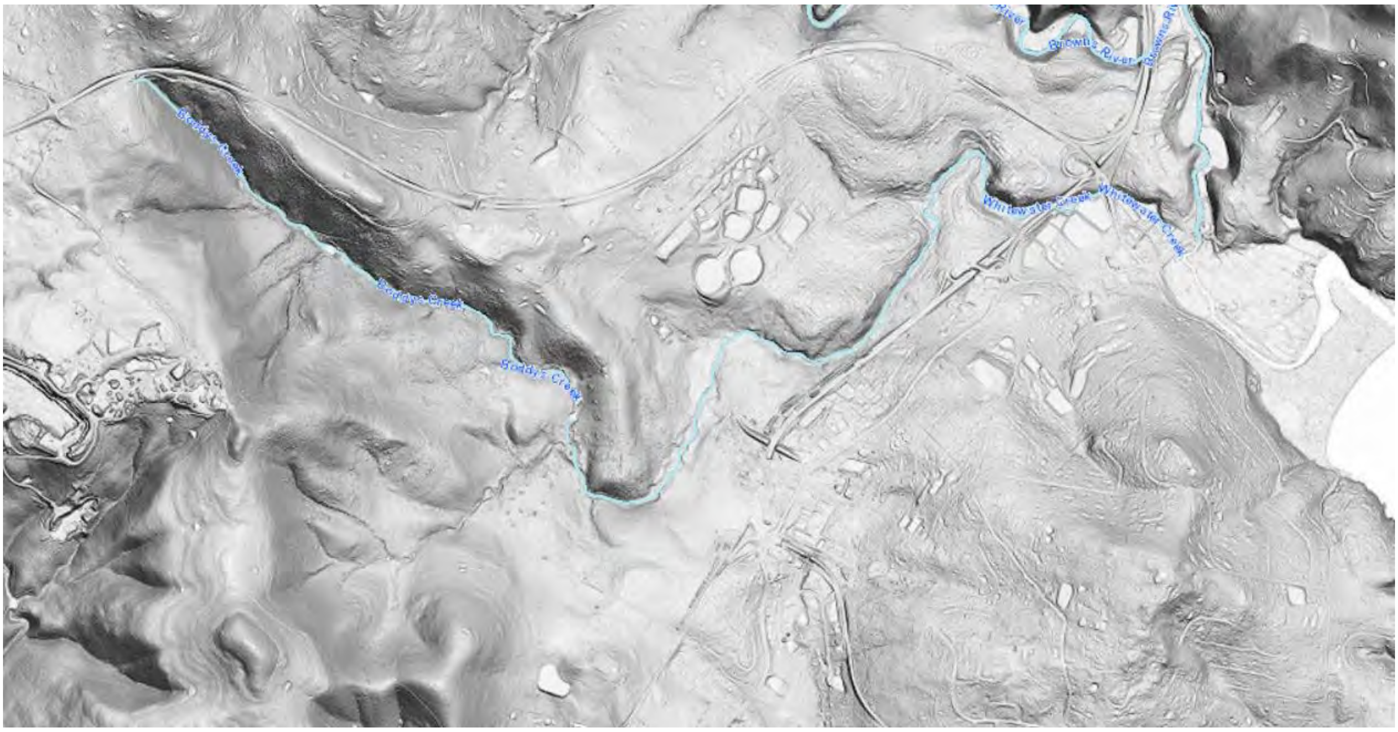
KINGBOROUGH COUNCIL

WHITEWATER CREEK FLOOD STUDY

FINAL REPORT



APRIL 2020

101 West Fyans Street
Newtown, VIC, 3220

Tel: (02) 9299 2855
Fax: (02) 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

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Project WHITEWATER CREEK FLOOD STUDY	**Project Number** **119087**
Client KINGBOROUGH COUNCIL	**Client's Representative** Alan H Walker & Alexander Aronsson
Project Manager Mark Colegate	

Revision History

Revision	Description	Distribution	Authors	Reviewed by	Date
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1	Final Report, Issued to Council	Alan Walker, Kingborough Council	Yuan Li, Mark Colegate	Fiona Ling, Mark Colegate	APR 20
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LIST OF ACRONYMS

ADR	Australian Disaster Resilience
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ARR	Australian Rainfall and Runoff
BOM	Bureau of Meteorology
DRM	Direct Rainfall Method
DEM	Digital Elevation Model
GIS	Geographic Information System
GPS	Global Positioning System
IFD	Intensity, Frequency and Duration (Rainfall)
IL/CL	Initial Loss / Continues Loss
LIST	Land Information System Tasmania
m AHD	meters above Australian Height Datum
PMF	Probable Maximum Flood
SRMT	Shuttle Radar Mission Topography
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software (hydraulic model)
WBNM	Watershed Bounded Network Model (hydrologic model)

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 2016 recommends the use of Annual Exceedance Probability (AEP). Annual Exceedance Probability (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 Average Recurrence Interval where there is no seasonality, or an event that is likely to occur twice in one year.

The Probable Maximum Flood 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.

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	

EXECUTIVE SUMMARY

WMAwater was engaged by Kingborough Council to develop a flood study for Whitewater Creek near the township of Kingston in Tasmania. The study comprises the development of hydrologic and hydraulic models to define design flood behaviour and establish the flood risk for the 5% AEP, 1% AEP and 0.5% AEP design storms and 2050 and 2100 Climate Change scenarios in the Whitewater Creek catchment.

The primary aim of the study is to provide Council with flood intelligence that can be used in the preparation of planning controls for development applications, developing flood plain management strategies and understanding the flood risks associated with a range of event probabilities.

The study uses hydrologic and hydraulic modelling techniques in order to define flood behaviour in the study area. The modelling programs used in the study are:

- WBNM (Hydrologic) for the upper rural area – the model converts rainfall to runoff and the flow hydrographs are input into the TUFLOW model.
- TUFLOW (Hydraulic) for the lower urbanised area – The 1D/2D hydraulic model was established to assess the complex overland flow regimes of the urban catchments to analyse flooding behaviour in the study area.

The models were calibrated to the May 2018 flood event, using scaled pluviograph records and registered flood marks. The models were used to assess design flood behaviour for a range of events including the 5% AEP, 1% AEP and 0.5% AEP events. Comprehensive mapping of the design flood information across the catchment was undertaken, including mapping of peak flood extents, levels, velocities and depths, as well as hydraulic hazards and categories.

Sensitivity analysis was conducted in terms of climate change (2050 and 2100 scenarios), surface roughness, and drainage and key structure blockage. The model results are sensitive to increased rainfall intensity (climate change), and generally are not sensitive to surface roughness and blockage variations other than for the blockage at the culvert under Huon Highway.

The modelling results and mapping, including the existing and climate change conditions, provide flood behaviour and intelligence information for Council to adopt for planning purpose.

Preliminary investigation of non-structural and structural mitigation options was conducted through concept design and modelling assessment. Non-structural measures, e.g., waterway maintenance, promoting flood awareness education, development of emergency management plan, and implementation of flood overlays to inform planning, are recommended due to their relatively low cost, high feasibility, and reasonable effectiveness. Construction of flood protection levees along hotspots along Whitewater Creek is recommended as structural options due to the sound benefit over reasonable cost.

1. INTRODUCTION

Kingborough Council (Council) commissioned WMAwater to develop a flood study for Whitewater Creek near the township of Kingston in Tasmania.

The study comprises the development of computational hydrologic and hydraulic models to define design flood behaviour and establish the flood risk for the 5%, 1% and 0.5% Annual Exceedance Probability (AEP) design storms and, the 2050 and 2100 Climate Change scenarios in the Whitewater Creek catchment.

Study outputs will provide Council with flood intelligence that can be used in the preparation of planning controls for development applications, developing flood plain management strategies and understanding the flood risks associated with a range of event probabilities.

Study outputs include maps of flood extent, depth, velocity and hazard for flood events with a range of Annual Exceedance Probabilities (AEPs), identification and assessment of flood mitigation options, reports detailing the study methods, investigations and conclusions, hydrologic and hydraulic models, and digital datasets.

The specific tasks undertaken for the study were as follows:

- the collection and collation of existing information relevant to the study which includes the data already held by Council as well as other information, such as development, topographic, GIS and rainfall data
- the preparation of hydrologic and hydraulic models capable of defining the flood behaviour for the study area for a wide range of design flood probabilities
- undertaking sensitivity analysis
- the interpretation and presentation of model results to describe and categorise flood behaviour and hazard for a range of design storm events for the existing catchment conditions
- analysis of hotspots
- identification and consideration of flood mitigation opportunities
- investigating and ultimately determining the flood risk in the Whitewater Creek.

A discussion of terminology and a glossary of other flood-related terms is provided in APPENDIX A.

2. BACKGROUND

2.1. Study Area

The Whitewater Creek catchment is situated at the southeast coastal area of Tasmania. It covers approximately 11.7 km^2, including its downstream tributary, i.e., Kingston Rivulet sub-catchment, as shown in Figure 1.

Whitewater Creek is a tributary to Browns River and flows from west to east through the western part of Kingston. Elevations range from 2 m AHD to 307 m AHD (mapping of the topography from LiDAR aerial survey is shown in Figure 2).

The upper and lower parts of the catchment are distinct in nature. The upper catchment is rural, covered by forests, pastures, and rural resources; whilst the lower catchment is urbanised with a mix of residential, commercial, and industrial land uses. The upper catchment is relatively steep, with grades ranging from approximately 6% to 20%; while the lower catchment is moderately steep to relatively flat, with grades of approximately 2% to 8% in the urbanised areas common. Drainage elements in the catchment include natural creek channels (including Whitewater Creek and tributaries), kerbs and gutters, and pits and pipes (mainly in lower urbanised catchment).

2.2. Historical Flooding

Flood events in the Kingston area are listed in the Kingston Beach Flood Study (Kingborough Council, 2015, Reference 1). Since that flood study was completed, a significant flood event occurred on 10th – 11th May 2018. Very high rainfalls occurred in the south of Tasmania on the evening of the 10th May 2018. The heavy rainfalls were due to a highly active line of thunderstorms that trained over Hobart and surrounding areas over a number of hours, with each thunderstorm following a similar path as it moved in from the east. Much of the rainfall fell within approximately 6 hours, leading to flash flooding in many streams, including Whitewater Creek. During this event, Kingston (Greenhill Drive) daily rainfall site recorded the highest rainfall in the 18 years of record. The 24-hour rainfall is estimated to be approximately a 0.5% AEP event. Shorter duration rainfalls are likely to be rarer. The pluviograph site at Hobart recorded a 2-hour rainfall that was approximately a 1 in 2000 AEP event. The 24-hour rainfall total at Kingston was larger than that recorded at Hobart.

The 2018 flood event resulted in significant damage to property around Whitewater Creek, with some residential properties being inundated to depths of 1.5m. Whitewater Creek experienced considerable bed and bank erosion in the Spring Farm area. The Summerleas Road crossing of Whitewater Creek was overtopped by approximately one metre, restricting access for around 4,500 residents. There was extensive damage to Channel Court shopping centre (Photo 1 and Photo 2).

Photo 1: Damage to Channel Court Shopping Centre due to May 2018 flood (https://www.abc.net.au/news/2018-05-14/flood-clean-up-inside-channel-court-kingston/9758324)

Photo 2: Entry to Channel Court Shopping Centre showing damage due to May 2018 flood (https://www.themercury.com.au/business/hobart-floods-kingston-cleanup-begins-after-night-of-drama/news-story/5c17096873811b050c78889c8f0ab57d)

2.2.1. Previous Studies

The Kingston Beach Flood Study (2015) included modelling of Whitewater Creek. Flood mapping in this report includes the area of Whitewater Creek immediately upstream of the confluence with Browns River only.

3. DATA

3.1. Overview

The first stage in the investigation of flooding matters is to establish the nature, size and frequency of the problem. On larger river systems there may be stream height and historical records dating back over a considerable period, in some cases over one hundred years. However, in smaller catchments, such as the Whitewater Creek Catchment, stream gauges and/or official historical records are generally not available, and there is more uncertainty about the frequency and magnitude of flood problems. Additionally, overland flooding in urban areas, such as the downstream part of Whitewater Creek Catchment (Kingston CBD), is highly dependent on localised changes to development, intensification of development (i.e. increased building sizes and more paved surfaces), and localised drainage features such as kerbs and guttering in roadways. These features are subject to relatively frequent modification and renewal, making it difficult to compare flood behaviour over time.

There is one rain gauge, Kingston (Greenhill Drive), within the catchment that recorded rainfall data for the May 2018 flood event. However, only daily data are available at this site and there are no sub-daily pluviograph records. The closest rain gauge with sub-daily records is Blackmans Bay Treatment Plant situated 4 km southeast of Kingston CBD. The pluviograph data from Blackmans Bay gauge for the 2018 event was scaled according to the ratio of event total rainfall (two-day total) between Kingston and Blackmans Bay. An understanding of historical flooding was obtained from an examination of rainfall records, photos and CCTV.

Airborne Light Detection and Ranging (LiDAR) data acquired in 2011 was utilised for catchment delineation and hydraulic modelling. Design and survey data supplied as part of the study, as well as the survey conducted for Kingston CBD during this project, were of mixed usability, and were used for updating LiDAR-based Digital Elevation Model (DEM) and informing other parameter setups. There were gaps in other datasets, including the Council GIS database where inverts of pits and pipes were only partially available. Such gaps are common for flood studies, since collection of detailed information about drainage networks is expensive and time consuming, and often beyond the resources available to Council. As part of this study, analysis of the available data along with site visits were undertaken to address the limitations of the data in key areas.

It should be recognised that while the information about the drainage system for this study is not perfect, this is often not a critical issue, since the majority of runoff cannot be contained within the formal drainage network, especially for the events modelled for this study. Sub-surface drainage networks are typically only designed to cater for the 20% to 5% AEP flow. Therefore, caution must be exercised when applying the broad catchment modelling results at individual properties, particularly for smaller floods or in areas where the pit/pipe drainage network plays a significant role in the flood behaviour.

3.2. Data Sources

Data utilised in the study has been collated from a variety of sources. Data provided by Council and collected from other sources are summarised in Table 1.

Table 1: Summary of Data

Data	Format	Comments	Source
Bridges	.tab	Point layer with limited dimensional information available	Kingborough Council
Culverts	.tab	Point layer with no cross-section information available; not covering all key culverts	Kingborough Council
Stormwater_Pit	.tab	Point layer with type information; no invert level information	Kingborough Council
Stormwater_Pipe	.tab	Line layer with diameter and limited invert level information; no cross-section type information, e.g., circulate or rectangular	Kingborough Council
Propmap	.tab	Property boundary layer (cadastre)	Kingborough Council
Easements	.tab	Easement layer	Kingborough Council
Local_Gov_Reserve	.tab	Local government reserves	Kingborough Council
planning_zone	.tab	Planning zone layer	Kingborough Council
Subdivisions	.tab	Approved subdivisions	Kingborough Council
Contour_5m	.tab	5m surface contours	Kingborough Council
Survey_Control	.tab	Survey control points	Kingborough Council
Spring_Farm_proposed_boundary	.tab	Property boundary of Spring Farm	Kingborough Council
Spring_Farm_proposed_easements	.tab	Easement layer of Spring Farm	Kingborough Council
municipality_kingborough	.tab; .shp	Kingborough municipality region	The LIST
list_hydline_kingborough	.tab; .shp	Watercourse	The LIST
LiDAR	.asc	Mt Wellington LiDAR data; 1m resolution; created on 20-01-2011	The LIST
DEM for Twin Ovals	.asc	0.14m resolution	Kingborough Council
Kingston Park Designs and TINs	.dwg; .pdf	Design for Kingston Park development with surface TIN information; proposed land fill TIN at northeast of the site	Kingborough Council
Whitewater Park	.dwg	Design for Stage 1 to Stage 3 with land fill contours and road strings	Kingborough Council
Spring Farm Designs	.dwg; .pdf	Design for Stage 1 to Stage 8 with land fill contours and road strings; no TIN file for design surface; pdf for Stage 8A	Kingborough Council
Java Head Road Bridge	.pdf	Design for Bridge at Java Head Road in Spring Farm	Kingborough Council
Bridge Drawings - B1260 – Channel Hwy	.pdf	Design for Bridge at Channel Highway	Kingborough Council
Bridge Drawings - B5713 Southern Outlet	.pdf	Design for Bridge at Southern Outlet	Kingborough Council
Walk bridge DS Summerleas Rd	.dwg; .pdf	Design for two Bridges downstream of Summerleas Road	Kingborough Council
Walk bridge DS Huon Hwy	.dwg; .pdf	Design for Bridge downstream of Huon Highway	Kingborough Council
Survey for Channel Court	.e57; .asc	Laser scan survey for flood path through Channel Court	Swanson Surveying
Flood height outputs from Kingston Beach Flood Study	.csv	Flood height outputs at confluence of Whitewater Creek and Browns River used as downstream boundary conditions	Kingborough Council

Flood photos for May 2018 event	.jpg	Flood photos for May 2018 event along Whitewater Creek and in Kingston CBD area	Kingborough Council
Site visit photos	.jpg	Photos taken during site visit on 01 and 02 Oct 2019	WMAwater
CCTV in Channel Court	.mp4	CCTV in Channel Court for 2018 event	Kingborough Council
Rainfall Daily	.csv	Rain gauge records in/around the catchment including Kingston (Greenhill Drive), for May 2018 event	Bureau of Meteorology
Rainfall Pluviograph	.csv	Rain gauge pluviograph records in/around the catchment including Blackmans Bay for May 2018 event	Bureau of Meteorology
Aerial imagery	.jpg	Nearmap Aerial imagery	Nearmap

3.3. Topography

3.3.1. LiDAR Data

Mt Wellington LiDAR, covering the catchment and its immediate surroundings, was obtained from Land Information System Tasmania (LIST) managed by Land Tasmania, a division of Department of Primary Industries, Parks, Water and Environment (DPIPWE). It was indicated that the data were collected in 2011. These data typically have accuracy in the order of +/- 0.15 m (for 70% of points) in the vertical direction on bare earth. The accuracy of the LiDAR data can be influenced by the presence of open water or vegetation (tree or shrub canopy) at the time of the survey.

The LiDAR-based DEM data, as shown in Figure 2, was used as the primary data set for hydrologic catchment delineation and hydraulic modelling.

The LiDAR was flown in 2011, therefore, it does not accurately reflect catchment morphology under the current state of development. The last nine years have seen an increase in residential development and road infrastructure within the downstream part of Whitewater Creek Catchment, including the Kingston CBD, resulting in relatively significant changes to catchment hydrology and hydraulics. The aerial imageries shown in Diagram 1 illustrate the land development from 2011 – 2019.

3.3.2. Design and Survey

To compensate for the lack of representation of current topography by the LiDAR-based DEM, information from the design drawings, additional DEM, and field survey were extracted and integrated. These include:

- design drawings for Whitewater Park, Spring Farm Development, and Kingston Park provided by Council
- additional DEM for Twin Ovals provided by Council
- laser scan survey for flow path through Channel Court, which was conducted by Swanson Surveying for this study.

Based on the above datasets, DEMs for those developed areas were constructed, as illustrated in Figure 3, and were then used to update LiDAR-based DEM for hydraulic modelling.

Diagram 1: Aerial Imagery (Nearmap) in 2011 and 2019. Cyan lines indicate the main areas of development between 2011-2019.

3.3.2.1. Whitewater Park

Three as constructed design drawings (.dwg) for Whitewater Park were provided, hereby called Stages 1 to 3. The following information was extracted from the drawings to create up-to-date topographic data:

- landfill contours (filled depth ≥ 0.3 m), which indicate the land to be filled for the development;
- road strings, including kerb Invert, kerb lip, kerb back, footpath, etc.

Due to the lack of a completed design surface, the landfill contours were used to update 2011 LiDAR-based DEM. Road strings were interpolated into the DEM to reflect the completed road surface. It should be noted that the majority of the landfill areas were within/around road reserves and only landfill with filled depth ≥ 0.3 m were provided, therefore, only limited value was added to the road strings by including land-fill contours.

Based on the assessment of aerial imageries (Nearmap) acquired on 06/03/2011, 06/03/2018, and 22/12/2019, drawings for Stage 1 was used to update surface topography to May 2018 conditions for calibration event modelling, and drawings for Stages 1 – 3 were used to update surface topography to existing conditions for design events modelling, as shown in Figure 3.

3.3.2.2. Spring Farm

The as constructed design drawings for Stages 1 – 8 were provided in DWG format, while the drawing for Stage 8A was provided in PDF format. Similar to procedure applied for Whitewater Park, the landfill contours (filled depth ≥ 0.3 m) and road strings for Spring Farm Development were extracted and used to create up-to-date topographic data.

Drawings for Stages 1 – 5 were used to update surface topography to May 2018 conditions for calibration event modelling, and drawings for Stages 1 – 8A were used to update surface topography to existing conditions for design events modelling as shown in Figure 3.

The civil design for Spring Farm Development (up to Stage 8A) is illustrated in Diagram 2 as an example.

3.3.2.3. Kingston Park

In addition to the design drawings, illustrated in Diagram 3 as an example, the surface design TIN file for Kingston Park Development and the TIN file for the proposed landfill at the northeast part of the site were provided.

The surface design TIN file was directly used to create the existing conditions DEM for this area. The landfill TIN file was not included in existing conditions modelling but was assessed for flood impacts as future developed conditions. The created DEM was illustrated in Figure 3.

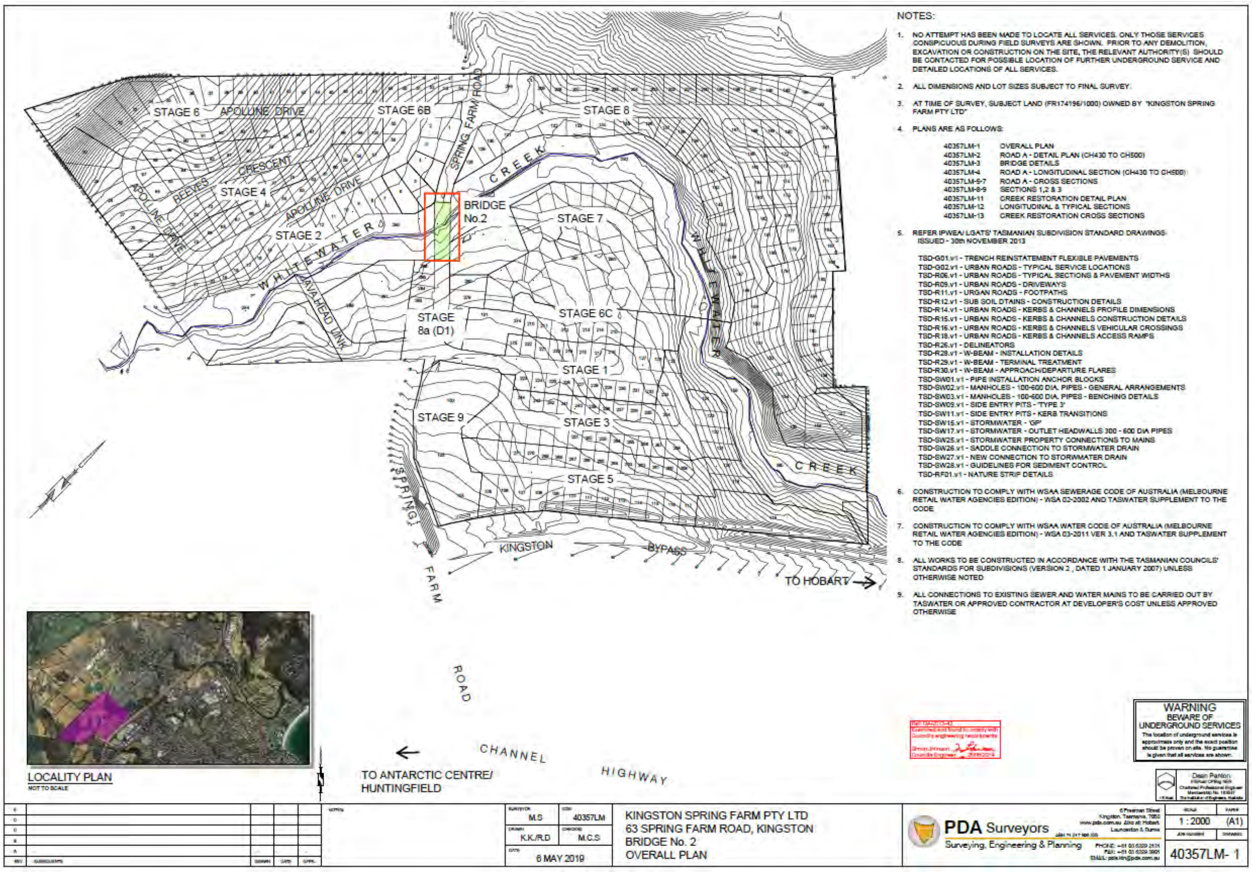


Diagram 2: Concept Plan for Spring Farm Development (PDA Surveyors).

