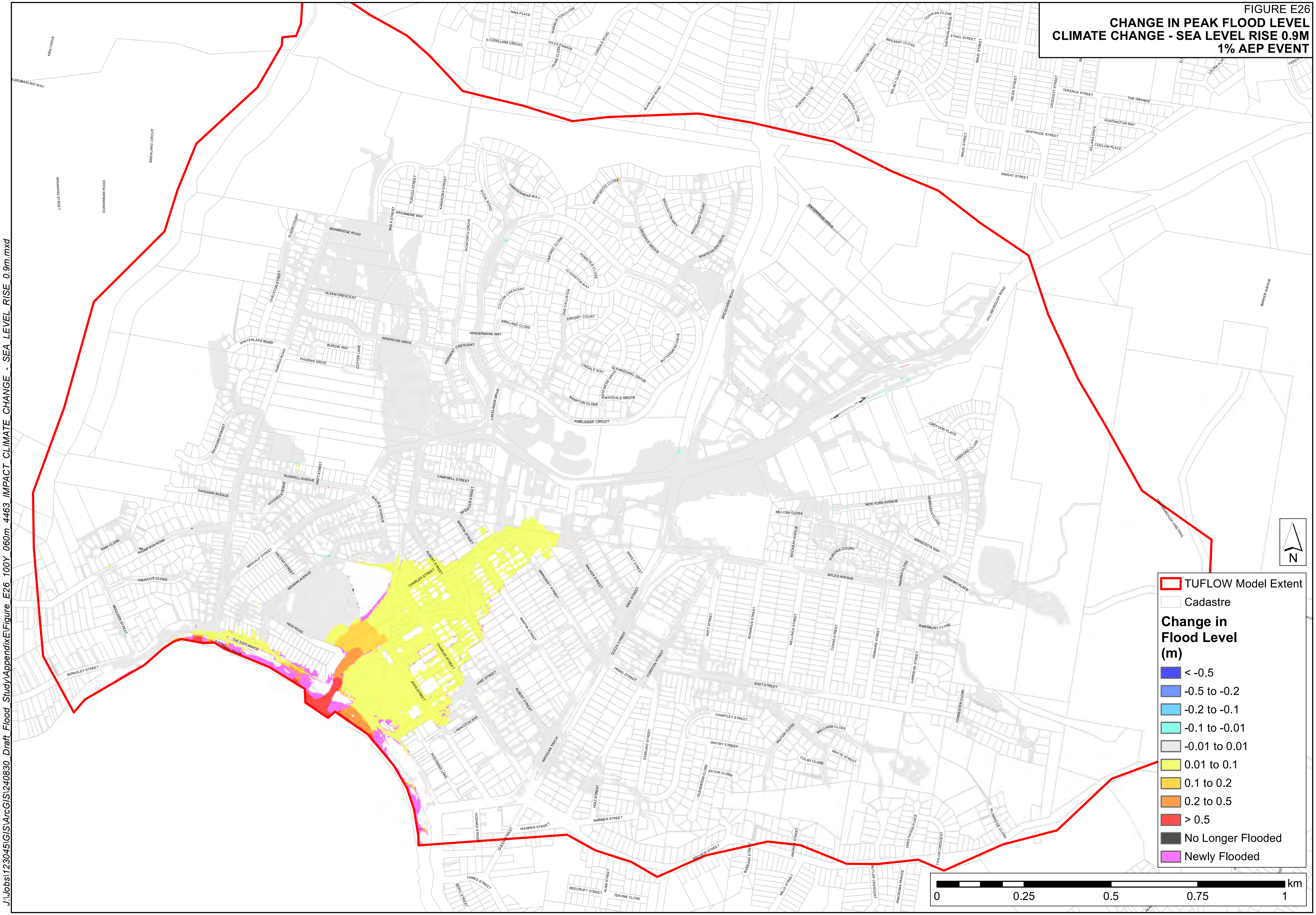


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FIGURE E27

CHANGE IN PEAK FLOOD LEVEL
CLIMATE CHANGE - RAINFALL INTENSITY
0.5% AEP VERSUS 1% AEP EVENT