Diagram 3: Concept Plan for Kingston Park.

3.3.2.4. Twin Ovals

The addition DEM covering the area from Twin Ovals to Drysdale Avenue Park was provided by Council with grid size of 0.143 m. A comparison between Twin Ovals DEM and LiDAR-based DEM indicated that the two are relatively consistent with each other with the Twin Ovals DEM showing more details in some area, e.g., road reserve. However, it was found that the LiDAR-based DEM showed better representation of a minor tributary between Nolan Crescent and Willowbend Road, therefore, the tributary area was trimmed from Twin Ovals DEM, as shown in Figure 3.

3.3.2.5. Channel Court

Channel Court, in the CBD of Kingston, was flooded in 2018 flood event. According to the aerial imageries as shown in Diagram 4, there was some development from 2011 – 2019, especially for the shopping centre at the southwest of Channel Court. Considering the lack of design/survey information and significance of flooding issues in this area, a Laser Scan survey was conducted in January 2020 by Swanson Surveying engaged by WMAwater, on behalf of Council.

The DEM for the main flow path, including the ground floor within the shopping centre, was constructed based on the survey data and used for updating LiDAR-based DEM in this area, as shown in Figure 3.

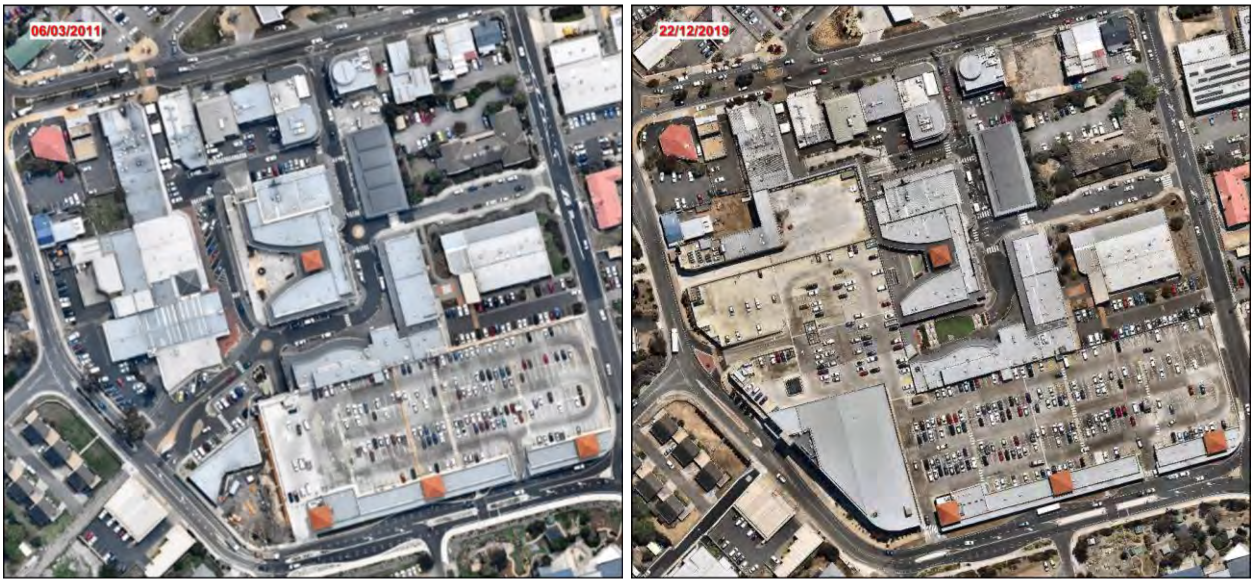


Diagram 4: Aerial Imagery (Nearmap) in 2011 and 2019 – Channel Court.

3.4. Hydraulic Structures

Structures including bridges and culverts can have a significant impact on flood behaviour. Therefore, appropriate representation of these structures is essential for the accuracy of the hydraulic model.

Infrastructure characteristics are partially documented in the GIS dataset provided by the Council, however, some key dimensions, i.e., invert levels, cross-sectional area, etc. are missing. Design information were provided by Council for the following structures along Whitewater Creek:

- Java Head Road bridge in Spring Farm
- Spring Farm Road bridge in Spring Farm
- two footbridges downstream of Summerleas Rd
- Southern Outlet box culvert
- Channel Highway bridge

The identification of key bridge and culverts and their representation in hydraulic model are detailed in Section 4.3.5.1.

3.5. Underground Drainage

GIS information pertaining to the underground drainage system, including pits and pipes, was provided by Council. Pipe diameters are generally provided while only part of the pipes/pits have invert level information available.

Additional pits and pipes information for stormwater assets within Kingston Plaza, which was known to be not flooded during 2018 event, were extracted from a photocopy of the design drawings, as shown in Diagram 5, to properly model the drainage capacity in this area.

The integration of pits and pipes for hydraulic modelling was detailed in Section 4.3.5.2.

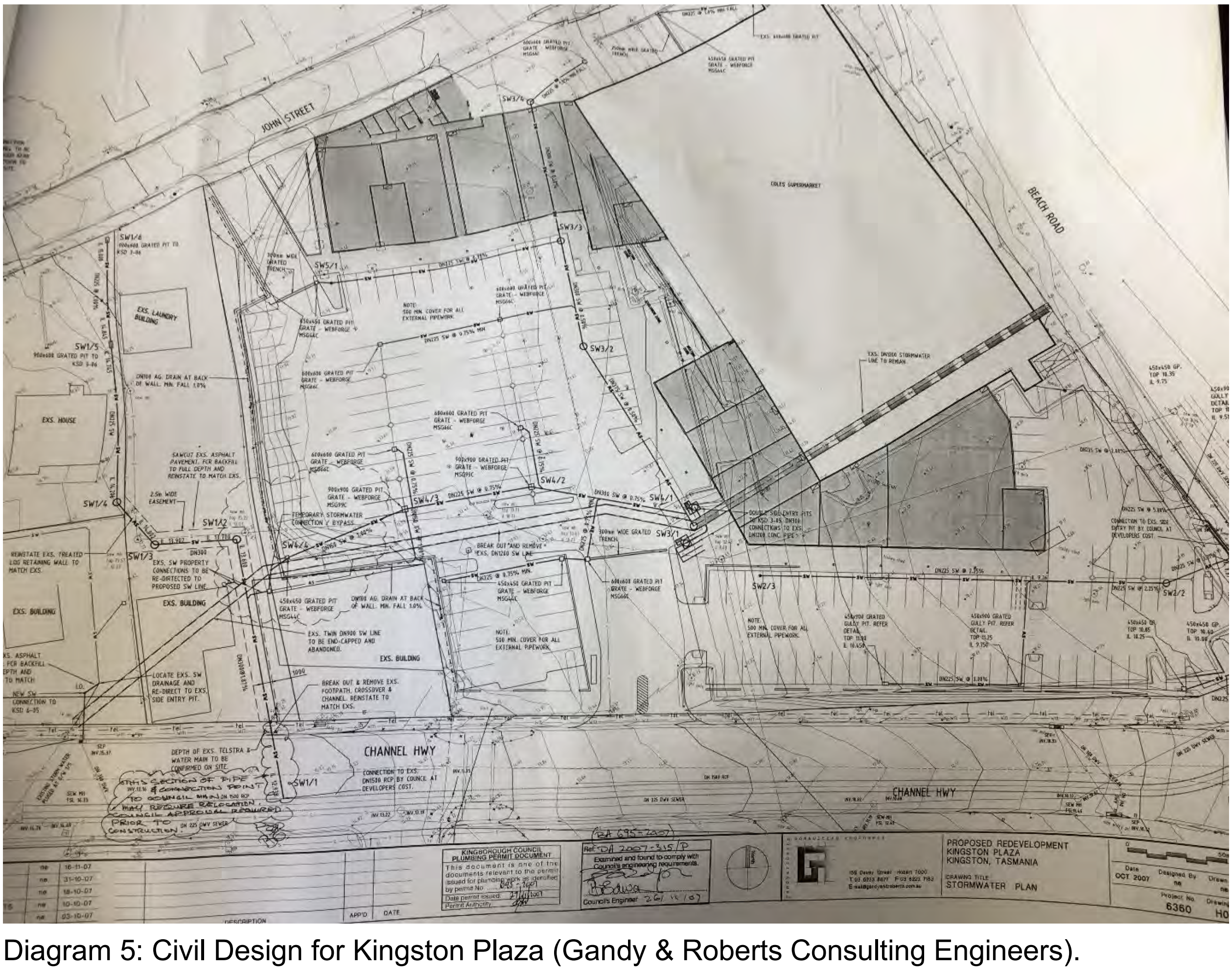


Diagram 5: Civil Design for Kingston Plaza (Gandy & Roberts Consulting Engineers).

3.6. Historical Rainfall Data

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). In many locations, long periods of daily rainfall data is available. However, pluviometers have generally only been installed for widespread use since the 1970s at limited locations. 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.

Rain gauges with rainfall records available from Australian Bureau of Meteorology (BoM) are shown in Diagram 6. There is one rain gauge, Kingston (Greenhill Drive), within the catchment with rainfall data for the May 2018 flood event. However, only daily data are available and there are no pluviograph records. Red and blue circles in Diagram 6 highlight gauges with pluviograph records for the May 2018 event. For this study, the pluviograph record from Blackmans Bay Treatment Plant, which is the closest pluviometer (4 km away from Kingston CBD) to the study catchment, was selected to reconstruct the May 2018 rainfall event.

As shown in Table 2, the two-day total rainfalls (9am 9/5/2018 – 9am 11/5/2018) based on BoM's daily rainfall data, covering the major period of the event, are 180.2 mm at Kingston and 149.8 mm at Blackmans Bay, respectively. The large differences in rainfalls at the two sites means that the data from Blackmans Bay cannot be applied directly to the catchment. Therefore, the pluviographic data from Blackmans Bay for May 2018 event was scaled according to the rate of two-day total between Kingston and Blackmans Bay, i.e., 180.2 mm / 149.8 mm, to be implemented as catchment wide pluviographic data for modelling.

It should be noted that there was small amount of rainfall recorded between 9am and 4pm on 11/5/2018 at Blackmans Bay pluviograph, however that amount was not recorded in daily rainfall at either Kingston or Blackmans Bay. Therefore, it is assumed that the rate of two-day rainfall (9am 9/5/2018 – 9am 11/5/2018) is the most appropriate information and can be used to scale the pluviographic data.

Table 2: Total Rainfall at Kingston and Blankmans Bay

Station Number	Station Name	Two-day total based daily data (9am 9/5/2018 – 9am 11/5/2018)	Event total based on pluviograph data (8am 10/5//2018 – 4pm 11/5/2018)
094222	Kingston (Greenhill Drive)	180.2 mm	-
094163	Blackmans Bay Treatment Plant	149.8 mm	124.6 mm

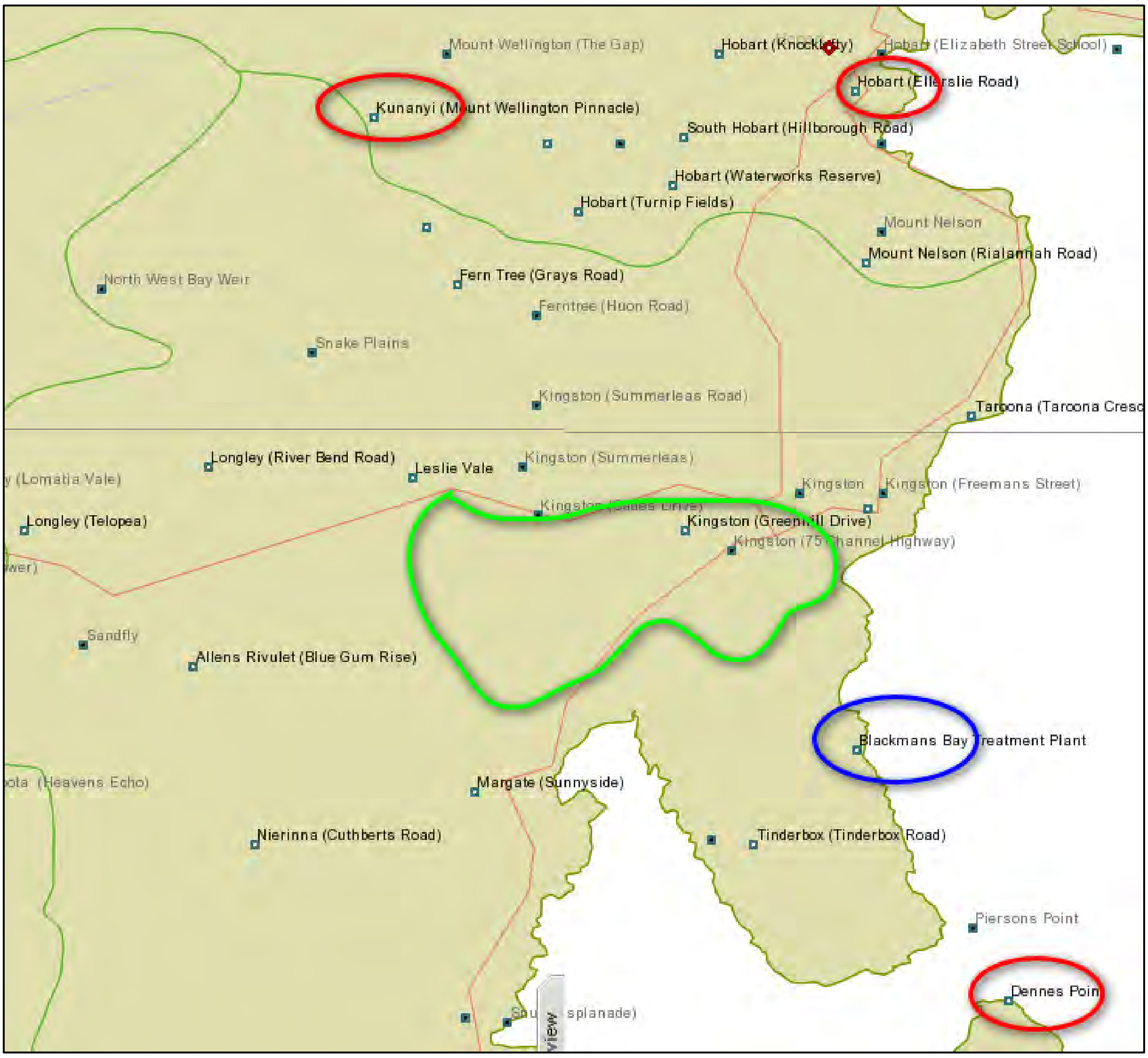


Diagram 6: Rain Gauges within and near the Study Catchment. Green line indicates the catchment location. Red and blue circles highlight gauges with pluviograph records for May 2018 event, while blue circle indicates the pluviograph data selected for use.

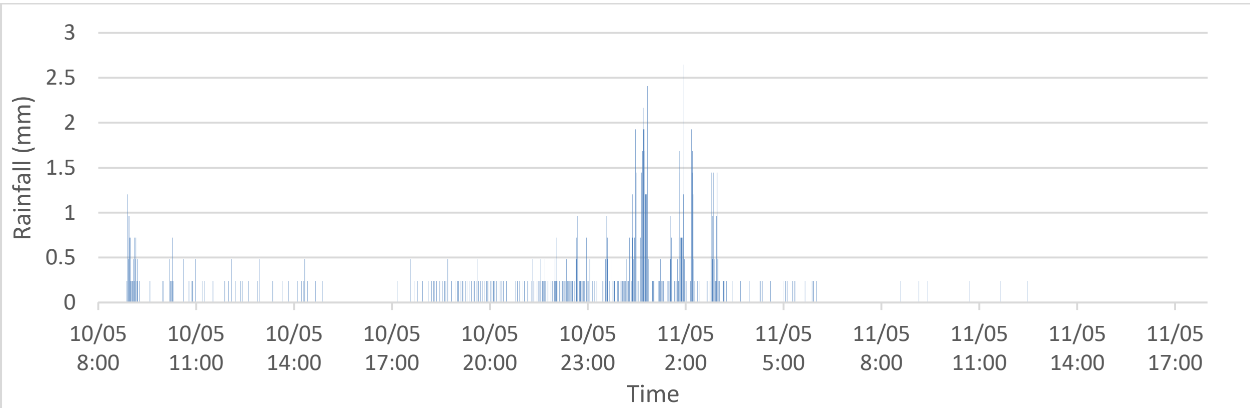


Diagram 7: Constructed Pluviograph at Kingston (Greenhill Drive) for May 2018 Event.

3.7. Design Rainfall and Losses

The catchment is covered by six (6) design rainfall intensity-frequency-duration (ARR 2016 IFD) grids, as illustrated in Diagram 8. The IFDs downloaded from BoM's 2016 IFD data page are shown in APPENDIX B.

The rural loss parameters were obtained from the ARR datahub and are provided in Table 3. These values were assessed and used for hydrologic and hydraulic modelling, detailed in Sections 4.2.2 and 4.3.1, respectively.

Table 3: ARR Losses at Catchment Centroid

Storm Initial Losses (mm)	Storm Continuing Losses (mm)
28.0	3.4

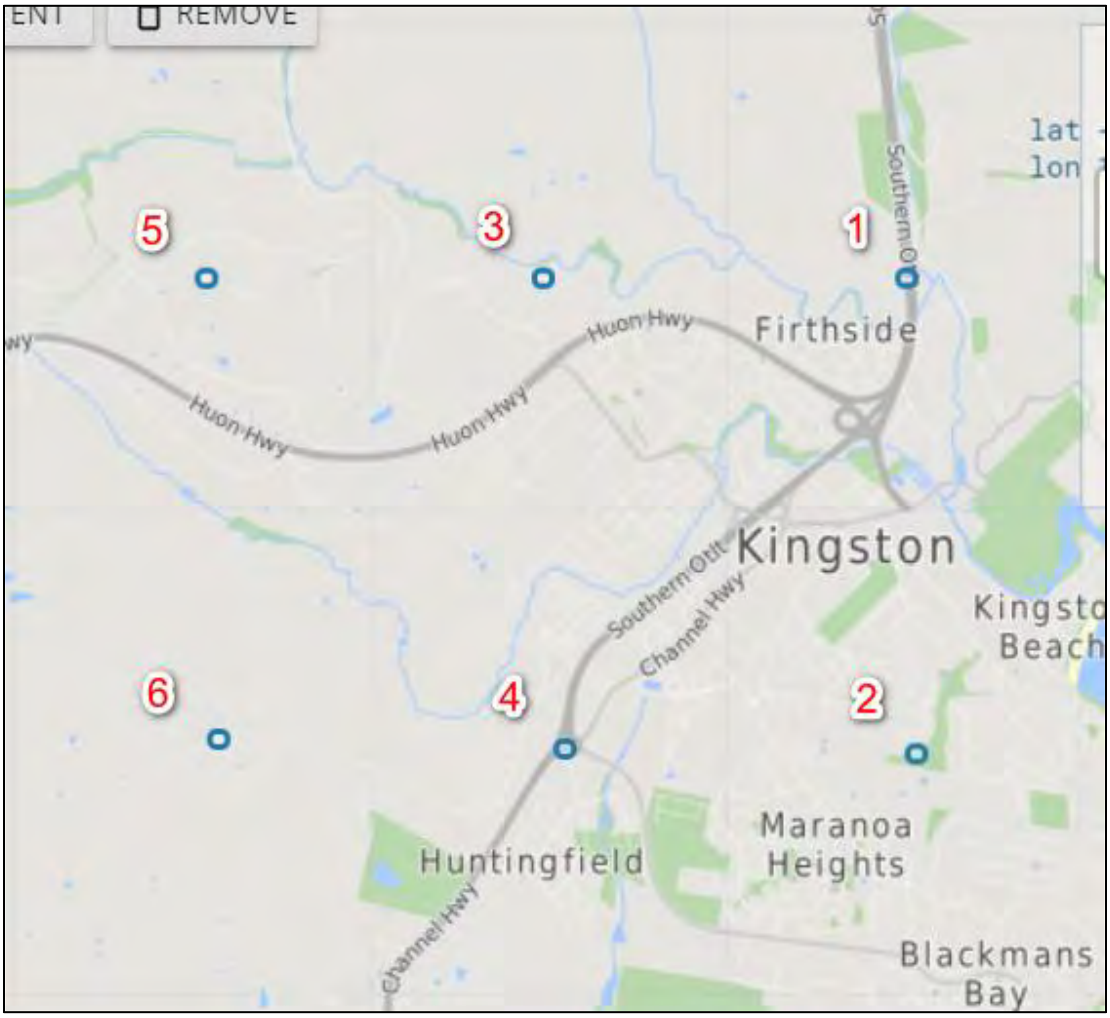


Diagram 8: BoM 2016 IFD Grids Covering the Catchment.

3.8. Photos and Videos

Photos provided by the Council for the May 2018 event and obtained during the site visit were geographically registered in the MapInfo workspace, as illustrated in Figure 4. Those photos were used to identify key hydraulic features, and photos with flood marks were used for model calibration.

CCTV in Channel Court during May 2018 event was provided by Council and was used for model validation purpose.

4. MODEL SETUP

4.1. Model Schematisation

The Whitewater Creek catchment was split into an upper catchment and a lower catchment. The upper catchment is a rural catchment covered by forests, pastures, and rural resources, whilst the lower catchment is urbanised with a mix of pervious and impervious surfaces and piped and overland flow drainage systems.

A coupled hydrologic-hydraulic model was developed to address the complex runoff generation and routing processes in the catchment, with the upper catchment simulated by a hydrologic model, i.e., the Watershed Bounded Network Model (WBNM), and the lower catchment simulated by a hydraulic model, i.e., a Direct Rainfall Method (DRM) based TUFLOW model. The output hydrographs of the upper catchment were used as upstream boundary condition for the lower catchment. Figure 5 illustrates the model schematisation of the whole catchment.

4.2. Hydrologic Model

Inflow hydrographs serve as inputs at the boundaries of the hydraulic model. In a flood study where long-term gauged streamflow records are not available, a hydrologic model (simulating rainfall-runoff and runoff routing processes) is generally used to provide these inflows. A range of hydrologic models are available as described in ARR 2019 (Reference 2). These models allow the rainfall depth to vary both spatially and temporarily over the catchment and readily lend themselves to calibration against recorded data.

Hydrologic modelling was undertaken using WBNM (Reference 3), a widely utilised hydrologic modelling software. The WBNM model includes a relatively simple but well supported method, where the routing behaviour of the catchment is primarily assumed to be correlated with the catchment area.

The WBNM model can be calibrated to streamflow data through adjustment of various model parameters including the stream lag factor, storage lag factor, and/or rainfall losses. Due to the absence of streamflow data it was not possible to perform an independent calibration of the hydrologic model to observed flows.

A hydrologic model for the upper catchment was created and used to calculate upstream boundary inflows for lower catchment hydraulic model.

4.2.1. Sub-catchment delineation

Delineation of sub-areas was carried out by applying a mathematical algorithm called Terrain analysis using Digital Elevation Models (TauDEM, Reference 4) to topographic data sets.

TauDEM is a suite of DEM tools for the extraction and analysis of hydrologic information from topography as represented by a DEM. LiDAR-based DEM was used for TauDEM analysis as there has not been notable development in the upper catchment since 2011. Please refer to

Section 3.3.1 for a detailed description of the LiDAR-based DEM used here.

TauDEM provides the distinct advantage of applying an objective technique to calculate the stream flow paths and directions, the contributing areas using both single and multiple flow direction methods, as well as to delineate the catchments and sub-catchments draining to each stream segment.

The LiDAR-based DEM, analysed in TauDEM, extended far enough to ensure that the entire contributing catchment area was defined.

The upper catchment covered by the hydrologic model is approximately 6.2 km^2. This was delineated into 32 sub-catchments with an average sub-catchment size of 19.44 hectares. The sub-catchment delineation is shown in Figure 5.

4.2.2. Losses and WBNM Lag Parameters

Methods for modelling the proportion of rainfall that is “lost” to infiltration are outlined in ARR 2019 (Reference 2). The methods are of varying degrees of complexity, with the more complex options only suitable if sufficient data are available. The method most typically used for design flood estimation is to apply an initial loss (IL) and continuing loss (CL) to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

Rainfall losses from a paved or impervious area are considered to consist of only an initial loss (a sufficient amount to wet the pavement and fill minor surface depressions). Losses from grassed and vegetated areas are comprised of an initial loss and a continuing loss. Based on the assessment of the planning zone GIS layer and the latest aerial imagery (Nearmap), the upper catchment is a purely rural catchment covered by forest, pasture, and environmental resources. Therefore, 100% fractional pervious was assumed and the rural losses from ARR data hub as detailed in Table 3 were directly used for hydrologic modelling.

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 WBNM parameters selected are summarised in Table 4. A value of 1.6 was selected for catchment lag to reflect relative steep slope in the upper catchment. Those parameters were validated in calibration (Section 5).

Table 4: Adopted WBNM Parameters for Calibration and Design

WBNM Parameters	Value
Lag Parameter (C)	1.6
Stream Lag Factor (natural channels)	1.0

4.3. Hydraulic Model

Hydraulic modelling was undertaken for the lower catchment (Figure 5) using TUFLOW HPC (build 2018-03-AC-iSP-w64) with Graphics Processing Unit (GPU) solver (Reference 5), a widely utilised 1D and 2D flood simulation software. Hydrographs from the WBNM hydrologic model at the interface between upper and lower catchments were input as upstream inflow into the TUFLOW model. Rainfall was directly applied onto the lower catchment as internal boundary conditions. Hydraulic modelling was carried out on a fixed 2 m grid.

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

In addition to 2D modelling of overland flows, TUFLOW can model drainage elements (pipes) as 1D elements as well as modelling creeks or open channels in 1D if required. The 1D and 2D components of the model can be dynamically linked during the simulation. In TUFLOW the ground topography is represented as a uniformly spaced grid with a ground elevation and a Manning's '*n*' roughness value assigned to each grid cell. The grid cell size is determined as a balance between the model result definition required and the computer run time (which is largely determined by the total number of grid cells, and the number of "wet" cells). A cell size of 2 m by 2 m was found to provide an appropriate balance for this study.

The TUFLOW hydraulic model extends upstream to where Whitewater Creek enters the Whitewater Park and Spring Farm area, and downstream to the confluence with Browns River. The total area included in the 1D/2D model covers 7.2 km^2 and the extents of the TUFLOW model are shown in Figure 5.

4.3.1. Rainfall

Rainfall records were directly applied to the lower catchment. Pre-burst and burst rainfall were both used for modelling.

For May 2018 event, the pre-burst and burst rainfall was constructed by scaling pluviograph data from Blackmans Bay to the Whitewater Creek Catchment by multiplying the rate of two-day rainfall between Kingston (Greenhill Drive) and Blackmans Bay, as detailed in Section 3.6.

For design events, burst rainfall for each event was constructed by applying the temporal pattern downloaded from ARR data hub to BoM 2016 IFD. The pre-burst for each event obtained from ARR data hub was uniformly distributed within 1hr a priori the burst, to allow the hydraulic model to simulate the pre-burst with enough flow propagation time. For instance, a 4.5-hour event was actually modelled with 5.5-hour rainfall input, i.e., 1-hour pre-burst plus 4.5-hour burst.

As shown in Diagram 8, the lower catchment, which was modelled through hydraulic model, was covered by four IFD grids, i.e., Grids 1 – 4. Therefore, the average IFDs of Grids 1 – 4 were used for hydraulic modelling for design events.

4.3.2. Losses

Since pre-burst rainfall was modelled directly in the hydraulic model, losses estimated for the lower catchment were also directly applied without subtracting pre-burst. Application of losses in the DRM hydraulic modelling differs from that for hydrologic modelling. The differences are in two aspects:

- hydraulic model will account for part of the storage loss by itself; and
- the spatial variation cannot be represented through sub-catchments.

4.3.2.1. Storage Loss by Hydraulic Model

As detailed in Chapter 11 of ARR 2016 Project 15 (Reference 6), in the process of rainfall becoming runoff and flow, catchment water losses occur due to a number of processes, including interception, infiltration, evaporation, transpiration and storages. It is expected that surface topography employed in DRM hydraulic model will account for part of the Initial Loss (IL) caused by storage. Therefore, the initial loss should be lower in a direct rainfall model when compared with a traditional initial and continuing loss hydrological model.

To quantify the IL in the hydraulic model, a synthetic DRM modelling assessment was conducted for the upper catchment. The upper rural catchment was used for assessment to avoid any impact by urban impervious area. A 1-hour 1% AEP event was modelled, and the IL and CL were both set to 0. The model was run for 24 hours to ensure all rainfall was converted and routed to the outlet of the catchment except for the storage loss accounted for by the surface topography. The storage loss accounted for by the surface topography was quantified to be **2.46 mm**, through the following equation:

$$\text{Storage Loss}_{\text{modelled}} = \text{Rainfall Depth} - \text{Runoff Depth} \qquad \text{Equation 1}$$

The IL of pervious area for lower catchment DRM hydraulic model was then quantified to be **25.54 mm**, through subtracting the storage loss accounted by the model from the rural IL obtained from ARR data hub (**28 mm**), i.e.,

$$IL_{DRM} = IL_{ARR} - \text{Storage Loss}_{\text{modelled}} \qquad \text{Equation 2}$$

The CL of previous area for lower catchment DRM hydraulic model was set to be **3.4 mm**, adopted directly from ARR data hub CL.

4.3.2.2. Losses Based on Planning Zones

The IL_{DRM} / CL quantified above are for 100% pervious areas. Urban area, such as the lower catchment, typically has a mix of pervious and impervious surfaces, and impervious surfaces normally have significant lower IL/CL. Based on the methodology in ARR 2019 (Reference 2), IL/CL were set for three different surface types, i.e., Pervious Area, Indirectly Connected Area, and Effective Impervious Area, as shown in Table 5.

Table 5: Losses and Assumptions for Three Surface Types

Surface Type	IL (mm)	CL (mm)
Pervious Area	**25.54** (IL $_{ARR}$ – Storage Loss $_{modelled}$)	**3.4** (ARR data hub)
Indirectly Connected Area	**17.14** (0.7 × IL $_{ARR}$ – Storage Loss $_{modelled}$)	**2.5** (ARR suggestion)
Effective Impervious Area	**0** (IL accounted by Storage Loss $_{modelled}$)	**0** (ARR suggestion)

It is very difficult to accurately delineate the three types of surfaces for the modelling extent. In this study, a single IL/CL was estimated for each planning zone based on the estimated percentages (%) of the three surface types, as shown in Table 6. The percentages (%) of the three surface types for each planning zone were estimated by visual inspection of sample aerial imageries (Nearmap), as shown in Table 7.

The planning zone GIS layer provided by the Council was firstly stamped by road reserve from the cadastre, and then refined against most recent aerial imagery to produce an up-to-date planning zone database for losses schematisation, as shown in Figure 6.

Table 6: Estimated Percentage (%) of Three Surface Types and Adopted Losses for Planning Zones

Planning Zone	IL (mm)	CL (mm)	Effective Impervious Area (%)	Indirectly Connected Area (%)	Pervious Area (%)
General Residential	11.12	1.59	40	50	10
Inner Residential	5.98	0.84	70	20	10
Low Density Residential	17.09	2.40	15	55	30
Rural Living	23.42	3.14	5	10	85
Environmental Living/Forest	25.54	3.40	0	0	100
Urban Mixed Use	17.07	2.36	20	40	40
Community Purpose	11.96	1.68	40	40	20
Recreation	22.58	3.05	5	20	75
Open Space	24.70	3.31	0	10	90
Local Business	5.98	0.84	70	20	10
General Business	5.98	0.84	70	20	10
Central Business	1.71	0.25	90	10	0
Commercial	5.98	0.84	70	20	10
Light Industrial	5.98	0.84	70	20	10
Rural Resource	25.12	3.36	0	5	95
Utilities / Road Reserve	7.70	1.09	60	30	10
Environmental Management	25.54	3.40	0	0	100

Table 7: Sample Imageries (Nearmap) for Planning Zones

 General Residential	 Inner Residential
 Low Density Residential	 Rural Living
 Environmental Living/Forest	 Urban Mixed Use
 Community Purpose	 Recreation

 Open Space	 Local Business
 General Business	 Central Business
 Commercial	 Light Industrial
 Rural Resource	 Utilities / Road Reserve

 Environmental Management	

4.3.3. Boundary Conditions

Flows from sub-catchments C028, C031, and C032, including their upstream sub-catchments contribution were used as upstream inflow to the hydraulic model. The locations of the three upstream inflow locations are shown in Figure 7. Rainfall was applied within the lower catchment as internal boundary conditions.

Two different type of downstream boundary conditions (Figure 7) were utilised in the model:

- **HQ Boundaries** – The outflow from this boundary is dependent on water level, using a rating curve in which the topographic gradient is assumed to equal the water level gradient (i.e. uniform flow); and
- **HT Boundary** – The water level at the boundary, which can be specified as a static or a varying water level over time.

The 5% AEP and 1% AEP peak flood levels at the confluence of Whitewater Creek and Browns River were extracted from the Kingston Beach Flood Study (Reference 1). The extracted 5% AEP peak flood level was used as a static downstream HT boundary condition at Browns River for 5%, 1%, and 0.5% AEP design events modelling, while the extracted 1% AEP peak flood level was used as downstream boundary condition for May 2018 flood event modelling. While limitations exist when using static flood levels from previous study (ARR 1987) as downstream boundary conditions, it was found that the boundary conditions had only a limited effect on modelling results upstream of Huon Highway.

Model extent was set slightly wider than the lower catchment extent, to minimize the impact of inter-catchment flow. HQ boundaries were applied around the model extent except for the confluence of Whitewater Creek and Browns River, so that the flow towards the neighbour catchments can flow out of the model extent freely.

4.3.4. Surface Roughness

The hydraulic efficiency of the flow paths within the TUFLOW model is represented in part by the hydraulic roughness or friction factor formulated as Manning's '*n*' values. This factor describes the net influence of bed roughness and incorporates the effects of vegetation and other features which may affect the hydraulic performance of the flow path.

The Manning's '*n*' values were applied according to planning zones described in Section 4.3.2.2 with certain update. The updates of the planning zones were in two aspects:

- The schematisation of Manning's '*n*' within the Channel Court shopping centre was refined with survey information, e.g., flow path on ground floor was assigned a low value while the shelves in Woolworth were assigned a high value; and
- The urbanised area outside the inundated (Whitewater Creek and Kingston CBD Tributary) area was extracted and a single low value was assigned.

The revised planning zones used for Manning's '*n*' schematisation are shown in Figure 8. The adopted Manning's '*n*' value for each zone is summarised in Table 8. These values have been adopted based on aerial imagery and site inspection, past experience in similar floodplain environments, and suggestions in ARR 2016 Project 15 (Reference 6).

It should be noted that the large Urban Non-inundated Area (Figure 8 was assigned a Manning's '*n*' of 0.02 based on the advice contained in ARR 2016 Project 15 (Reference 6) for DRM hydraulic models. This assumes that most of the rainfall falling on roofs and other impervious areas will be directly routed to the ground (kerb and channel) and underground (pipes) drainage systems with low roughness and quick routing. ARR 2016 Project 15 suggested a Manning's '*n*' of 0.015 to 0.02 should be applied to non-inundated buildings. As planning zones were used to assign Manning's '*n*', the upper value, i.e., 0.02, was adopted.

Table 8 Manning's '*n*' Coefficient

Planning Zone	Manning's '*n*' (constant or depth varying)
General Residential	0, 0, 0.03, 0.02, 0.1, 0.15
Inner Residential	0, 0, 0.03, 0.02, 0.1, 0.15
Low Density Residential	0, 0, 0.03, 0.04, 0.1, 0.10
Rural Living	0, 0, 0.03, 0.08, 0.1, 0.04
Environmental Living/Forest	0.12
Urban Mixed Use	0.05
Community Purpose	0.045
Recreation	0, 0, 0.03, 0.08, 0.1, 0.035
Open Space	0, 0, 0.03, 0.06, 0.1, 0.04
Local Business	0, 0, 0.03, 0.02, 0.1, 0.15
General Business	0, 0, 0.03, 0.02, 0.1, 0.15
Central Business	0, 0, 0.03, 0.02, 0.1, 0.20
Commercial	0, 0, 0.03, 0.02, 0.1, 0.20
Light Industrial	0, 0, 0.03, 0.02, 0.1, 0.15
Rural Resource	0, 0, 0.03, 0.08, 0.1, 0.04
Utilities / Road Reserve	0.03
Environmental Management	0.10
Urban Non-inundated Area	0.02
Channel Court Ground Floor	0.016

NOTE: For depth varying Manning's '*n*', the six values represent – depth (m), '*n*', depth (m), '*n*', depth (m), '*n*'

Sealed roads are typically assigned a Manning's '*n*' of 0.18 to 0.02. However, the Utilities / Road Reserve zone in our schematisation covers buffer areas (i.e., lawn in front of properties) of road reserves, as illustrated in Table 7. Therefore, a Manning's '*n*' of 0.03 was adopted.

The adopted values were validated during the calibration process (Section 5).

4.3.5. Hydraulic Structures

4.3.5.1. Bridges and Culverts

Ten main structures, 5 culverts and 5 bridges, along Whitewater Creek were identified during the site inspection and initial modelling assessment. The locations of the structures are illustrated in Figure 9. These culverts and bridges were partially documented in the GIS dataset provided by the Council, however, some key dimensions, i.e., invert levels, depth, and/or cross-sectional area, were missing. There was limited useful information from the GIS database.

As described in Section 3.4, design information of six (6) structures, including 5 bridges and 1 culvert, were provided by Council. The design information was used to setup the structures in hydraulic model. Where no design information available, i.e., for the other 4 culverts, approximation was made based on visual inspection of photos taken during site visit. Table 9 shows the setup of the 5 bridges in hydraulic model, while Table 10 shows the setup of the 5 culverts in hydraulic model. The photos/aerial imageries and design drawings of the 10 structures used to extract/estimate key dimensions can be seen in Table 11.

Table 9: Hydraulic Model Parameters for Bridges

Parameters		Java Head Rd Bridge	Spring Farm Rd Bridge	Foot Bridge I	Foot Bridge II	Channel Hwy Bridge
Obvert (m AHD) / Depth (m)	L1 Obvert	49.4 - 49.8	47.2 - 47.8	25.6	24.9	2.17
	L2 Depth	1.1	0.8	0.06	0.06	0.2
	L3 Depth	1.0	1.0	1.0	1.0	1.0
Blockage (%)	L1	0	0	6	6	5
	L2	100	100	100	100	100
	L3	25	25	30	30	25
Form Loss Coefficient	L1	0	0	0.02	0.02	0.05
	L2	0.18	0.18	0.13	0.13	0.18
	L3	0.1	0.1	0.06	0.06	0.1

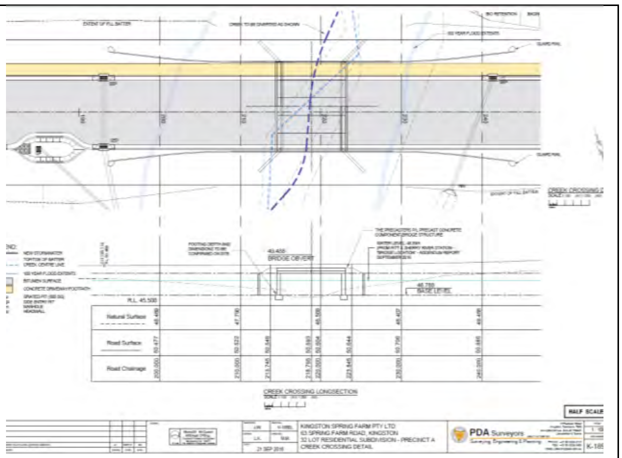
NOTE: Modifiers (2d_zsh) were used to update DEM to the invert levels provided in design drawings. L1 obverts were defined either constant or spatially varying using 2d_lfcsh_p.

Table 10: Hydraulic Model Parameters for Culverts

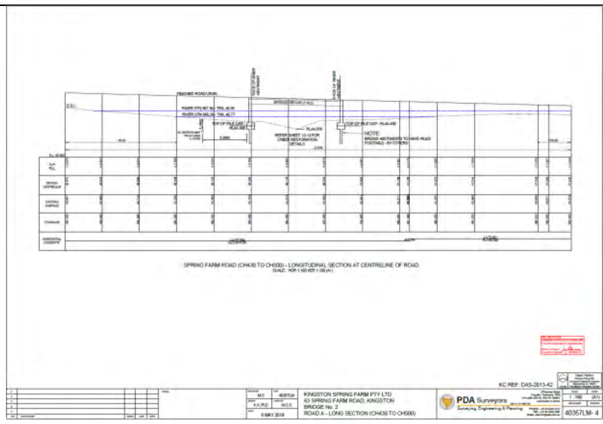
Parameters	Summerleas Rd Culvert	Whitewater Cres Culvert	Sourthern Outlet Tunnel	Sourthern Outlet Culvert	Huon Hwy Culvert
Type	R (Box)	I (Arch)	R (Box)	R (Box)	R (Box)
Width (m)	2	7	4	3	2.5
Height (m)	2	2.8	2.5	3	2.5
Number	2	1	1	1	1

NOTE: Upstream and downstream invert levels were set in accordance the DEM. HW x-section was estimated using width and height based on typical arched culvert assumption for Whitewater Cres Culvert. Widths and heights were estimated based on visual inspection for all culverts, except for Southern Outlet Culvert, where information was extracted from design plan.

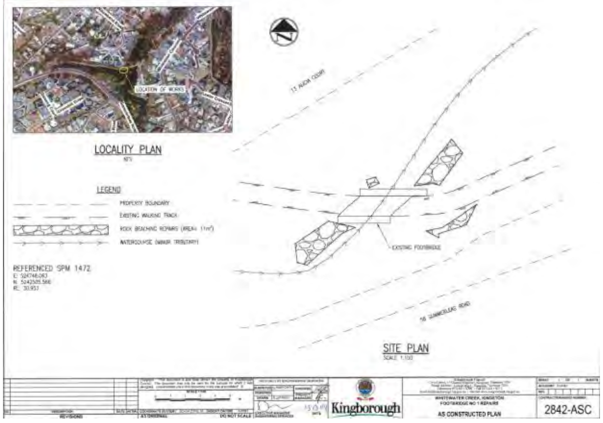
Table 11: Photos/Aerial Imageries and Design Drawings for Hydraulic Structures

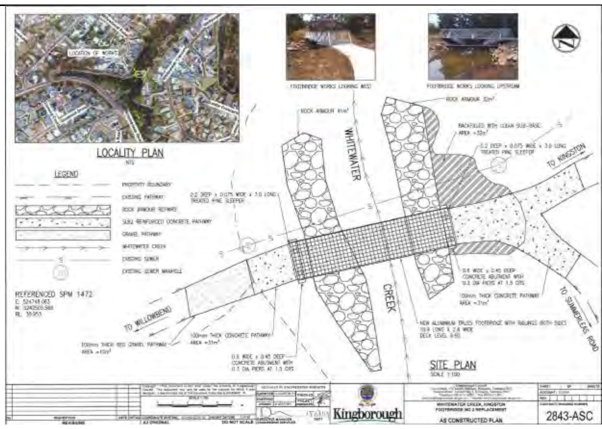
Java Head Road Bridge

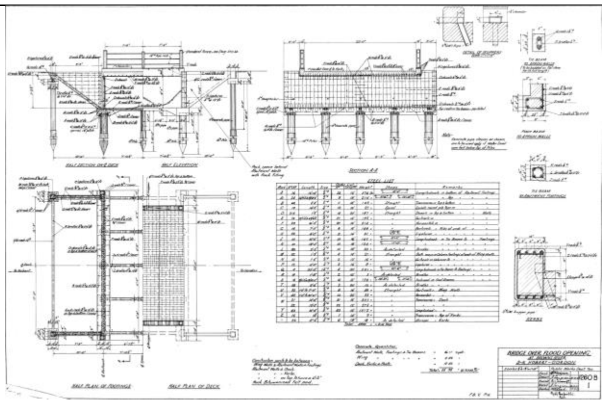
Spring Farm Road Bridge



Foot Bridge I d/s Summerleas Rd



Foot Bridge II d/s Summerleas Rd



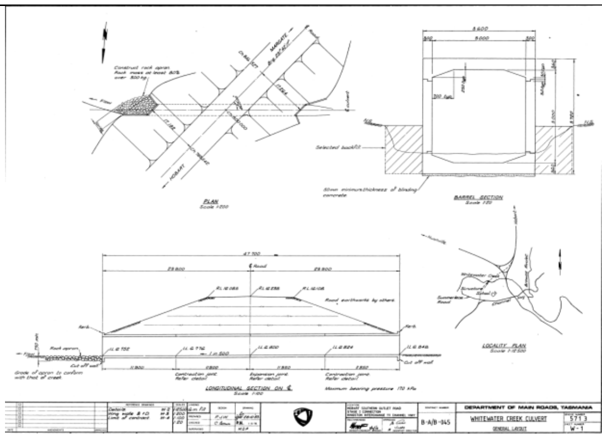
Channel Highway Bridge

Summerleas Road Culvert

Whitewater Crescent Culvert

Southern Outlet Tunnel

Huon Highway Culvert



Southern Outlet Culvert

4.3.5.2. Pits and Pipes

The underground stormwater drainage network was modelled in TUFLOW as a 1D network dynamically linked to the 2D overland flow domain. As the scope of this study was focused on large flood events, i.e., 5%, 1%, and 0.5% AEPs, only major pipes (diameter > 450 mm) were modelled, except for a few certain areas, such as Kingston Plaza, where minor pipes (diameter ≤ 450 mm) were included. The schematisation of the stormwater network is illustrated in Figure 9.