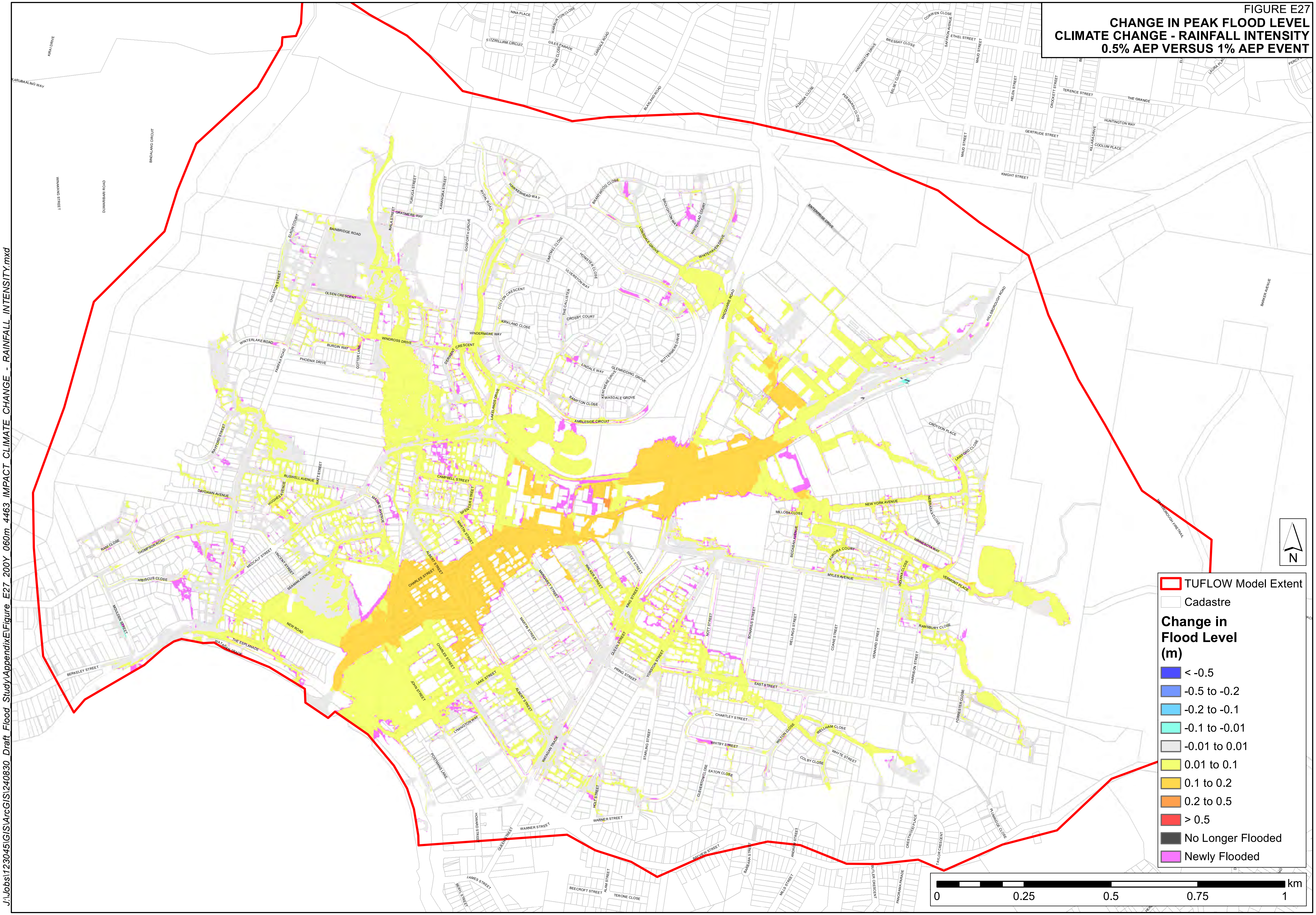

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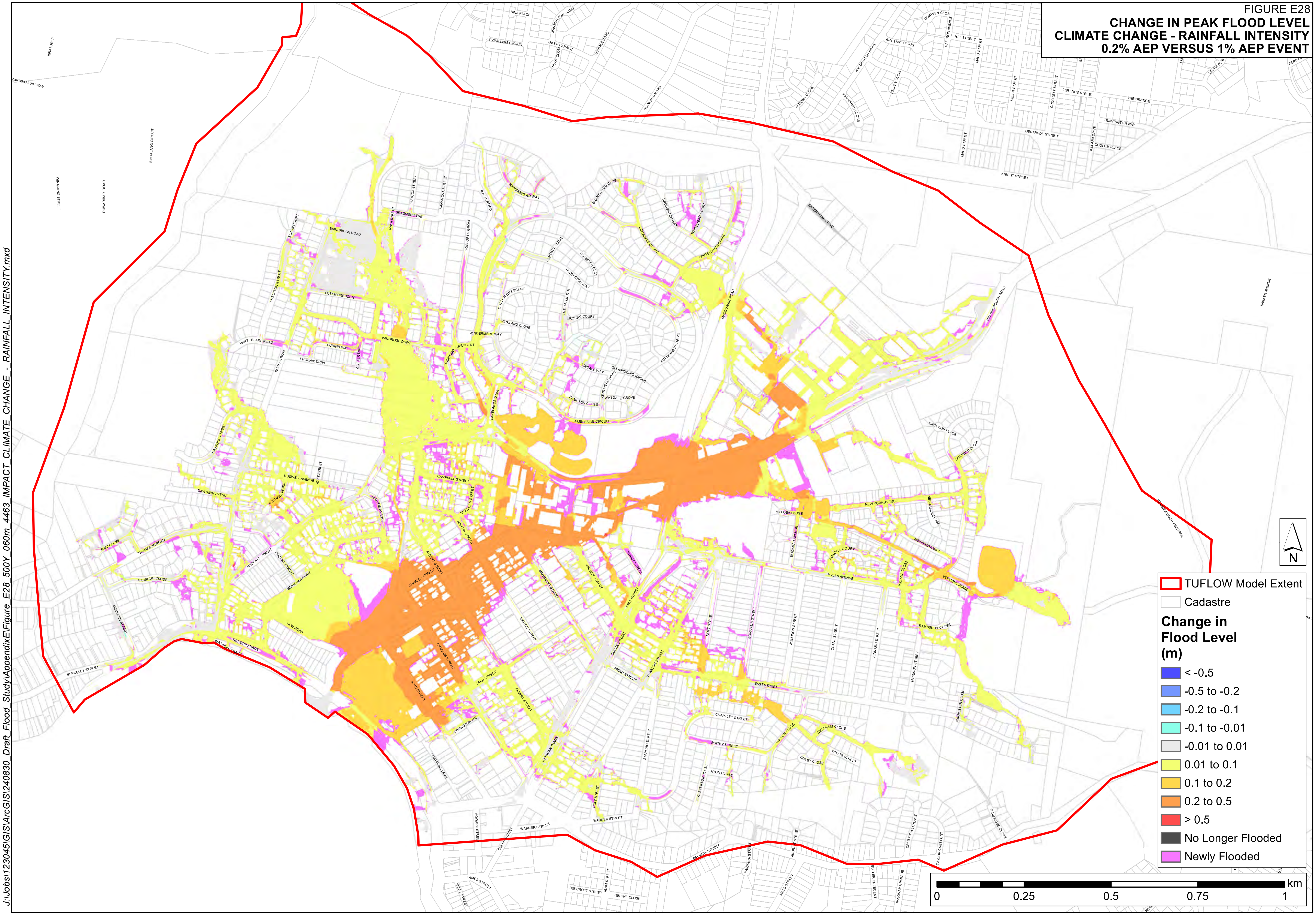
FIGURE E28

CHANGE IN PEAK FLOOD LEVEL
CLIMATE CHANGE - RAINFALL INTENSITY
0.2% AEP VERSUS 1% AEP EVENT



J:\Jobs\123045\GIS\ArcGIS\240830_Draft_Flood_Study\AppendixE\Figure_E28_500Y_060m_4463_IMPACT_CLIMATE_CHANGE_-_RAINFALL_INTENSITY.mxd