Details of the 1D solution scheme for the pit and pipe network are provided in the TUFLOW user manual (Reference 5). For the modelling of inlet pits the "R" pit channel type was utilised, which requires a width and height dimension for the inlet in the vertical plane. The width dimension represents the effective inlet length exposed to the flow, and the vertical dimension reflects the depth of flow where the inlet becomes submerged, and the flow regime transitions from the weir equation to the orifice equation. For lintel inlets, the width was based on the length of the opening which was assumed to be 1.2 m for all inlet pits. As only major pipes were included, inlet pits connected to minor pipes were clustered to connect to major pipe system.

In cases where invert level data were not available, invert levels at the start and end points of the pipe system were estimated based on pipe diameter and minimum depth. Invert levels in the middle of the pipe systems were interpolated by TUFLOW.

4.3.5.3. Road Kerbs and Gutters

LiDAR typically does not have sufficient resolution to adequately define the kerb and gutter system within roadways. The density of the aerial survey points is in the order of one per square metre, and the kerb/gutter feature is generally of a smaller scale than this, so the LiDAR does not pick up a continuous line of low points defining the drainage line along the edge of the kerb.

To deal with this issue, Reference 6 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."*

The road gutter network plays a key role for overland flow in urbanised area, e.g., the lower catchment. In order to model the system effectively, the gutters were stamped into the mesh using the method described above. The method used was to digitise breaklines along the gutter lines and reduce the ground levels along those model cells by 0.1 m, creating a continuous flow path in the model (Figure 10). This was not applied to areas with design or feature survey data, e.g., Spring Farm, Whitewater Park, Kingston Park, and Channel Court.

For Spring Farm and Whitewater Park, road strings were modelled in hydraulic model (Figure 10) together with DEM constructed based on design drawings (Figure 3). For Kingston Park and Channel Court, only DEM constructed based on design TIN or Survey data were used for modelling (Figure 3).

4.3.6. Building Representation

Buildings play a significant role in urban flooding in two ways: 1) high imperviousness and runoff generation rate and 2) impact on flood propagation. The first aspect is typically treated in the hydrologic part of a flood model; in this case, it was represented through the losses parameterisation in direct rainfall, as detailed in Section 4.3.2.

There are several approaches to representing building effects on flood propagation in traditional hydraulic flood modelling, including:

- modifying elevation of building footprints to floor level, which requires floor level survey
- raising elevation of building footprints to prevent flow through
- deactivating building footprints from the model to prevent flow through
- implementing high roughness values to slow down flow velocity.

For a DRM hydraulic model, it is not optimal to directly apply the above methods, due to the fact that rainfall is applied everywhere, including the building footprints. In this case, the following procedures were implemented to represent building effects on flood propagation:

- Low roughness was applied to urban zones in non-inundated areas to represent rainfall falling on roofs (Section 4.3.4).
- Depth-varying roughness was applied to urban zones in inundated areas to represent building effects on flood propagation Section 4.3.4).
- For Channel Court and Kingston Plaza, the following modifications were applied to ensure a detailed representation of building effects:
    - Building footprints were digitized and deactivated from the model
    - Rainfall on deactivated buildings was distributed on a 2 m buffer area along the edge of these buildings by adding SA Rainfall layer
    - The internal ground floor of Channel Court Main Mall was kept active in the model to allow flood propagation through the Main Mall; however, rainfall was distributed along the edge of the Main Mall rather than directly on the ground floor
    - The effects of shops and Woolworths on the Main Mall ground floor were represented by adding internal walls.

The building representation for the Channel Court and Kingston Plaza area is illustrated in Figure 11. It should be mentioned that the methodology implemented at Channel Court and Kingston Plaza can be applied through the whole catchment for future investigation, which was outside the scope of this study.

5. MODEL CALIBRATION

5.1. Objectives

The objective of the calibration process is to build a robust hydrologic and hydraulic modelling system that can replicate historical flood behaviour in the catchment being investigated. If the modelling system can replicate historical flood behaviour, then it can more confidently be used to estimate design flood behaviour.

Typically, in urban areas calibration/validation information is lacking. For this study, the model was calibrated to a recent extreme event, the May 2018 event, as described in Section 2.2.

5.2. Approach

As there was no streamflow data available within the catchment for May 2018 event, photos with flood marks and CCTV were used as the main information to validate the model. Due to the lack of information, the model was parameterised with reasonable details, e.g., integrating detailed design and survey information (Section 3.3.2), developing a detailed loss quantification and schematisation methodology for DRM hydraulic model (Section 4.3.2), and incorporating depth-varying Manning's '*n*', before it was evaluated using data from the historical event. The calibration was conducted essentially in a joint "validation" manner for hydrologic and hydraulic models.

There are four "registered" flood water levels based on flood marks for May 2018 event at U3/30 Lester Crescent, 34 Lester Crescent, 16 Freeman Street, and 15 Channel Highway, as shown in Photo 3 to Photo 6. These water levels were used as the primary source for model validation. Additional water marks (not measured) and CCTV in Channel Court Shopping Centre were also used as approximate references for model evaluation, as shown in Photo 5 to Photo 16.

Photo 3: High Water Mark (approx. **1.5 m** above ground) at U3/30 Lester Crescent, Kingston

Photo 4: Debris Mark Measured at **1.17 m** above Ground (**RL 27.5 m** from DA Plans 2010) opposite 16 Freeman Street, Kingston

Photo 5: High Water Mark (approx. **1.2 m** above ground) at 34 Lester Crescent, Kingston

Photo 6: High Water Mark in Emergency Exit Door **0.25 m** above Floor Level (Ground Floor Level = RL **15.0m** in 1996 Original Drawings) at 15 Channel Highway, Kingston (Kingborough Civic Centre)


Photo 7: Flood Marks in Channel Court (The Main Mall Ground Floor)


Photo 8: Flood Marks in Channel Court (Priceline at the Main Mall Ground Floor)


Photo 9: CCTV at 11-05-2018 00:08:58 in Channel Court Main Mall (Mr Johnson)


Photo 10: CCTV at 11-05-2018 00:31:14 in Channel Court Main Mall (Mr Johnson)


Photo 11: CCTV at 11-05-2018 00:09:04 in Channel Court Main Mall (Millers to Habitat)


Photo 12: CCTV at 11-05-2018 00:24:28 in Channel Court Main Mall (Millers to Habitat)

Photo 13: CCTV at 11-05-2018 00:00:24 in Channel Court Main Mall (Woolworths)

Photo 14: CCTV at 11-05-2018 00:27:16 in Channel Court Main Mall (Woolworths)

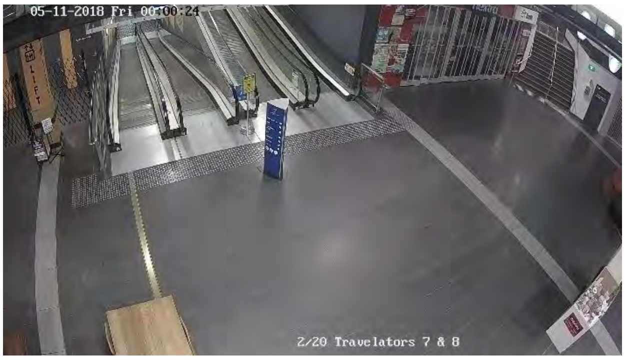


Photo 15: CCTV at 11-05-2018 00:14:43 in Channel Court Main Mall (Travelators)

Photo 16: CCTV at 11-05-2018 00:31:13 in Channel Court Main Mall (Travelators)

5.3. Calibration results

The May 2018 event was simulated using hydrologic and hydraulic modelling detailed in Section 4. Peak flood depth map is shown in Figure 13. A comparison of modelled and the four recorded flood levels is presented in Table 12. The modelled water levels are 0.06 m – 0.16 m higher than recorded levels, which are within tolerance levels and indicate a reasonable fit between model estimations and observations.

Modelled levels on the ground floor inside the Main Mall of Channel Court were also presented in Table 12. The floor level is 24.94 m AHD based on survey. The modelled peak flood depth in front of Woolworths and in the middle of the ground floor are 0.42 m and 0.53 m, respectively, which are considered to be reasonable based on the flood marks (Photo 7 and Photo 8) and CCTV records (Photo 9 to Photo 16). The locations of the validation points listed in Table 12 are illustrated in Figure 13.

The parameters detailed in Section 4 remained unchanged after calibration/validation and adopted for design event modelling.

Table 12: Comparison of Modelled and Recorded Flood Levels

Location	Recorded Level (m AHD)	Modelled Level (m AHD)	Difference (Modelled - Recorded) (m)
Unit 3/ 30 Lester Cres	15.81	15.88	0.06
34 Lester Cres	15.79	15.89	0.09
16 Freeman St	28.70	28.86	0.16
15 Channel Hwy	15.26	15.32	0.06
Channel Court – Woolworths	25.34	25.36	0.02
Channel Court – Ground Floor in Mall	25.41	25.47	0.06

NOTE: Recorded levels at 16 Freeman St and 15 Channel Hwy are based on DEM-based floor levels. Recorded levels at Unit 3/ 30 Lester Cr and 34 Lester Cr are based on registered floor levels provided with Photo 4 and Photo 6. Recorded levels in Channel Court were obtained through comparison of CCTV footage and laser scanned levels.

6. DESIGN EVENT MODELLING

6.1. Temporal Patterns and Durations Selection

Temporal patterns are a hydrologic tool that describe how rainfall falls over time and are often used in hydrograph estimation. Advice in ARR 1987, was to adopt a single burst temporal pattern for each rainfall event duration. However, ARR 2019 (Reference 2) discusses the potential inaccuracies with adopting a single temporal pattern and recommends an approach where an ensemble of temporal patterns is investigated.

An ensemble of 10 temporal patterns is applicable across AEP ranges for durations ranging from 10 mins to 7 days. However, assessment of all durations and patterns in the hydraulic model is inefficient in terms of run time and data storage, especially when multiple iterations and scenarios were conducted for this study and there will be possible future applications by Council. Therefore, critical temporal patterns and durations were selected based on an initial modelling experiment.

For each duration, the temporal pattern producing the flood level closest to the mean of all patterns at a given location is typically deemed as being the representative pattern at that location. The duration producing the highest flood level at the given location is deemed as being the critical duration at that location.

To determine the critical storm duration(s) and select representative temporal pattern(s) for various parts of the catchment, modelling of the 1% AEP event was undertaken for a range of design storm durations from 15 minutes to 6 hours, with the consideration of the catchment size and previous experience.

Each duration utilised ten temporal patterns from ARR 2019 (Reference 2). The following process was undertaken in order to determine the critical duration(s) and representative pattern(s):

1. Run hydrologic and hydraulic models of 10 temporal patterns for each duration for the 1% AEP event. The hydraulic model was run with a 3 m × 3m grid size for this purpose to reduce the run time.
2. Determine the mean peak level from the 10 model runs at all hydraulic grids within the catchment from each duration modelled.
3. Select the critical duration(s), producing the maximum mean peak level for representative areas, Whitewater Creek area and Kingston CBD area in this case.
4. For each critical duration, select a single temporal pattern that produces the closest peak level to the mean peak level in representative areas.
5. Examine the representativeness of selected critical durations and temporal patterns outside the representative areas, to ensure that they produce peak flows close enough to the maximum mean peak level, e.g., within ± 0.1m, over most of the catchment.

It was found that the 4.5-hour duration was critical for Whitewater Creek and 30 min duration was critical for Kingston CBD and some other urbanised areas for the 1% AEP event. Temporal patterns #6614 and #6777 (ARR 2019 pattern numbers) were selected for 4.5-hr and 30 min durations, respectively. The patterns and durations selected for 1% AEP were extended to 5% and 0.5% AEP events modelling.

6.2. Design Results

The results from this study are presented in Appendix D as:

- Peak flood extent and levels in Figure C1 to Figure C3
- Peak flood velocities in Figure C4 to Figure C6
- Peak flood depths in Figure C7 to Figure C9
- Hydraulic hazard in Figure C10 to Figure C12
- Hydraulic categories in Figure C13 to Figure C15

The results were provided in digital format compatible with Council's Geographic Information Systems. The digital data should be used in preference to the figures in this report as they provide more detail. Please note that all flood maps and tables below are presented for flood depth ≥ 0.05 m. Areas with flood depth ≤ 0.05 m are treated as non-inundated area.

6.2.1. Summary of Results

A number of reporting locations were digitalized to obtain flood levels, depths, and flows from TUFLOW, which were provided in csv files together with the model and other result files. Table 13, Table 14, and Table 15 summarise results at key locations (as shown in Figure 14), a subset of the original reporting locations. These key locations coincide with the key locations used for the sensitivity analysis discussed in Section 7.

Table 13: Peak Flood Levels (m AHD) at Key Locations (depth ≥ 0.05 m)

ID	Location	5% AEP	1% AEP	0.5% AEP
Point01	16 Freeman St (Loading Bay)	-	27.65	28.00
Point02	Channel Court - Entrance from Freeman St	26.99	27.62	27.96
Point04	Channel Court - Woolworths	-	-	25.05
Point06	Channel Court - Ground Floor	-	25.01	25.16
Point11	Channel Court - Entrance from Central	-	24.71	24.81
Point22	15 Channel Hwy (West)	15.27	15.28	15.29
Point24	15 Channel Hwy (South)	15.26	15.29	15.30
Point26	Kingston Plaza Central Car Park	-	11.23	11.23
Point31	Kingston Plaza South Car Park	11.61	11.73	11.78
Point36	3/14 Channel Hwy	6.16	6.29	6.31
Point43	WWC U/S Southern Outlet	11.82	13.74	14.47
Point44	WWC U/S Huon Hwy	6.96	7.47	7.54
Point45	Unit 3/ 30 Lester Cres	-	-	14.50
Point47	34 Lester Cres	-	-	14.63
Point48	46 Whitewater Cres	-	17.28	17.36
Point50	48 Whitewater Cres	-	17.91	18.00
Point51	50 Whitewater Cres	-	18.07	18.16
Point54	Unit 1/ 59 Whitewater Cres	-	21.19	21.26
Point56	Unit3/ 31 Whitewater Cres	-	26.16	26.26
Point57	29 Whitewater Cres	-	26.63	26.70
Point62	16 Freeman St	27.94	28.21	28.30
Point64	Unit 6/ 20-22 Freeman St	31.21	31.39	31.47
Point66	12 Sherburd St	36.06	36.14	36.18

Table 14: Peak Depths (m) at Key Locations (depth ≥ 0.05 m)

ID	Location	5% AEP	1% AEP	0.5% AEP
Point01	16 Freeman St (Loading Bay)	-	0.39	0.73
Point02	Channel Court - Entrance from Freeman St	0.17	0.81	1.15
Point04	Channel Court - Woolworths	-	-	0.10
Point06	Channel Court - Ground Floor	-	0.08	0.22
Point11	Channel Court - Entrance from Central	-	0.07	0.18
Point22	15 Channel Hwy (West)	0.24	0.26	0.26
Point24	15 Channel Hwy (South)	0.35	0.38	0.39
Point26	Kingston Plaza Central Car Park	-	0.08	0.08
Point31	Kingston Plaza South Car Park	0.19	0.31	0.36
Point36	3/14 Channel Hwy	0.11	0.23	0.25
Point43	WWC U/S Southern Outlet	3.76	5.68	6.42
Point44	WWC U/S Huon Hwy	3.96	4.47	4.54
Point45	Unit 3/ 30 Lester Cres	-	-	0.19
Point47	34 Lester Cres	-	-	0.05
Point48	46 Whitewater Cres	-	0.23	0.31
Point50	48 Whitewater Cres	-	0.22	0.31
Point51	50 Whitewater Cres	-	0.16	0.25
Point54	Unit 1/ 59 Whitewater Cres	-	0.10	0.17
Point56	Unit3/ 31 Whitewater Cres	-	0.11	0.21
Point57	29 Whitewater Cres	-	0.08	0.16
Point62	16 Freeman St	0.39	0.65	0.75
Point64	Unit 6/ 20-22 Freeman St	0.43	0.60	0.68
Point66	12 Sherburd St	0.17	0.25	0.28

Table 15: Peak Flows (m^3/s) at Key Cross-sections

ID	Location	5% AEP	1% AEP	0.5% AEP
XS01	Smmerleas Rd U/S	18.32	33.33	39.61
XS02	Smmerleas Rd D/S	18.45	33.44	39.87
XS05	Southern Outlet U/S	26.93	43.20	48.89
XS07	Channel Court U/S	3.51	6.40	7.97
XS08	Spring Farm West	13.72	23.39	27.56
XS09	Spring Farm Rd	14.87	25.68	30.22
XS10	Spring Farm East	17.12	29.83	35.34
XS14	Southern Outlet D/S	26.25	42.84	48.75
XS16	Harris Ct D/S	1.88	3.84	4.81
XS18	Sherburd St U/S	3.82	6.45	8.23
XS19	Channel Court D/S	1.01	1.51	1.77
XS21	Channel Court Entrance from Freeman St	0.00	0.62	2.39
XS25	Twin Ovals	0.00	2.91	3.44

6.2.2. Hydraulic Hazard

Hazard classification plays an important role in informing floodplain risk management in an area. Provisional hazard categories have been determined for the Whitewater Creek catchment in accordance with the Australian Disaster Resilience Handbook Collection (Reference 11).

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. The Australian Disaster Resilience Handbook Collection deals with floods in Handbook 7 (Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia). The supporting guideline 7-3 (Reference 11) contains information relating to the categorisation of flood hazard. A summary of this categorisation is provided in Diagram 9.

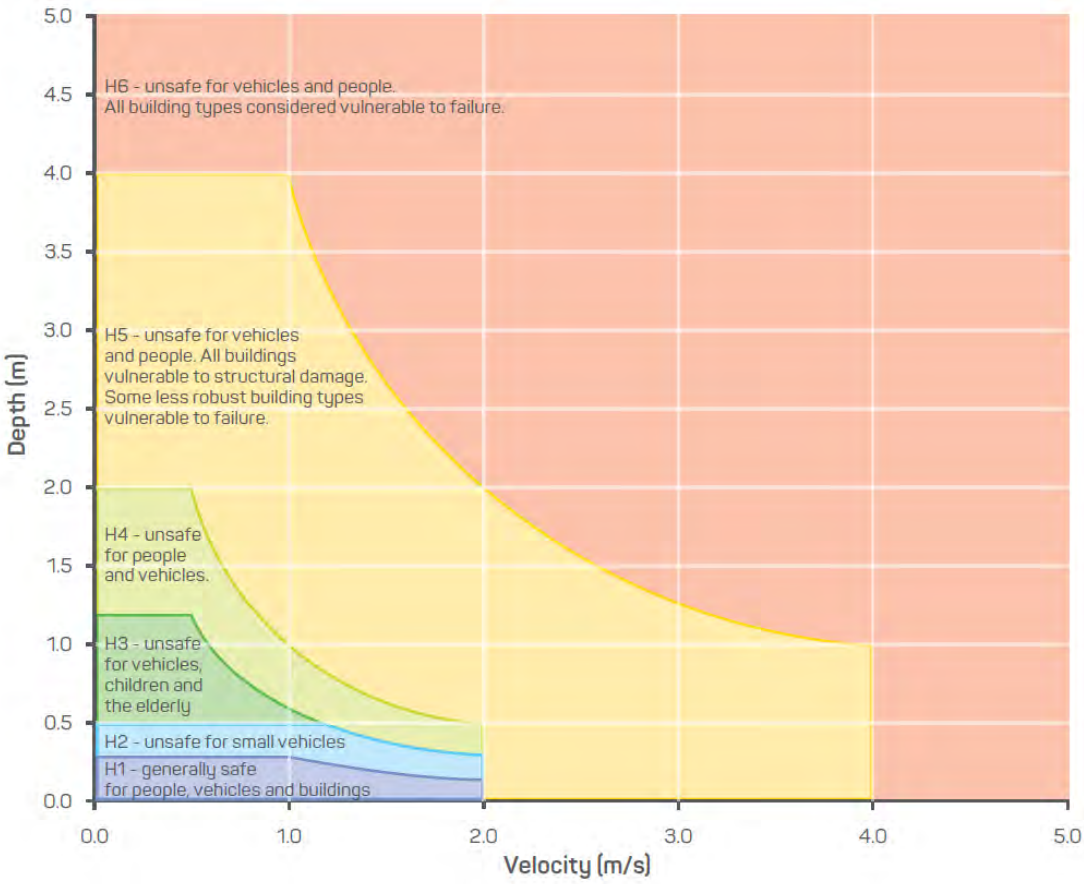


Diagram 9: General Flood Hazard Vulnerability Curves (ADR)

This classification provides a more detailed distinction and practical application of hazard categories, identifying the following 6 classes of hazard:

- H1 – No constraints, generally safe for vehicles, people and buildings;
- H2 – Unsafe for small vehicles;
- H3 – Unsafe for all vehicles, children and the elderly;
- H4 – Unsafe for all people and all vehicles;
- H5 – Unsafe for all people and all vehicles. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure. Buildings require special engineering design and construction; and
- H6 – Unsafe for all people and all vehicles. All building types considered vulnerable to failure.

The hazard maps created using the Australian Disaster Resilience (ADR) classification are presented in Figure C10 to Figure C12 for the 5% AEP, 1% AEP, and 0.5% AEP events.

6.2.3. Hydraulic Categorisation

Floodplains can be classified into the following hydraulic categories depending on the flood function:

- Floodways
- Flood Storage and
- Flood Fringe.

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 are available, such as the method defined by Howells *et al* (Reference 12).

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

- Floodway is defined as areas where:
    - the peak value of velocity multiplied by depth (V x D) > 0.25 m^2/s, **AND** peak velocity > 0.25 m/s, **OR**
    - peak velocity > 1.0 m/s **AND** peak depth > 0.1 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.

Figure C13 to Figure C15 show the provisional hydraulic categorisations for the Whitewater catchment for the 5% AEP, 1% AEP, and 0.5% AEP events.

7. SENSITIVITY ANALYSIS

7.1. Overview

A number of sensitivity analyses were undertaken for 1% AEP events to establish the variation in design flood levels and flow that may occur if different parameter assumptions were made. These sensitivity scenarios are summarised in Table 16.

Table 16: Overview of Sensitivity Analyses

Scenario	Description
Climate Change	Sensitivity to climate change assessed for: • Year 2050 scenario with 10% IFD increase; • Year 2100 scenario with 30% IFD increase.
Manning's '*n*'	Sensitivity to hydraulic roughness assessed for: • Manning's '*n*' decreased by 20%; • Manning's '*n*' increased by 20%.
Blockage	Sensitivity to blockage assessed for: • 50% blockage of bridge and culverts; • 50% blockage of all inlet pits; • 50% blockage of bridge and culverts, and all inlet pits.

7.2. Climate Change

The increase in rainfall intensities due to climate change adopted for this study were those used in Kingston Beach Flood Study (Reference 1). Rainfall intensities were assumed to increase 10% and 30% by Year 2050 and Year 2100, respectively. Flood depth mapping is presented in Figure D1 and Figure D2. The climate change sensitivity results at key locations are shown in Table 17.

The increase in rainfall results in an increase in peak flood levels at the majority of the locations analysed. The largest variation in flood level occurred at Whitewater Creek upstream of the Southern Outlet with modelled peak flood levels up to 1.55 m higher under the Year 2100 climate scenario.

Modelling indicates that increases in rainfall intensity would have a significant impact on mainstream flood levels and a less substantial impact on peak flood levels in steep overland flow areas. Peak flood level increases at Whitewater Creek upstream of Southern Outlet are particularly notable, with peak flood level increases of 0.56 m and 1.55 m, for the Year 2050 and Year 2100 climate scenarios. This is because the culvert across Southern Outlet forms a significant hydraulic constraint and can cause significant volume of water to be detained upstream of Huon Highway. This could impact on the properties along Whitewater Creek immediately upstream of Huon Highway.

In Kingston Rivulet sub-catchment, peak flood level increases on Freeman Street upstream of Channel Court are most notable (Points 1 and 2), with peak flood level increases of 0.26 m and 0.67 m, for the Year 2050 and Year 2100 climate scenarios respectively. This is because the drainage system under Channel Court restricts the flow in Kingston Rivulet, which resulted in significant flooding in May 2018 event. This is likely to cause more serious issues if a similar probability event happens under future climate scenarios.

Table 17: Results of Climate Change Analysis for 1% AEP (depth ≥ 0.05 m)

ID	Location	Peak Flood Level (m AHD)			Difference (m)	
		Existing	Y2050	Y2100	Y2050 - Existing	Y2100 - Existing
Point01	16 Freeman St (Loading Bay)	27.65	27.91	28.32	0.26	0.67
Point02	Channel Court - Entrance Freeman St	27.62	27.88	28.29	0.26	0.67
Point04	Channel Court - Woolworths	-	25.03	25.15	-	-
Point06	Channel Court - Ground Floor	25.01	25.12	25.28	0.11	0.27
Point11	Channel Court - Entrance from Central	24.71	24.79	24.89	0.08	0.19
Point22	15 Channel Hwy (West)	15.28	15.29	15.30	0.00	0.02
Point24	15 Channel Hwy (South)	15.29	15.30	15.31	0.01	0.02
Point26	Kingston Plaza Central Car Park	11.23	11.23	11.24	0.00	0.01
Point31	Kingston Plaza South Car Park	11.73	11.77	11.92	0.04	0.18
Point36	3/14 Channel Hwy	6.29	6.30	6.33	0.01	0.04
Point43	WWC U/S Southern Outlet	13.74	14.29	15.28	0.56	1.55
Point44	WWC U/S Huon Hwy	7.47	7.52	7.60	0.05	0.13
Point45	Unit 3/ 30 Lester Cres	-	-	15.30	-	-
Point47	34 Lester Cres	-	-	15.31	-	-
Point48	46 Whitewater Cres	17.28	17.34	17.47	0.06	0.19
Point50	48 Whitewater Cres	17.91	17.98	18.12	0.07	0.21
Point51	50 Whitewater Cres	18.07	18.14	18.28	0.07	0.21
Point54	Unit 1/ 59 Whitewater Cres	21.19	21.22	21.72	0.03	0.53
Point56	Unit3/ 31 Whitewater Cres	26.16	26.24	26.39	0.07	0.23
Point57	29 Whitewater Cres	26.63	26.68	26.79	0.06	0.16
Point62	16 Freeman St	28.21	28.28	28.43	0.07	0.22
Point64	Unit 6/ 20-22 Freeman St	31.39	31.45	31.56	0.06	0.17
Point66	12 Sherburd St	36.14	36.17	36.24	0.03	0.09

7.3. Roughness Variations

Flood depth mapping for roughness variations is presented in Figure D3 and Figure D4. The results at key locations are shown in Table 18.

Overall peak flood level results were shown to be relatively insensitive to 20% variations in the roughness parameter. Varying the roughness parameter by 20% typically resulted in a peak flood height difference within ± 0.1 m. A slightly greater change in peak flood levels occurred on Freeman Street upstream of Channel Court (Points 1 and 2). This is likely because the entire contributing area upstream of Channel Court is highly urbanised, and the catchment response is fast. In this area, the change in the roughness results in a relatively larger change in response time, hence the flow detention on Freeman Street in front of Channel Court.

Table 18: Results of Roughness Sensitivity Analysis for 1% AEP (depth ≥ 0.05 m)

ID	Location	Peak Flood Level (m AHD)			Difference (m)	
		Existing	- 20%	+ 20%	- 20%	+ 20%
Point01	16 Freeman St (Loading Bay)	27.65	27.77	27.53	0.11	-0.13
Point02	Channel Court - Entrance Freeman St	27.62	27.73	27.49	0.12	-0.13
Point04	Channel Court - Woolworths	-	-	-	-	-
Point06	Channel Court - Ground Floor	25.01	25.06	-	0.05	-
Point11	Channel Court - Entrance from Central	24.71	24.74	-	0.03	-
Point22	15 Channel Hwy (West)	15.28	15.28	15.29	-0.01	0.00
Point24	15 Channel Hwy (South)	15.29	15.28	15.29	0.00	0.00
Point26	Kingston Plaza Central Car Park	11.23	11.23	11.23	0.00	0.00
Point31	Kingston Plaza South Car Park	11.73	11.72	11.73	-0.01	0.00
Point36	3/14 Channel Hwy	6.29	6.28	6.30	-0.01	0.01
Point43	WWC U/S Southern Outlet	13.74	13.84	13.66	0.10	-0.08
Point44	WWC U/S Huon Hwy	7.47	7.48	7.48	0.00	0.00
Point45	Unit 3/ 30 Lester Cres	-	-	-	-	-
Point47	34 Lester Cres	-	-	-	-	-
Point48	46 Whitewater Cres	17.28	17.22	17.31	-0.05	0.04
Point50	48 Whitewater Cres	17.91	17.88	17.94	-0.03	0.03
Point51	50 Whitewater Cres	18.07	17.95	18.16	-0.12	0.09
Point54	Unit 1/ 59 Whitewater Cres	21.19	21.08	21.22	-0.11	0.03
Point56	Unit3/ 31 Whitewater Cres	26.16	26.12	26.21	-0.05	0.05
Point57	29 Whitewater Cres	26.63	26.62	26.63	-0.01	0.01
Point62	16 Freeman St	28.21	28.20	28.21	-0.01	0.00
Point64	Unit 6/ 20-22 Freeman St	31.39	31.36	31.41	-0.03	0.02
Point66	12 Sherburd St	36.14	36.12	36.16	-0.02	0.02

7.4. Blockage Variations

There are multiple factors to be considered in assessing the potential for blockage of drainage systems, e.g. the type and size of the structure, the type of mobility of debris causing blockage, and catchment land use. In this study, 50% blockage was applied to bridges and culverts for open channels, inlet pits for underground drainage, or both. Flood depth mapping for blockage variations is presented in Figure D5 to Figure D7. The results at key locations are shown in Table 19.

Peak flood levels at most locations were found to be relatively insensitive to blockage, with a few notable exceptions. Whitewater Creek upstream of Southern Outlet is very sensitive to the blockage of the culvert under the Southern Outlet, showing peak level differences up to 2.23 m compared to no-blockage conditions. This is consistent with the sensitivity to climate change scenarios shown at this site and highlighting this site as requiring particular attention for flood management and mitigation design.

Table 19: Results of Blockage Sensitivity Sensitivity Analysis for 1% AEP (depth ≥ 0.05 m)

ID	Location	Peak Flood Level (m AHD)				Difference (m)		
		Existing	50% Block Bridge, Culverts	50% Block Pits	50% block Bridge, Culverts Pits	50% Block Bridge, Culverts	50% Block Pits	50% block Bridge, Culverts Pits
Point01	16 Freeman St (Loading Bay)	27.65	27.65	27.70	27.70	0.00	0.04	0.04
Point02	Channel Court - Freeman St	27.62	27.62	27.67	27.67	0.00	0.05	0.05
Point04	Channel Court - Woolworths	-	-	-	-	-	-	-
Point06	Channel Court - Ground Floor	25.01	25.01	25.04	25.03	0.00	0.02	0.02
Point11	Channel Court - from Central	24.71	24.70	24.72	24.72	0.00	0.02	0.02
Point22	15 Channel Hwy (West)	15.28	15.28	15.28	15.28	0.00	0.00	0.00
Point24	15 Channel Hwy (South)	15.29	15.29	15.29	15.29	0.00	0.00	0.00
Point26	Kingston Plaza Centre Parking	11.23	11.23	11.25	11.25	0.00	0.02	0.02
Point31	Kingston Plaza South Parking	11.73	11.73	11.73	11.73	0.00	0.00	0.00
Point36	3/14 Channel Hwy	6.29	6.29	6.28	6.28	0.00	-0.01	-0.01
Point43	WWC U/S Southern Outlet	13.74	15.94	13.74	15.97	2.20	0.01	2.23
Point44	WWC U/S Huon Hwy	7.47	7.54	7.48	7.54	0.06	0.00	0.06
Point45	Unit 3/ 30 Lester Cres	-	15.94	-	15.97	-	-	-
Point47	34 Lester Cres	-	15.95	-	15.98	-	-	-
Point48	46 Whitewater Cres	17.28	17.27	17.27	17.27	0.00	0.00	-0.01
Point50	48 Whitewater Cres	17.91	17.91	17.91	17.91	0.00	0.00	0.00
Point51	50 Whitewater Cres	18.07	18.07	18.07	18.06	0.00	0.00	0.00
Point54	Unit 1 / 59 Whitewater Cres	21.19	21.19	21.19	21.19	0.00	0.00	0.00
Point56	Unit3 / 31 Whitewater Cres	26.16	26.16	26.16	26.16	-0.01	0.00	-0.01
Point57	29 Whitewater Cres	26.63	26.62	26.63	26.62	0.00	0.00	0.00
Point62	16 Freeman St	28.21	28.21	28.23	28.24	0.00	0.03	0.03
Point64	Unit 6/ 20-22 Freeman St	31.39	31.39	31.45	31.45	0.00	0.06	0.06
Point66	12 Sherburd St	36.14	36.14	36.15	36.15	0.00	0.00	0.00

8. FLOODING HOTSPOTS

Some of the key areas where flooding is problematic, sometimes referred to as "hotspots," are discussed below. Figure E1 provides an overview of the locations discussed.

8.1. Summerleas Road Culvert

Summerleas Road crossing of Whitewater Creek is a significant hydraulic constraint during flood events. There are two 2×2 m (based on visual inspection) box culverts underneath Summerleas Road. The road was overtopped by approximately 1 m causing restricted access problems to the primary access for a population of approximately 4,500 residents during May 2018 flood event, as shown in Photo 17.

Design flood levels at the sag point upstream of the culverts and on the road surface, flows within the twin culverts underneath the road and overland flows over the road embankment, as well as inflows and outflows are summarised in Table 20. Figure E2 shows design flood behaviour for the 1% AEP event and the key reporting locations for flood levels and flows.

Photo 17: Summerleas Road crossing of Whitewater Creek after 2018 Event

Table 20: Design Flood Behaviour near Summerleas Road Culvert

Event	Peak Level (m AHD)	Peak Depth (m)	Peak Level (m AHD)	Peak Depth (m)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Flow (across road) (m^3/s)	
	Sag Point		Overtop		Creek	Creek	Culvert	Overland
5% AEP	28.76	2.53	28.71	0.11	18.30	18.36	17.22	1.15
1% AEP	29.22	2.99	28.90	0.30	33.36	33.45	20.01	13.42
0.5% AEP	29.33	3.10	28.95	0.35	39.67	39.78	20.44	19.32

8.2. 29-31 Whitewater Crescent

Properties at 29-31 Whitewater Crescent are located along Whitewater Creek, between Summerleas Road Culvert and a confluence where Drysdale Creek joins Whitewater Creek (Figure E1). Several calls were received by Council during and after the May 2018 event around this region according to the map provided by Council. Photo 18 shows the aerial and street view of this spot.

Photo 18: 29-31 Whitewater Crescent. Nearmap Aerial (upper) and Google Street (lower) Views

Table 21: Design Flood Behaviour near 29-31 Whitewater Crescent

Event	Peak Level (m AHD)	Peak Depth (m)	Peak Level (m AHD)	Peak Depth (m)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)
	29 Whitewater Cres		U3 / 31 Whitewater Cres		Creek	Creek
5% AEP	Not flooded		Not flooded		18.40	18.43
1% AEP	26.77	0.24	26.16	0.11	33.48	33.41
0.5% AEP	26.86	0.33	26.26	0.21	39.85	39.82

Design flood levels in front of affected houses, as well as inflows and outflows of this section of Whitewater Creek are summarised in Table 21. Figure E3 shows design flood behaviour for the 1% AEP event and the key reporting locations for flood levels and flows.

As indicated by Table 21 and Figure E3, it is identified that water overtops the north bank of Whitewater Creek causing inundation of 29 Whitewater Creek and Unit 3 / 31 Whitewater Creek under 1% and 0.5% AEP events. There are fences around the properties, e.g., the south and west side of 29 Whitewater Cres, however, those fences are woody fences as inferred from Photo 18, which is a general case for properties along Whitewater Creek, e.g., Photo 19 to Photo 21. Those woody fence are not deemed capable to protect properties from flood inundation. Flood mitigation measures are discussed in Section 9.

8.3. Whitewater Crescent Culvert

Whitewater Crescent crossing of Whitewater Creek, with an arch culvert underneath, act as another constraint during flood events, and properties immediately upstream of the culvert are subject to inundation. Photo 19 shows the Culvert and upstream properties, i.e., 59 Whitewater Crescent, after May 2018 event.


Photo 19: Whitewater Crescent Culvert (left) and 59 Whitewater Crescent (right) after 2018 Event

Table 22: Design Flood Behaviour near Whitewater Crescent Culvert

Event	Peak Level (m AHD)	Peak Depth (m)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Flow (across road) (m^3/s)	
	59 Whitewater Cres		Creek	Creek	Culvert	Overland
5% AEP	20.63	0.43	24.34	24.50	24.51	0.0051
1% AEP	21.03	0.83	43.49	43.79	43.87	0.0053
0.5% AEP	21.35	1.15	51.48	51.76	50.88	0.0054

Design flood levels along the fence in front of 59 Whitewater Crescent, flows within the culvert underneath the road and overland flows over the road embankment, as well as inflows and outflows are summarised in Table 22. Figure E4 shows design flood behaviour for the 1% AEP event and the key reporting locations for flood levels and flows.

As shown in Figure E4 and Table 22, there is almost no overland flow across the road embankment and no significant water detention upstream of the culvert. The flood depth is over 1 m in 0.5% AEP event along the fence. However, the houses are located at higher elevation, and are not subject to significant flooding. Photo 19 illustrates that fences were affected after the May 2018 event.

8.4. 46-50 Whitewater Crescent

Properties at 46-50 Whitewater Crescent are located along Whitewater Creek, downstream of Whitewater Crescent Culvert (Figure E1). Calls were received by Council during and after the May 2018 event around this region according to the map provided by Council. Photo 20 shows the fence destroyed after May 2018 event.

Design flood levels in front of houses, as well as inflows and outflows of this section of Whitewater Creek are summarised in Table 23. Figure E5 shows design flood behaviour for the 1% AEP event and the key reporting locations for flood levels and flows.

It is shown that the main buildings (houses) at 46, 48, and 50 Whitewater Crescent are subject to different levels of inundation during 1% and 0.5% AEP events. Flood mitigation measures are explored and detailed in Section 9.

Table 23: Design Flood Behaviour near 46-50 Whitewater Crescent

Event (AEP)	Peak Level (m AHD)	Peak Depth (m)	Peak Level (m AHD)	Peak Depth (m)	Peak Level (m AHD)	Peak Depth (m)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)
	46 Whitewater Cres		48 Whitewater Cres		50 Whitewater Cres		Creek	Creek
5%	Not flooded		Not flooded		Not flooded		24.49	24.55
1%	17.28	0.23	17.91	0.22	18.07	0.16	43.88	44.03
0.5%	17.36	0.31	18.00	0.31	18.16	0.25	51.91	52.07

Photo 20: Fence Damage along 46-50 Whitewater Crescent after 2018 Event

8.5. 30-34 Lester Crescent

30-34 Lester Crescent are located upstream of Southern Outlet Culvert, which is a significant constraint during flood events. As noted in Section 5, houses at 34 Lester Crescent and Unit 3 / 30 Lester Crescent were subject to 1.2 m and 1.5 m of inundation during May 2018 event. Photo 21 illustrates 34 Lester Crescent and Unit 3 / 30 Lester Crescent after May 2018 event.

Design flood levels in front of affected houses, as well as inflows and outflows of this section of Whitewater Creek are summarised in Table 24. Figure E6 shows design flood behaviour for the 1% AEP event and the key reporting locations for flood levels and flows.

Based on the modelling results, houses at 34 Lester Crescent and Unit 3 / 30 Lester Crescent are free of inundation during 5% and 1% AEP events, and are subject to 0.26 m to 0.27 m of inundation during 0.5% AEP. The May 2018 event was related to a rarer flood event. Based on the 4.5hr (10/05/2018 10:30pm – 11/05/2018 03:00pm) cumulation of the reconstructed pluviograph at Kingston gauge, the May 2018 event is equivalent to a 0.1% (1 in 1000) AEP event.

Flood depths and heights in front of houses at 34 Lester Crescent and Unit 3 / 30 Lester Crescent under climate change conditions are summarised in Table 25. Although those houses are only subject to very rare to extreme events, i.e., 0.5% AEP or rarer, under current conditions, more frequent significant inundation is likely under future climate change conditions due to the possible increase of rainfall intensities. Flood mitigation measures are explored for this location in Section 9.

Photo 21: 34 Lester Crescent (left) and Unit 3 / 30 Lester Crescent after 2018 Event

Table 24: Design flood behaviour near 30-34 Lester Crescent

Event	Peak Level (m AHD)	Peak Depth (m)	Peak Level (m AHD)	Peak Depth (m)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)
	34 Lester Cres		U3 / 30 Lester Cres		Creek	Creek
5% AEP	Not flooded		Not flooded		24.72	25.20
1% AEP	Not flooded		Not flooded		44.03	43.77
0.5% AEP	14.53	0.26	14.50	0.27	51.11	49.87

Table 25: Flood Depth and Height near 30-34 Lester Crescent under Climate Change Conditions

Event		Peak Level (m AHD)	Peak Depth (m)	Peak Level (m AHD)	Peak Depth (m)
		34 Lester Cres		U3 / 30 Lester Cres	
2050	5% AEP	Not flooded		Not flooded	
	1% AEP	14.38	0.11	14.33	0.11
	0.5% AEP	15.05	0.78	15.04	0.82
2100	5% AEP	0.00	0.00	0.00	0.00
	1% AEP	15.31	1.04	15.30	1.08
	0.5% AEP	16.10	1.83	16.10	1.87

8.6. Southern Outlet Culvert

Southern Outlet crossing of Whitewater Creek, with a box culvert underneath, acts as a significant constraint during rare to extreme flood events. Photo 22 shows the culvert underneath the Southern Outlet.

Design flood levels at the sag point upstream of the culvert, as well as inflows and outflows are summarised in Table 26. Figure E7 shows design flood behaviour for the 1% AEP event and the key reporting locations for flood levels and flows.

It is notable that the road crossing and the culvert underneath cause significant detention of flood water upstream of the culvert, which results in inundation of upstream properties, e.g., 30-34 Lester Crescent. The maximum depth is 6.92 m at the sag point upstream of the culvert during 0.5% AEP event. Water flows through the culvert without overtopping for 5%, 1%, and 0.5% AEP event. Flood mitigation measures are discussed in Section 9 to reduce the water detention and resultant inundation to upstream properties.

Table 26: Design Flood Behaviour near Southern Outlet Culvert

Event	Peak Level (m AHD)	Peak Depth (m)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Flow (across road) (m^3/s)
	Southern Outlet Sag		Creek	Creek	Culvert
5% AEP	11.81	4.27	26.61	26.82	26.84
1% AEP	13.72	6.18	42.74	42.79	47.80
0.5% AEP	14.46	6.92	48.59	48.98	48.63

Photo 22: Southern Outlet Culvert

8.7. Huon Highway Culvert

Huon Highway crossing of Whitewater Creek, with a box culvert underneath, is another significant constraint during rare to extreme flood events. Photo 23 shows the culvert underneath the Huon Highway.

Design flows within the culvert underneath the road and overland flows over the road embankment, as well as inflows and outflows, are summarised in in Table 27. Figure E8 shows design flood behaviour for the 1% AEP event and the key reporting locations for flows.

It is noted that the flood water flows through both the culvert and overland during 1% and 0.5% AEP events. In a 0.5% AEP event, there is 20.21 m^3/s overland peak flow, which is a considerable portion of the total flow. This constraint causes significant detention between Southern Outlet culvert and Huon Highway culvert. The piece of land subject to inundation currently forms a good reserved detention area. Any proposed civil work in this area, e.g., land fill, needs to be carefully assessed before implementation.

Photo 23: Huon Highway Culvert

Table 27: Design Flood Behaviour near Huon Highway Culvert

Event	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Flow (across road) (m^3/s)	
	Creek	Creek	Culvert	Overland
5% AEP	25.16	26.15	25.44	0.00
1% AEP	43.55	46.42	29.55	14.80
0.5% AEP	49.26	51.86	30.01	20.21

9. FLOOD MITIGATION

Flood mitigation is the consideration of contingencies and preventative measures as well as protective measures that may be put in place to avoid the realisation of flood risk. Flood mitigation measures are methods of reducing the impact of floods. These methods may be structural solutions (e.g. retarding basins, levees) or non-structural (e.g. land-use planning, early warning systems).

9.1. Structural Mitigation Measures

Structural flood management measures are designed to protect people and property by counteracting the flood event in order to reduce the hazard or to influence the course or probability of occurrence of the event.

For this study, structural mitigation measures were considered to manage flooding at the hotspots identified along Whitewater Creek. Containment of flows within the waterway corridor and reducing attenuation at key hydraulic constraints were assessed in this study.

The proximity of the existing state of development to the Whitewater Creek floodplain limits the type of structural mitigation measures available.

Although the opportunity for large regional structures is limited, a regional retarding basin could be considered for the open space downstream of Spring Farm. It would be designed to mitigate stormwater runoff from the developing area upstream and reduce the cumulative impacts downstream.

This option has not been analysed within the scope of this study. Therefore, the following structural mitigation measures were analysed –

- Option A: Construction of flood levees
- Option B: Duplication of the Whitewater Creek culvert @ Southern Outlet

9.1.1. Option A: Flood Levees

Construction of levees along the banks of Whitewater Creek will aim to reduce the risk to property from flood waters by confining the watercourse to its channel and preventing it spilling over into the floodplain.

A floodplain, by definition, is subject to regular natural flooding. Its function is to temporarily store and convey floodwaters that cannot be carried by the watercourse channel. Levees stop this natural flow. This action is required here as existing development has encroached into the Whitewater Creek floodplain.

Full confinement of floods by levees will not always be possible or desirable. During extreme floods, water levels may overtop the levee, increasing the risk of breach and failure.

For example, a levee designed and built to exclude a 1-in-200-year (0.5%) AEP flood may be overtopped by a larger flood. Levee systems provide protection from the more frequent flood events up to and including the design event. It is standard practice to provide a minimum 600 mm freeboard above the design event, and this provides a safety margin during larger events.

An effective levee is appropriately located and is designed and constructed according to accepted standards, with a carefully prepared foundation, core and capping to withstand flooding of a specific magnitude.

The levee could be constructed using earthen embankments (raised footpath), crib walls and or concrete retaining walls as shown, respectively, in Photo 24.

Photo 24: Earthen Embankments (L), Crib Walls (M), and Concrete Retaining Walls (R)

Location and costs generally dictate the type of levee chosen for a particular situation. Where the location is not suited to a conventional earthen bank, as may be the case in and around urban areas, a crib wall or concrete retaining wall may be better suited.

For this study, levee flood protection measures were applied in three (3) locations. The locations of the three levee structures can be seen in Figure F1 and are discussed below.

Levee 1 will provide protection for two properties currently flooded under existing conditions. Levee objective is to confine flood flows to Whitewater Creek alignment by directing over-bank flows through the watercourse and not through adjoining properties.

Levee 2 is positioned downstream of Whitewater Crescent and will provide protection for six (6) properties currently affected by flood flows within Whitewater Creek.

Levee 3 is positioned along the waterway frontage of four (4) properties upstream of the Southern Outlet. The properties in this location experienced the most severe flood impacts during the 2018 flood event, as flood flows in Whitewater Creek become impeded by the Southern Outlet.

The proposed levees were represented by 2d_zsh modifiers (2m lines) in TUFLOW and the modelling results are presented in Sections 9.1.1.1 – 9.1.1.2. In total, it is expected that the modelled levee structures will provide flood protection for twelve (12) properties currently affected by Whitewater Creek flooding in the 0.5% AEP event.

Conceptual design and costing of the proposed levees are presented in Section 9.1.4.

9.1.1.1. Flood Protection Capability

The impacts of flood levees are presented in Change in Flood Height (Figure F2 and Figure F4) and Change in Flood Velocity (Figure F3 and Figure F5) mapping. Preliminary modelling suggests the levee structures can provide an improved level of flood protection for properties adjacent to Whitewater Creek, in flood events up to and including the 0.5% AEP. The proposed levees appear to be capable of providing flood protection to affected properties from the Whitewater Creek flood flows without creating adverse impacts on downstream properties.

9.1.1.2. Discussion

The structural levees can be designed to contain the flood flows within the Whitewater Creek channel. However, the levee structures also prevent the free draining of local runoff flows from the immediate urban catchment area. This results in attenuation and ponding behind the levee structures. A pumped stormwater system may need to be considered to assist drainage at these locations.

For this analysis, a pumped system was introduced to address the ponding of local catchment runoff behind the flood levee. The simulated pump capacities are detailed in Table 28 for the 1% and 0.5% AEP design events.

Table 28: Simulated Levee Pump Capacities

AEP	Levee 1	Levee 2	Levee 3
1%	0.08 m^3/s	0.42 m^3/s	0.39 m^3/s
0.5%	0.07 m^3/s	0.36 m^3/s	0.34 m^3/s

9.1.2. Option B: Southern Outlet Culvert Duplication

The Southern Outlet crossing forms a significant obstruction to flood flows within Whitewater Creek. Whitewater Creek experiences significant afflux (increase in water level) caused by the constriction of flow at the culvert.

Afflux appears to extend upstream more than 250 metres resulting in impacts on residential property. Increasing the flow capacity of the culvert, i.e., duplicating the existing 3.0-metre x 3.0-metre box culvert; can reduce afflux by reducing the constriction effect.

Increasing the flow capacity of the Southern Outlet culvert will reduce the flood storage potential upstream and put more pressure on downstream storages.

9.1.2.1. Flood Protection Capability

The impacts of culvert duplication are presented in Change in Flood Height (Figure F7 and Figure F9) and Change in Flood Velocity (Figure F8 and Figure F10) mapping. Duplication of the Southern Outlet culvert minimises the extent of afflux upstream of the Southern Outlet. The reduction in flood level caused by afflux occurs over a 250-metre length of the Whitewater Creek waterway. Preliminary assessment indicates that four properties benefit from reductions in flood impacts.

9.1.2.2. Discussion

Under the estimated existing 0.5% AEP flood event, only three of the four affected properties have an associated risk to buildings. The culvert duplication works appears to remove the risk to buildings within two of the three properties, and greatly reduce the risk to the third property, as shown in Figure F9.

Construction costs associated with duplicating the Southern Outlet culvert will be quite high and are unlikely to provide a good cost benefit solution. Preliminary analysis suggests that culvert duplication does not present enough flood protection capability to be considered a feasible structural flood mitigation measure.

9.1.3. Summary

The effectiveness of the levee flood mitigation measures (Option A) and the Southern Outlet culvert duplication (Option B) on the mitigation of flood impacts can be seen in the change in flood characteristics detailed in Table 29, as well as flood impact mapping (Figure F2 to Figure F5, Figure F7 to Figure F10).

Table 29: Change in Flood Height (m) at Key Locations

Hotspot	Location	Option A		Option B	
		1% AEP	0.5% AEP	1% AEP	0.5% AEP
Hotspot 2	Unit3 / 31 Whitewater Cres	-0.05	-0.15	0.00	0.00
	29 Whitewater Cres	No longer flooded	No longer flooded	0.00	0.00
Hotspot 4	46 Whitewater Cres	No longer flooded	No longer flooded	0.00	0.00
	48 Whitewater Cres	-0.17	-0.23	0.00	0.00
	50 Whitewater Cres	No longer flooded	-0.23	0.00	0.00
Hotspot 5	34 Lester Cres	No longer flooded	No longer flooded	No longer flooded	No longer flooded
	Unit3 / 30 Lester Cres	No longer flooded	No longer flooded	No longer flooded	No longer flooded
Hotspot 6	Southern Outlet U/S Sag	-0.02	0.01	-2.12	-2.40

NOTE: “No longer flooded” indicates that the location is flooded (Depth ≥ 0.05m) under unmitigated conditions but dry (Depth < 0.05m) under mitigated conditions; “Not flooded” indicates that the location is dry under both unmitigated and mitigated conditions.

Change in Flood Height mapping for Year 2050 and Year 2010 1% AEP conditions is presented in Figure F11 to Figure F14. It can be seen that the effects of both options (levees and culvert duplication) are more profound under climate change conditions, due to the increased severity of flooding issue associated with the potential increase in rainfall intensity.

9.1.4. Conceptual Design and Costing

Cost is an important consideration for the feasibility of proposed measures. The duplication of the culvert under Southern Outlet (Option B) can be expensive, and only properties directly upstream of the culvert can benefit. Construction of flood levees along Whitewater Creek (Option A) is more recommended for further detailed assessment.

Conceptual design and cost estimation were conducted for the three flood levees (Option A). A typical flood levee cross-section profile shown in Diagram 10 was adopted for conceptual design. The dimensions of the levees are summarised in Table 30. The proposed profile and dimensions are based on the assumption that the ground surface is flat. Feature survey is suggested to provide information for detailed design if this option is proceeded to the next stage.

An indicative cost estimation for the levee construction is shown in Table 31 and detailed breakdown of cost for each levee is provided in APPENDIX G, based on the estimated volume required for earthwork (Table 30). Design and construction of pump systems are excluded.

It should be noted that the proposed dimensions are designed for 1% AEP with 0.5 m freeboard. The design and cost estimation can be refined to less frequent events and/or climate change scenarios. The levee height and cost of Levee 3 can be subject to a more notable increase than that of the other two levees due to the significant stormwater detention caused by the Southern Outlet Culvert.

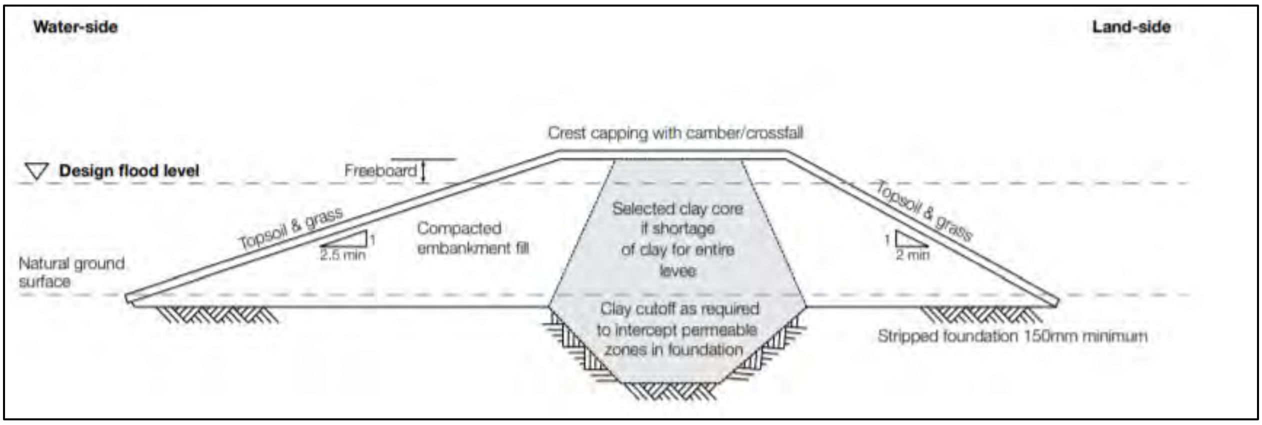


Diagram 10: Adopted Flood Levee Cross-section Profile (Reference 13)

Table 30: Key Parameters of Levees

Levee	Length (m)	Top Width (m)	Base Width (m)	Height (m)	Volume (m^3)
1	142	2	8.75	1.50	1,150
2	170	2	9.65	1.70	1,680
3	182	2	5.83	0.85	610

Table 31: Estimated Cost for Construction of Levees

Item No	Description	Amount
1	Site establishment, Construction Management Plan and Environmental management Plan	$20,000.00
2	Construction of Levee No.1 (1.5m High)	$113,370.00
3	Construction of Levee No.2 (1.7m High)	$150,930.00
4	Construction of Levee No.3 (0.85m High)	$81,730.00
5	PROVISIONAL ITEM – Contingencies (30%)	$109,809.00
6	PROVISIONAL ITEM – Design, documentation, contract administration and supervision of works (10%)	$36,603.00
	SUBTOTAL:	$512,442.00
	GST:	$51,244.20
	TOTAL AMOUNT (Including GST):	$563,686.20

9.1.5. Limitation of Structural Measures

Structural flood management measures often are not an adequate answer. The disadvantage of this strategy is its finiteness of effectiveness. For flood events above the design flood the structures lose their containment function and may result in a more significant flood impact through the breaching or failure of mitigation measures.

The probability of these events of failure will increase in the future due to climate change. However, the projections of climate change are too uncertain to be used as the basis for deriving assured design high water levels. Designing for less frequent flood is prudent to cover off on increased flood levels due to climate change.

As a consequence of the paradigm change in flood management, structural measures are no longer regarded as necessarily being the best solution to manage flooding. In fact, failure response strategies (non-structural mitigation measures), which control the pathway behind the vulnerability of the built environment, infrastructure and population are favoured.

9.2. Non-structural Mitigation Measures

Non-structural measures are a set of mitigation and/or adaptation measures that do not make use of traditional structural flood defence measures. They are designed to reduce damage without having a physical influence on the flood event.

These measures include:

- information, education and communication tools (flood maps, public presentations, collaborative platforms etc.)
- spatial planning (flood risk adapted landuse)
- building regulation and improvement of building flood resistance (wet-proofing and dry-proofing)
- flood action plans at a local scale (infrastructure maintenance)
- financial preparedness (insurance of residual risk and reserve funds)

- emergency response (evacuation and rescue plans, forecasting and warning services)
- recovery measures (disaster recovery plans, financial provisions of government).

Council has the opportunity to implement planning measures to ensure future development will be designed to have flood resilience and will not exacerbate current flood issues. Examples of planning scheme measures are map overlays that identify land subject to flooding.

The purpose of overlays is to minimise the effects of overland flows and flooding on new buildings and to ensure new developments don't adversely affect existing properties. Flood related overlays provide information on drainage issues that can be addressed at the beginning of the development proposal and/or design process.
Flood overlays can be categorised as land that is subject to flooding from –

- urban drainage systems (i.e., Special Building Overlays)
    - Control measure: set appropriate design conditions to address flood risk, i.e., floor levels
- waterways and open drainage systems, flood depth <1 metre (i.e., Land Subject to Inundation Overlays)
    - Control measure: Development works must consider a range of flood impacts and design criteria
- waterways and open drainage systems actively conveying flood flows, depths >1 metre (i.e., Floodway Overlay)
    - Control measure: Development not encouraged
- major urban overland flow path (i.e., Urban Floodway Zone)
    - Control measure: Development is typically prohibited in these areas.

Flood overlays can be determined based on flood modelling results. Figure F15 illustrate an exemplary flood overlays generated based on the flood depth outputs for 1% AEP design event. The following criteria were used to produce flood overlays:

- Special Building Overlay: main waterway area with depth between 0.05 m and 0.5 m and area outside main waterway area with depth ≥ 0.05 m
- Land Subject to Inundation Overlay: main waterway area with depth between 0.5 m and 1.0 m
- Floodway Overlay: main waterway area with depth ≥ 1.0 m.

10. CONCLUSIONS AND RECOMMENDATIONS

10.1. Conclusions

The Whitewater Creek is a tributary to Browns River, situated at the southeast coastal area of Tasmania. The Whitewater Creek catchment has a history of flooding issues as summarised in the Kingston Beach Flood Study (Kingborough Council, 2015, Reference 1). The most recent flood event occurred on 10th – 11th May 2018 was an extreme event, causing significant flood damage on properties in the catchment.

Hydrologic and hydraulic models have been developed in this study to improve the understanding of the flood issues and investigate potential solutions. The models were jointly calibrated to the May 2018 event to ensure a reliable representation of the flood characteristics within the catchment. Flood mapping, including peak flood extents, levels depths, velocities, as well as hydraulic hazards and categories, was produced for 5%, 1% and 0.5% AEP design events, which provides flood intelligence and behaviour information for Council to adopt for planning purposes. Sensitivity analysis was conducted for key factors, including roughness variations, blockage variations, and climate change scenarios (Years 2050 and 2100), to inform considerations required for floodplain risk management and future planning.

Modelling results were discussed in association with flood issues across the catchment, with a number of critical locations (hotspots) identified, including:

- the restricted access problem at Summerleas Road crossing of Whitewater Creek
- flow restrictions immediately upstream of Whitewater Crescent Culvert, Southern Outlet Culvert, and Huon Highway Culvert
- flood inundation issue at 29-31, 46-50, 59 Whitewater Crescent, and 30-34 Lester Crescent.

Three levees along the boundaries of 29-31, 46-50, 59 Whitewater Crescent, and 30-34 Lester Crescent were proposed as structural measures to alleviate flood inundation at the three hotspots. Modelling assessments indicate that the levees with proper pumping systems can improved level of flood protection for properties adjacent to Whitewater Creek, in flood events up to and including the 0.5% AEP, without creating adverse impacts on downstream properties. Conceptual design and indicative cost estimation were conducted for the proposed levees.

The duplication of Southern Outlet Culvert was assessed as another structural measure. Preliminary assessment indicates that four properties benefit from reductions in flood impacts. However, construction costs associated with duplicating the Southern Outlet culvert can be quite high and are unlikely to provide a good cost benefit solution.

Flood overlays were provided as an example of non-structural flood mitigation measures, which can be further explored in the future.

10.2. Recommendations

The modelling assessments conducted in this study are subject to limitations and uncertainties related to the input data, including LiDAR and Survey based topography, planning zones, underground drainage database, hydraulic structure information, and assumed downstream boundary conditions. It is recommended to further improve the models or at least to take the limitations into account when the information is used for future planning. Specifically, the following actions are recommended:

- Undertaking structural survey for culverts along Whitewater Creek across:
  - Summerleas Road
  - Whitewater Crescent
  - Huon Highway
- Integrating constructed/designed topography of recent and future developments, e.g., Spring Farm and Whitewater Park. The land fill and road string data of Spring Farm and Whitewater Park were provided and incorporated into the current model, which can be improved by integrating full constructed/designed surface into the model. This can be particularly important if the model is used for detailed assessment at those areas
- Integrating building footprints into the model. Preferably, Council can update its GIS database to include building footprints for future use
- Not using mapping results downstream of Huon Highway due to the uncertainty associated with the boundary condition assumptions at Browns River extracted from the Kingston Beach Flood Study (Reference 1)
- Integrating detailed underground drainage system into the model if the model is used for detailed assessment of local flood issues, e.g., lot scale assessment
- Updating climate change scenarios in accordance with new climate change guidelines when they are available. The 2050 and 2100 climate change scenarios were modelled with 10% and 30% IFD increase assumptions, respectively, adopted from the Kingston Beach Flood Study (Reference 1). Projections of climate change are highly uncertain; therefore, the climate change scenarios were undertaken as a sensitivity analysis in this study. Council should be cognisant of the uncertainty in the future climate modelling if the climate change flood mapping is adopted for planning purposes.

Structural and non-structural flood mitigation measures have been discussed and preliminarily investigated. Based on the flood issues revealed and initial assessment of the mitigation measures, the following actions are recommended:

- Implementing flood overlays to inform planning and land development
- Developing waterway maintenance plan, particularly regular removal of debris at key bridges and culverts. It should be noted that Whitewater Creek is constrained by key infrastructures, e.g., Southern Outlet Culvert, during flooding
- Further investigating other non-structural measures, e.g., promoting flood awareness education and development of emergency management plan
- Further investigating the proposed flood protection levees along Whitewater Creek as a preferred potential structural mitigation option.

Community consultation is recommended to provide insights from the community on the outputs of the study and opinions on the implementation and/or further investigation of potential mitigation measures. Based on WMAwater's experience, a combination of newsletters, questionnaires, and public consultation sessions can provide opportunities to utilize local knowledge from the community and obtain useful and feasible suggestions. Elective interviews with owners and landlords of flood affected properties are also recommended, especially for further assessments of mitigation measures.

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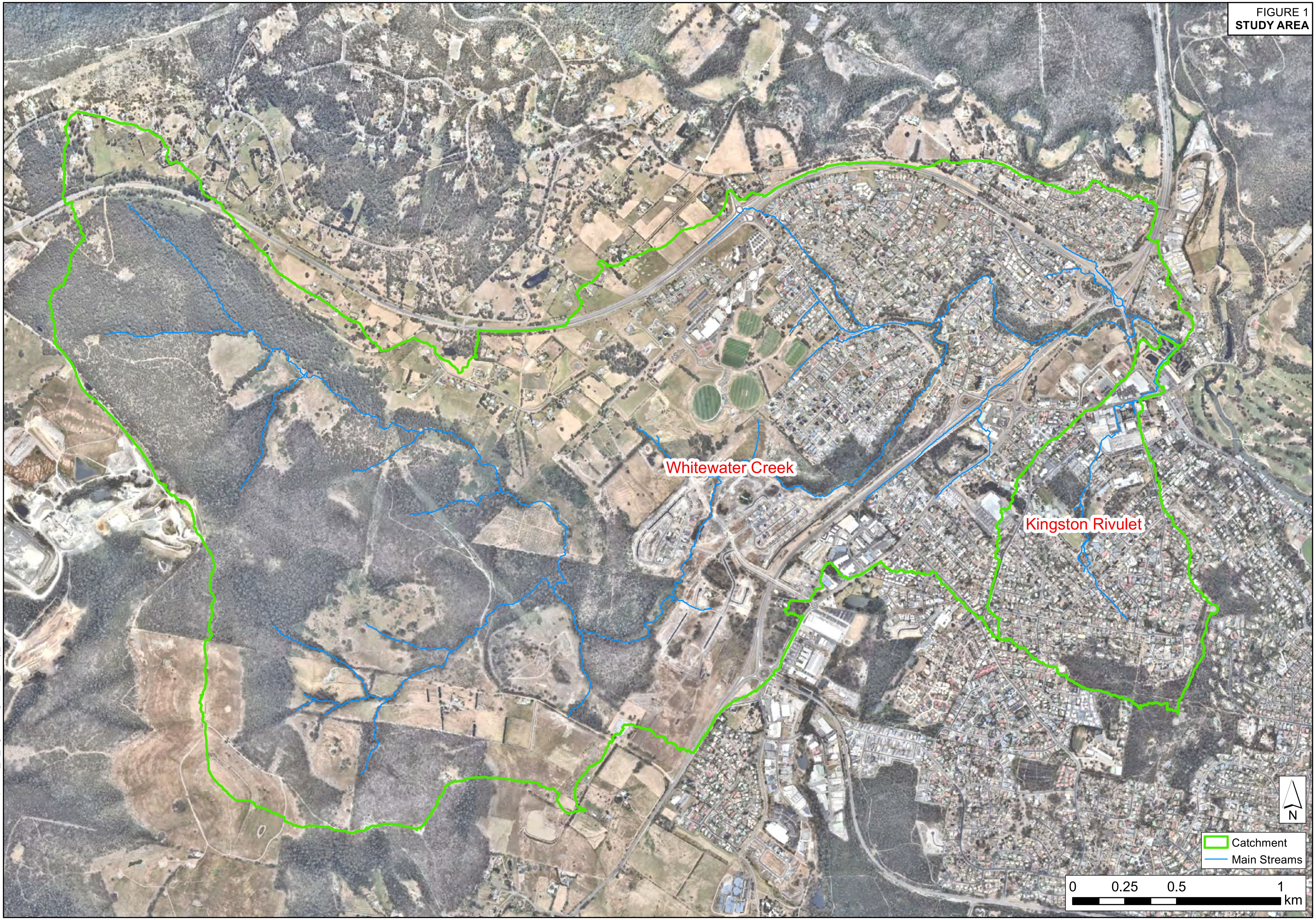
Figures



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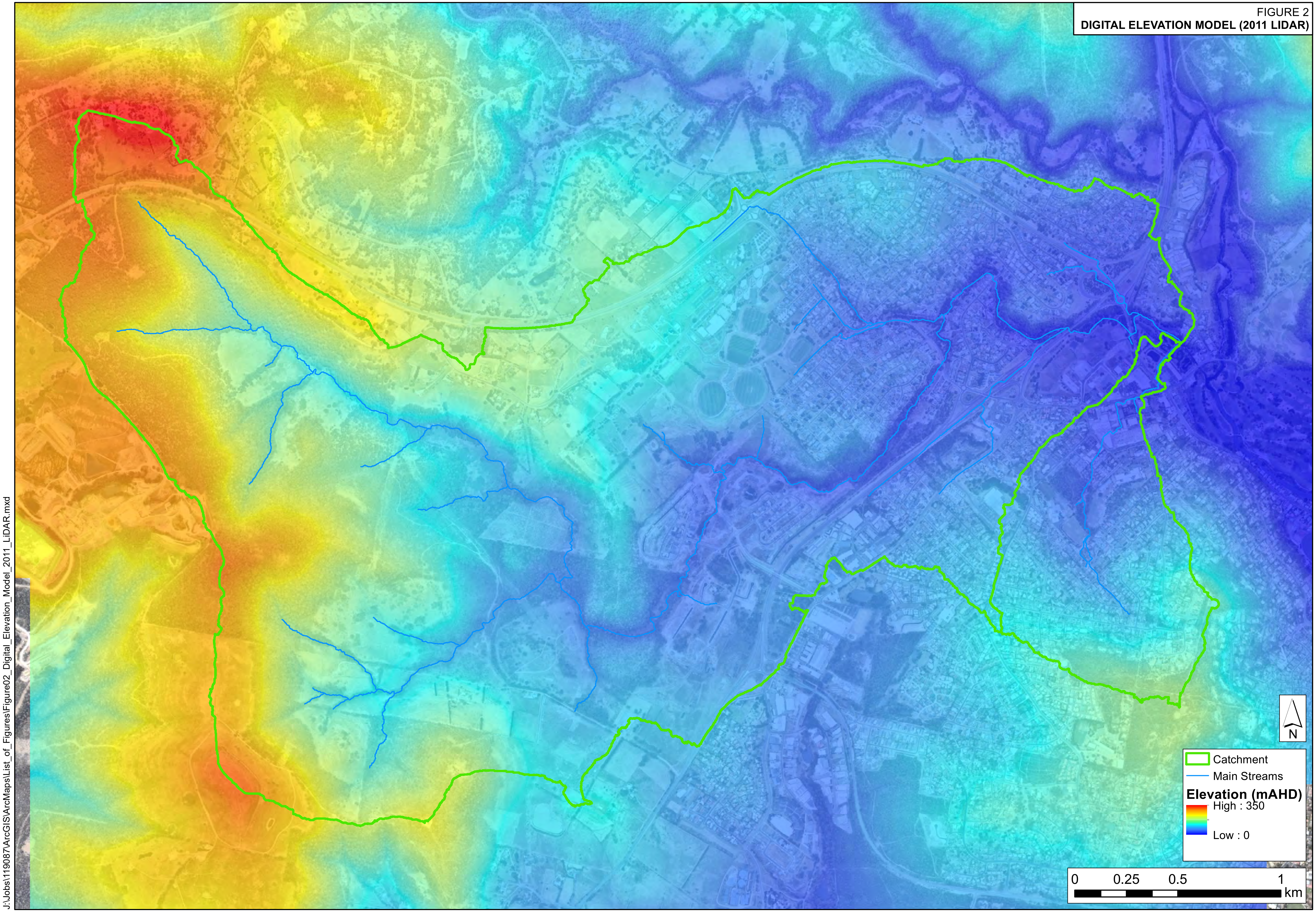
FIGURE 2
DIGITAL ELEVATION MODEL (2011 LIDAR)
Catchment
Main Streams
Elevation (mAHD)
High : 350
Low : 0
N
0
0.25
0.5
1
km
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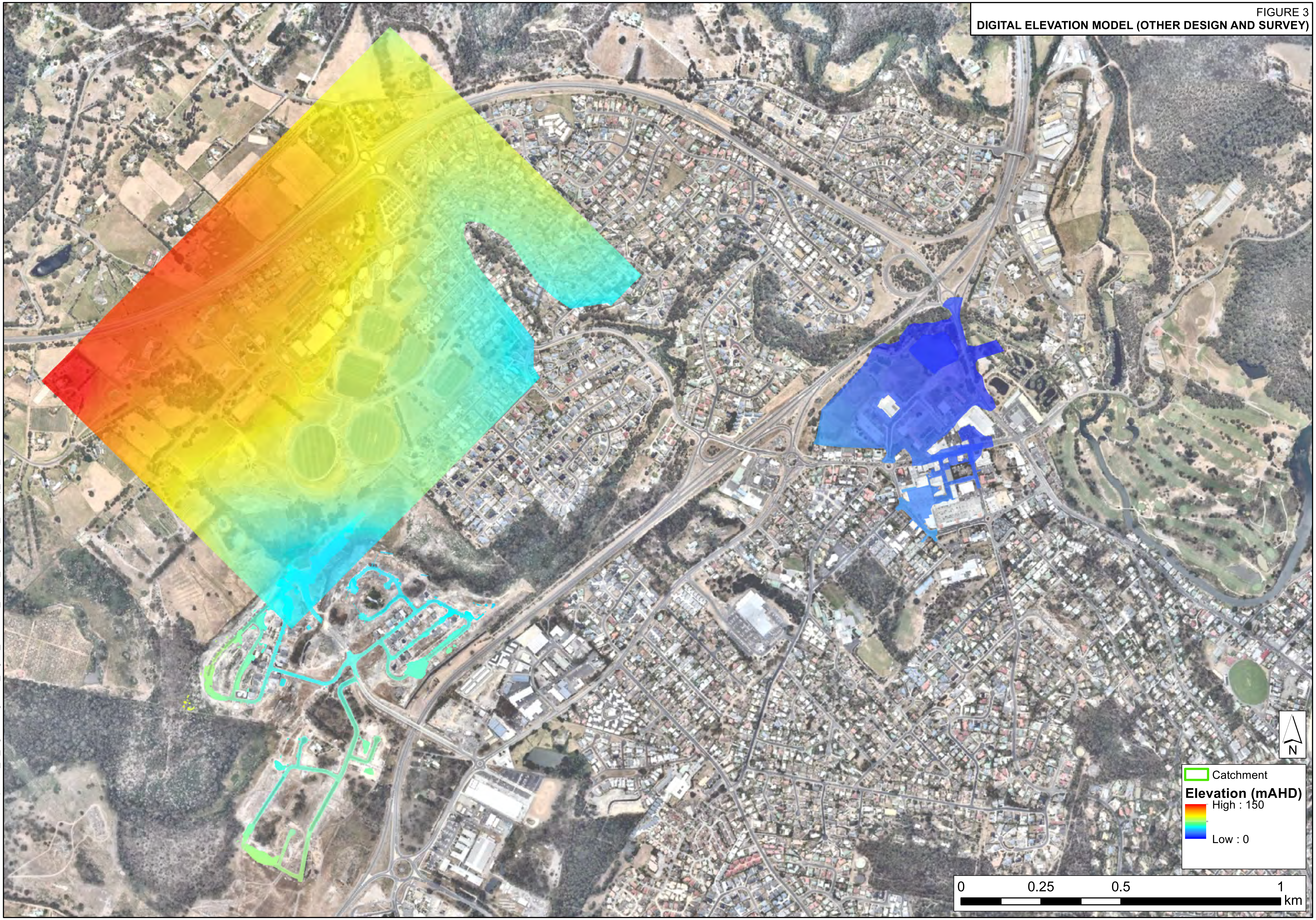

FIGURE 4
LOCATIONS OF PHOTOS AND CCTV

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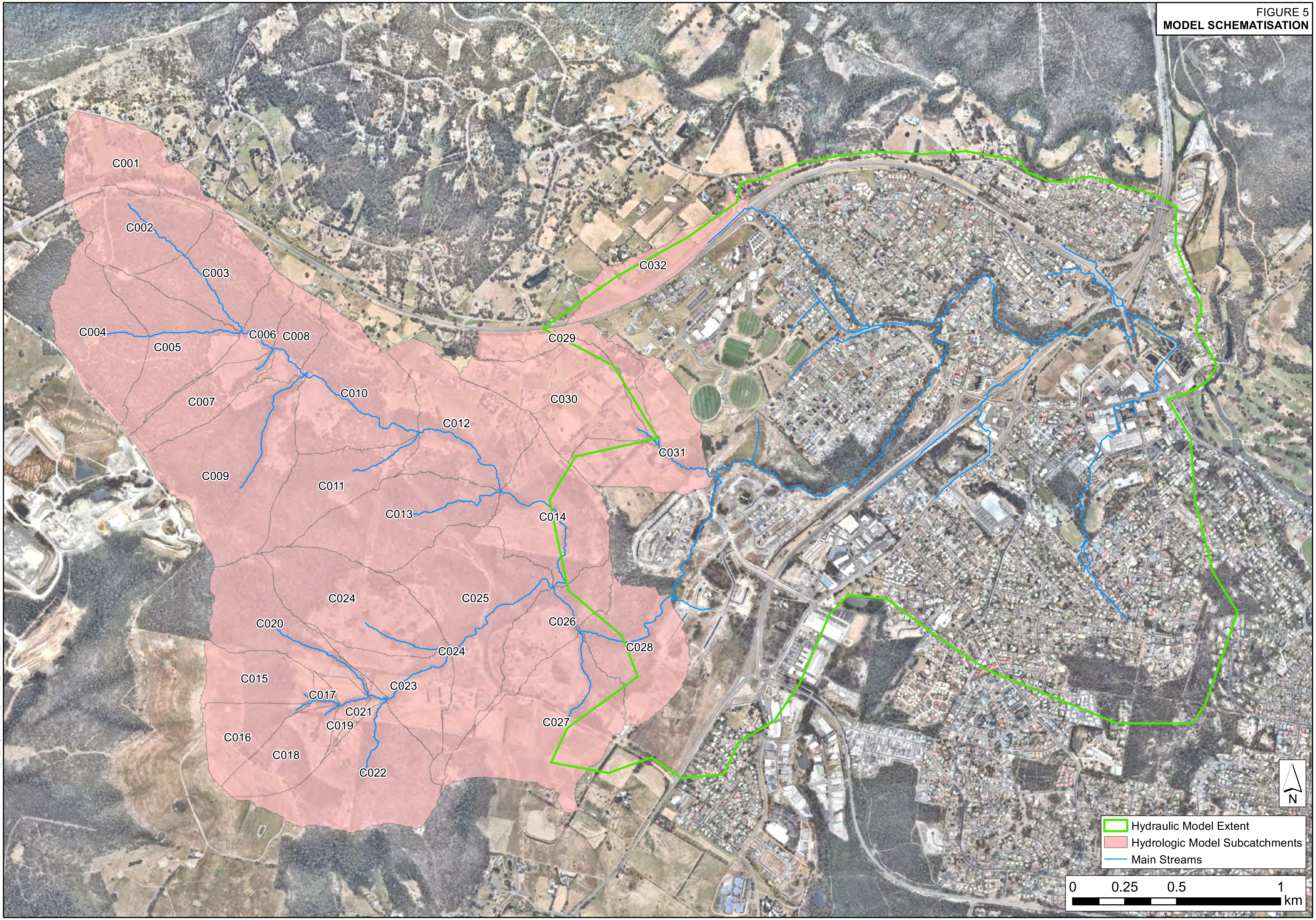

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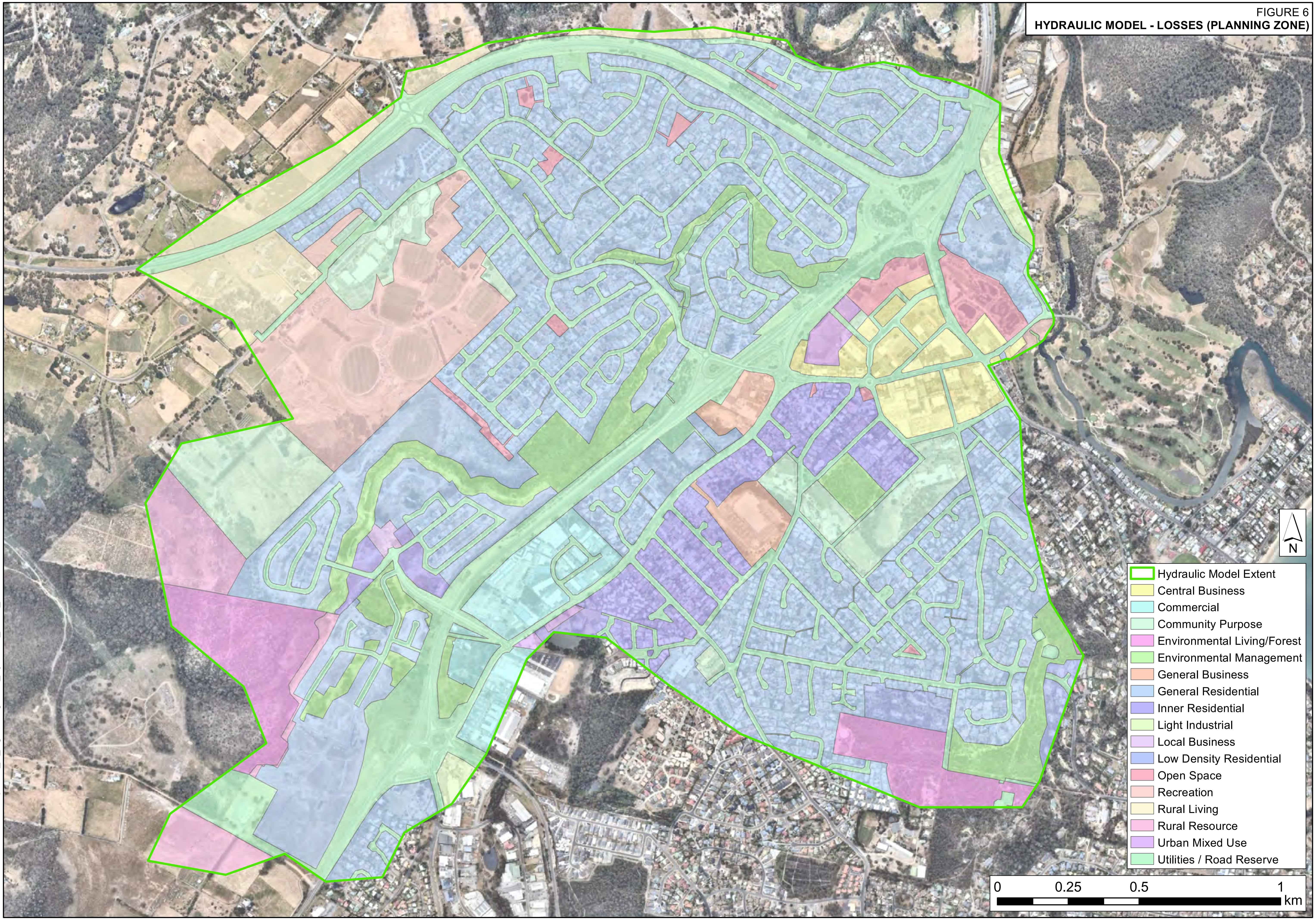
FIGURE 6

HYDRAULIC MODEL - LOSSES (PLANNING ZONE)



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J:\Jobs\119087\ArcMaps\List_of_Figures\Figure07_Hydraulic_Model_Boundary_Conditions.mxd

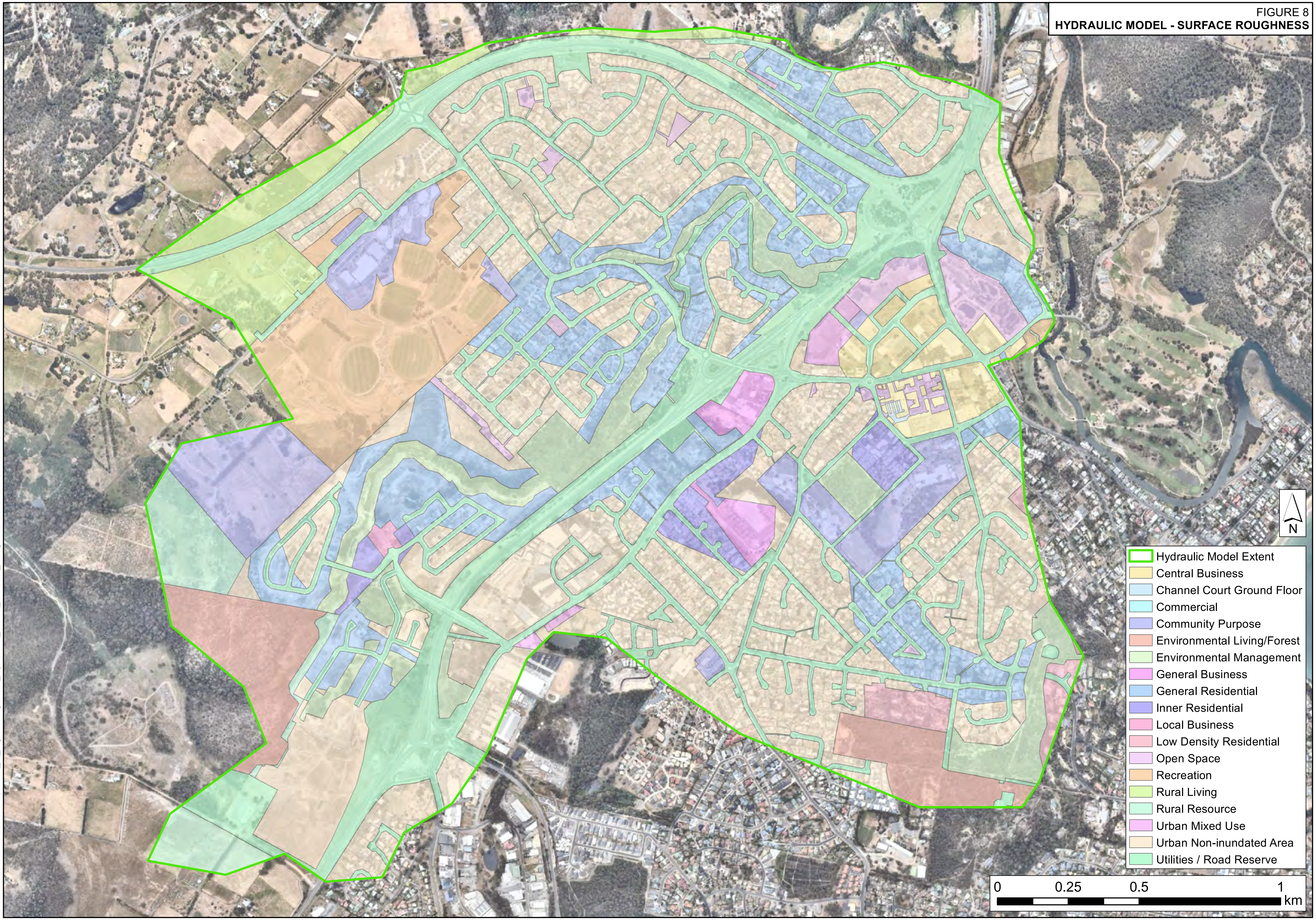

FIGURE 8
HYDRAULIC MODEL - SURFACE ROUGHNESS

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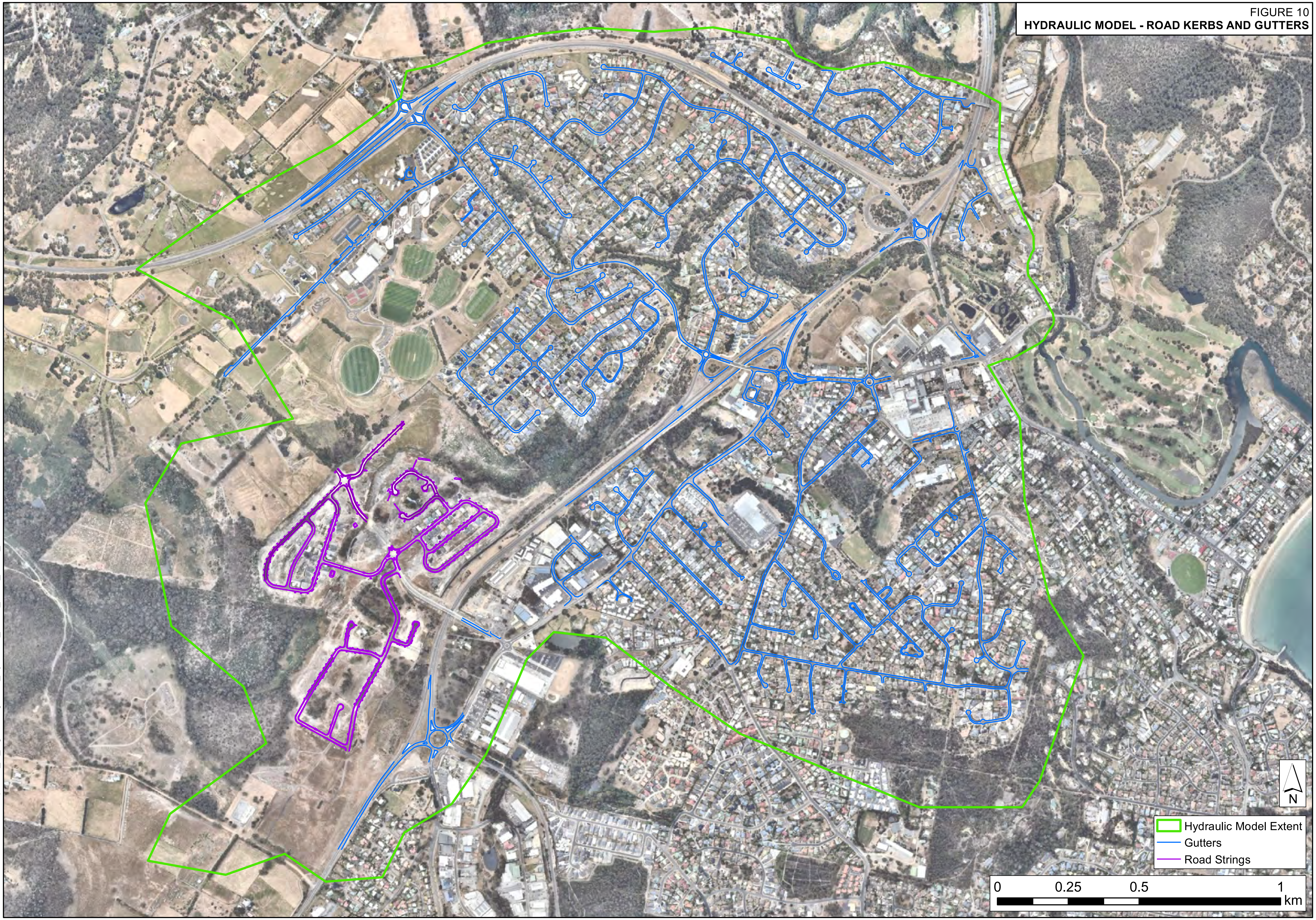
FIGURE 9
HYDRAULIC MODEL - DRAINAGE
N
Hydraulic Model Extent
Pits
Pipes
Culverts
Bridges
0
0.25
0.5
1
km
J:\Jobs\119087\ArcGIS\ArcMaps\List_of_Figures\Figure09_Hydraulic_Model_Drainage.mxd

FIGURE 10
HYDRAULIC MODEL - ROAD KERBS AND GUTTERS



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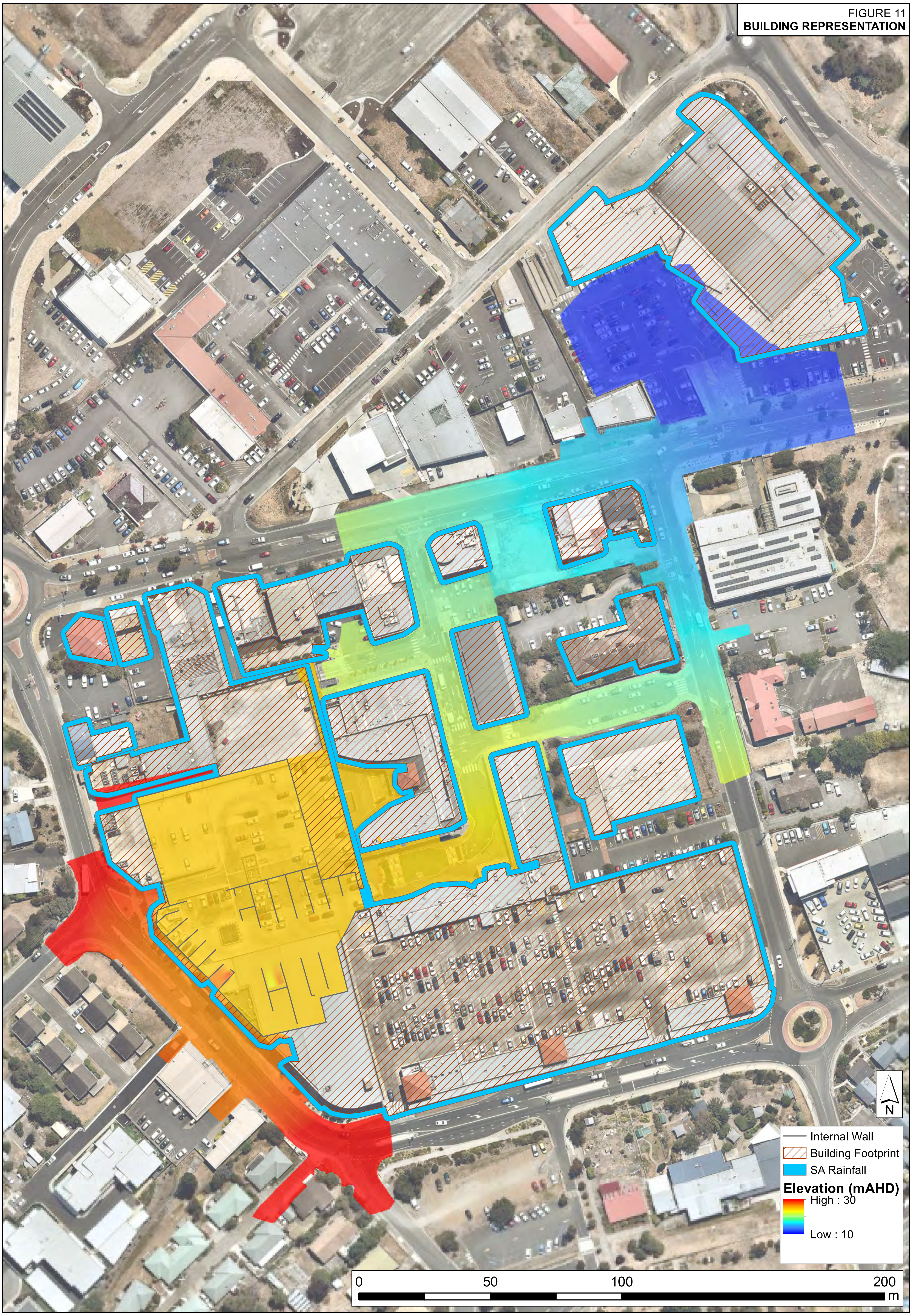

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FIGURE 12
PEAK FLOOD DEPTHS - MAY 2018 EVENT

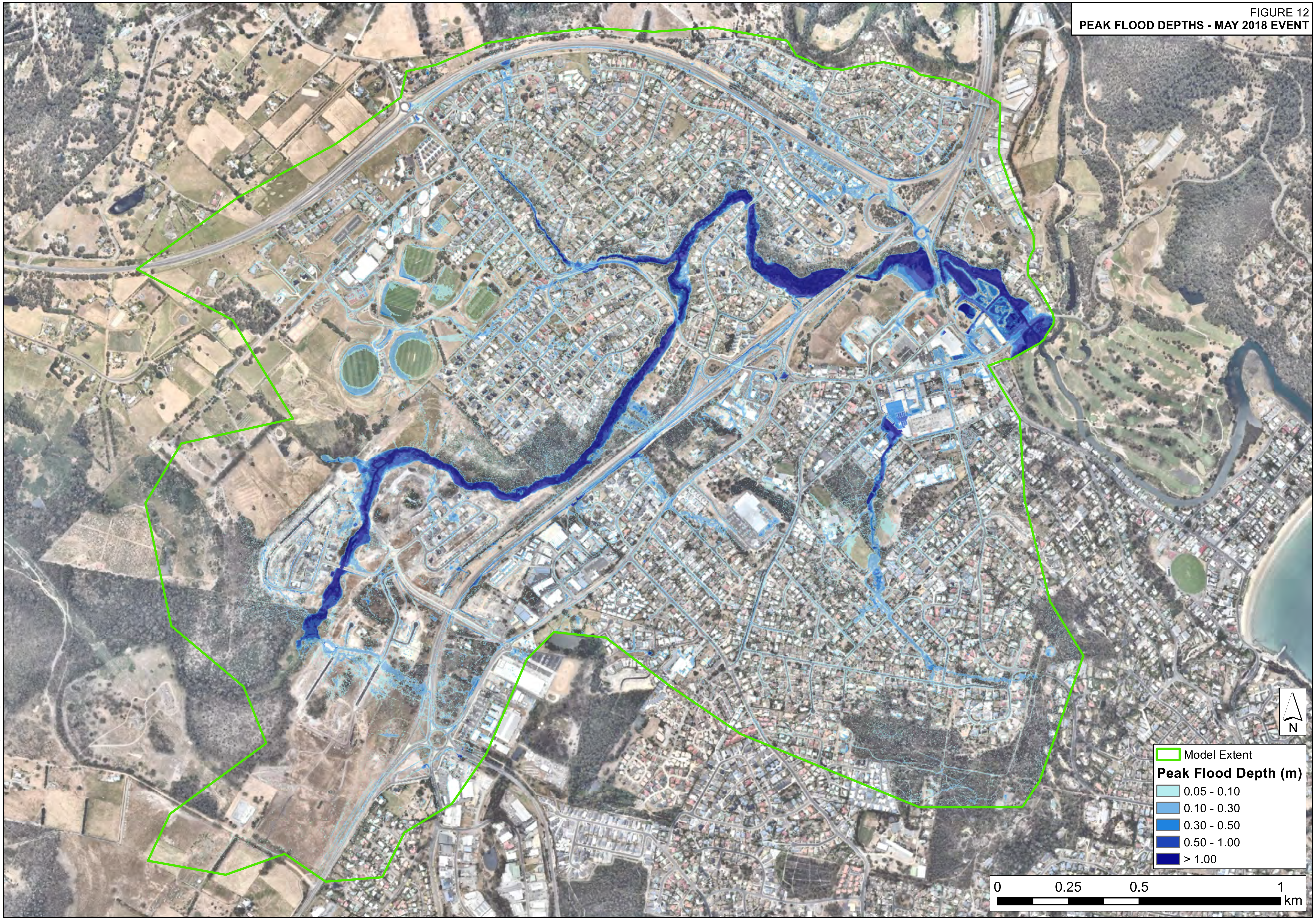

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FIGURE 13
VALIDATION LOCATIONS - MAY 2018 EVENT
34 Lester Cr
Unit 3/ 30 Lester Cr
15 Channel Hwy
Channel Court - Woolworth
Channel Court - Ground Floor in Mall
16 Freeman St
N
Model Extent
Validation Locations
Peak Flood Depth (m)
0.05 - 0.10
0.10 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
0 50 100 200
Meters
J:\Jobs\119087\ArcGIS\ArcMaps\List_of_Figures\Figure13_Validation_Locations_May 2018_Event.mxd

FIGURE 14
REPORTING LOCATIONS
PEAK FLOOD DEPTHS - EXISTING
1% AEP EVENT

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Appendix A

APPENDIX A. GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils — Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.

Annual Exceedance Probability (AEP) — The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).

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

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

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

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

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

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

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

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

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

	redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.

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

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

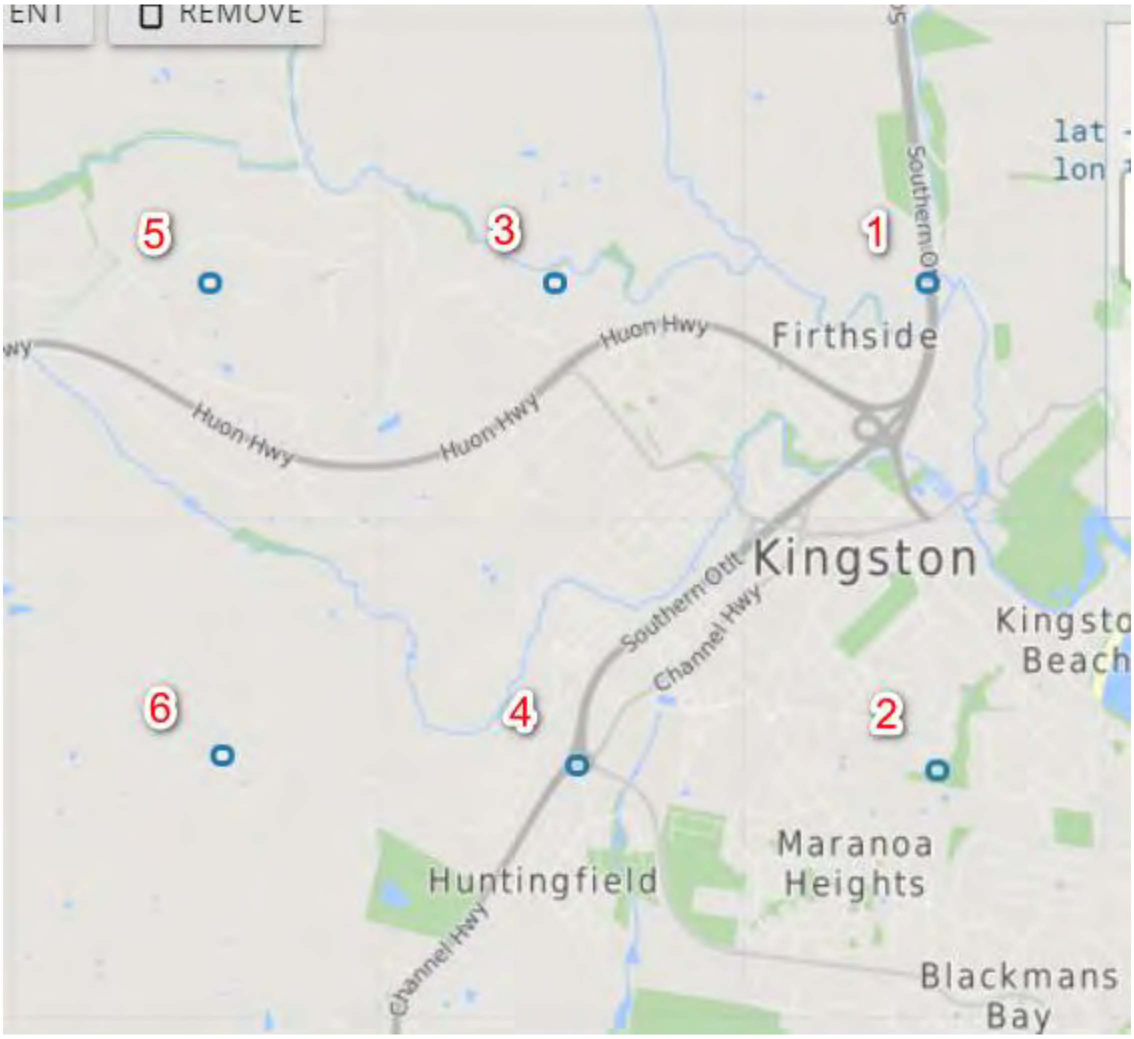
	complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard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

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

APPENDIX B. IFDs

Appendix B

B.1. IFD Grids


REMOVE
lat
lon
1
2
3
4
5
6
Southern Otl
Firthside
Huon Hwy
Huon Hwy
Huon Hwy
Kingston
Southern Otlt
Channel Hwy
Kingston Beach
Maranoa Heights
Huntingfield
Channel Hwy
Blackmans Bay

B.2. IFD Tables

Table B 1: BoM 2016 IFD for Grid 1

Grid 1	Annual Exceedance Probability (AEP) Rainfall intensity in mm/h										
Duration	**63.20 %**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**	**1 in 200**	**1 in 500**	**1 in 1000**	**1 in 2000**
1 min	1.04	1.18	1.66	2.01	2.38	2.9	3.34	3.8	4.41	4.9	5.41
2 min	1.79	2.02	2.75	3.26	3.77	4.43	4.94	5.6	6.49	7.21	7.98
3 min	2.37	2.68	3.69	4.4	5.11	6.06	6.8	7.71	8.95	9.94	11
4 min	2.85	3.23	4.48	5.38	6.29	7.53	8.52	9.68	11.2	12.5	13.8
5 min	3.26	3.7	5.16	6.22	7.31	8.83	10.1	11.4	13.3	14.8	16.3
10 min	4.71	5.36	7.57	9.22	11	13.5	15.6	17.8	20.6	22.9	25.3
15 min	5.71	6.5	9.18	11.2	13.3	16.4	19	21.6	25.1	27.9	30.8
20 min	6.51	7.41	10.4	12.7	15.1	18.5	21.4	24.4	28.3	31.5	34.8
25 min	7.2	8.18	11.5	14	16.5	20.2	23.3	26.5	30.8	34.2	37.8
30 min	7.82	8.87	12.4	15	17.8	21.7	24.9	28.3	32.9	36.5	40.4
45 min	9.39	10.6	14.8	17.8	20.9	25.2	28.7	32.7	37.9	42.1	46.6
1 hour	10.7	12.2	16.8	20.2	23.5	28.2	31.9	36.2	42	46.7	51.6
1.5 hour	13	14.7	20.3	24.2	28.1	33.3	37.3	42.5	49.3	54.7	60.5
2 hour	15	17	23.4	27.8	32.2	37.9	42.3	48.1	55.8	62	68.5
3 hour	18.3	20.9	28.9	34.2	39.4	46.2	51.4	58.5	67.9	75.4	83.4
4.5 hour	22.5	25.7	35.8	42.5	49	57.4	63.8	72.7	84.4	93.7	104
6 hour	25.9	29.8	41.8	49.7	57.4	67.4	75	85.6	99.4	110	122
9 hour	31.5	36.4	51.6	61.8	71.6	84.7	94.7	108	125	139	154
12 hour	35.9	41.6	59.4	71.5	83.2	99.1	111	127	147	164	181
18 hour	42.4	49.3	71.2	86.3	101	121	137	156	182	202	223
24 hour	47.1	54.8	79.6	96.8	114	137	156	178	206	229	253
30 hour	50.6	58.9	85.7	104	123	149	170	194	225	250	276
36 hour	53.4	62.1	90.3	110	130	158	180	205	237	264	292
48 hour	57.4	66.6	96.7	118	140	169	193	219	253	282	312
72 hour	62.3	72	104	126	149	180	205	230	267	297	329
96 hour	65.5	75.4	107	130	152	183	208	234	273	303	336
120 hour	68	78	110	132	155	185	209	237	276	307	340
144 hour	70.2	80.4	113	135	156	186	210	238	279	310	343
168 hour	72.3	82.8	115	137	159	188	212	240	281	313	346

Note:
 # The 50% AEP IFD does not correspond to the 2 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 1.44 ARI.
** The 20% AEP IFD does not correspond to the 5 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 4.48 ARI.*

Table B 2: BoM 2016 IFD for Grid 2

Grid 2	**Annual Exceedance Probability (AEP) Rainfall intensity in mm/h**										
Duration	**63.20 %**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**	**1 in 200**	**1 in 500**	**1 in 1000**	**1 in 2000**
1 min	1.04	1.19	1.67	2.02	2.39	2.92	3.36	3.82	4.43	4.92	5.44
2 min	1.79	2.03	2.77	3.28	3.79	4.45	4.97	5.63	6.53	7.26	8.03
3 min	2.38	2.7	3.71	4.42	5.14	6.09	6.84	7.76	9	10	11.1
4 min	2.87	3.25	4.51	5.41	6.32	7.58	8.58	9.74	11.3	12.6	13.9
5 min	3.27	3.72	5.19	6.26	7.36	8.89	10.1	11.5	13.4	14.8	16.4
10 min	4.73	5.39	7.61	9.27	11	13.6	15.7	17.8	20.7	23	25.4
15 min	5.74	6.53	9.22	11.2	13.4	16.5	19.1	21.7	25.2	28	31
20 min	6.54	7.44	10.5	12.8	15.2	18.6	21.5	24.5	28.4	31.6	34.9
25 min	7.23	8.21	11.5	14	16.6	20.3	23.4	26.6	30.9	34.4	38
30 min	7.85	8.91	12.5	15.1	17.9	21.8	25	28.5	33	36.7	40.5
45 min	9.43	10.7	14.9	17.9	21	25.4	28.9	32.8	38.1	42.3	46.8
1 hour	10.8	12.2	16.9	20.3	23.7	28.4	32.1	36.4	42.3	47	52
1.5 hour	13.1	14.9	20.5	24.4	28.3	33.6	37.7	42.8	49.7	55.2	61
2 hour	15.1	17.1	23.7	28.1	32.5	38.3	42.8	48.6	56.4	62.7	69.3
3 hour	18.5	21.1	29.2	34.7	40	47	52.2	59.4	69	76.6	84.7
4.5 hour	22.8	26.1	36.4	43.3	49.9	58.6	65.1	74.1	86.1	95.6	106
6 hour	26.3	30.3	42.6	50.8	58.6	69	76.8	87.5	102	113	125
9 hour	32.1	37.2	52.8	63.3	73.5	87.1	97.4	111	129	143	158
12 hour	36.7	42.6	61.1	73.5	85.7	102	115	131	152	169	186
18 hour	43.5	50.7	73.4	89	104	125	142	161	187	208	230
24 hour	48.4	56.4	82.1	100	118	142	161	184	213	237	262
30 hour	52.1	60.7	88.5	108	128	154	176	201	232	258	286
36 hour	55	64	93.3	114	135	163	186	212	246	273	302
48 hour	59.2	68.8	100	122	144	175	200	226	262	291	323
72 hour	64.4	74.5	107	130	154	186	211	238	276	307	340
96 hour	67.7	78	111	134	157	189	215	242	282	314	347
120 hour	70.3	80.7	114	137	160	191	216	245	285	317	351
144 hour	72.7	83.2	117	139	162	193	217	246	288	320	355
168 hour	74.9	85.7	119	142	164	195	219	248	291	323	358

Table B 3: BoM 2016 IFD for Grid 3

Grid 3	Annual Exceedance Probability (AEP) Rainfall intensity in mm/h									
Duration	**63.20 %**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**	**1 in 200**	**1 in 500**	**1 in 1000**	**1 in 2000**
1 min	1.04	1.19	1.67	2.03	2.4	2.93	3.37	3.83	4.45	4.94	5.46
2 min	1.8	2.03	2.78	3.3	3.81	4.49	5.01	5.69	6.59	7.32	8.1
3 min	2.39	2.7	3.73	4.44	5.17	6.13	6.89	7.83	9.07	10.1	11.1
4 min	2.87	3.25	4.52	5.43	6.35	7.62	8.63	9.8	11.4	12.6	14
5 min	3.27	3.72	5.2	6.28	7.38	8.93	10.2	11.6	13.4	14.9	16.5
10 min	4.73	5.39	7.62	9.29	11	13.6	15.7	17.9	20.7	23	25.5
15 min	5.73	6.53	9.23	11.3	13.4	16.5	19.1	21.8	25.3	28.1	31
20 min	6.54	7.44	10.5	12.8	15.2	18.7	21.6	24.5	28.5	31.7	35
25 min	7.23	8.22	11.6	14	16.7	20.4	23.5	26.7	31	34.5	38.1
30 min	7.84	8.91	12.5	15.2	17.9	21.9	25.1	28.6	33.1	36.8	40.7
45 min	9.43	10.7	14.9	18	21.1	25.5	29	33	38.3	42.5	47
1 hour	10.8	12.2	17	20.4	23.8	28.5	32.3	36.7	42.5	47.3	52.3
1.5 hour	13.1	14.9	20.6	24.5	28.5	33.8	37.9	43.1	50	55.6	61.4
2 hour	15.1	17.2	23.7	28.2	32.7	38.5	43.1	49	56.8	63.1	69.8
3 hour	18.5	21.2	29.3	34.8	40.2	47.2	52.5	59.8	69.4	77.1	85.2
4.5 hour	22.8	26.1	36.5	43.4	50.1	58.8	65.4	74.4	86.4	96	106
6 hour	26.3	30.3	42.6	50.9	58.8	69.2	77.1	87.7	102	113	125
9 hour	32.1	37.1	52.8	63.3	73.5	87.1	97.4	111	129	143	158
12 hour	36.6	42.5	60.9	73.4	85.5	102	115	130	152	168	186
18 hour	43.4	50.5	73	88.5	104	125	141	161	187	207	229
24 hour	48.2	56.2	81.6	99.3	117	141	160	182	212	235	260
30 hour	51.9	60.4	87.9	107	126	153	174	199	231	256	284
36 hour	54.8	63.7	92.6	113	134	162	185	210	244	271	300
48 hour	59	68.4	99.1	121	143	173	198	224	260	289	320
72 hour	64.2	74	106	129	152	184	209	236	274	304	337
96 hour	67.5	77.5	110	133	156	188	213	240	279	311	344
120 hour	70.1	80.3	113	136	159	190	214	242	283	314	348
144 hour	72.5	82.9	116	138	161	192	216	244	285	317	351
168 hour	74.8	85.5	119	142	164	194	218	245	288	320	354

Table B 4: BoM 2016 IFD for Grid 4

Grid 4	Annual Exceedance Probability (AEP) Rainfall intensity in mm/h									
Duration	**63.20 %**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**	**1 in 200**	**1 in 500**	**1 in 1000**	**1 in 2000**
1 min	1.04	1.19	1.67	2.02	2.39	2.92	3.36	3.82	4.43	4.93	5.45
2 min	1.79	2.03	2.77	3.28	3.79	4.46	4.98	5.66	6.55	7.28	8.06
3 min	2.38	2.7	3.71	4.42	5.14	6.1	6.85	7.79	9.03	10	11.1
4 min	2.86	3.25	4.5	5.4	6.32	7.58	8.59	9.76	11.3	12.6	13.9
5 min	3.27	3.71	5.18	6.25	7.35	8.89	10.1	11.5	13.4	14.9	16.4
10 min	4.72	5.37	7.59	9.26	11	13.5	15.7	17.8	20.7	23	25.4
15 min	5.72	6.51	9.2	11.2	13.4	16.5	19.1	21.7	25.2	28	31
20 min	6.53	7.42	10.5	12.7	15.1	18.6	21.5	24.4	28.4	31.6	34.9
25 min	7.22	8.2	11.5	14	16.6	20.3	23.4	26.6	30.9	34.4	38
30 min	7.83	8.89	12.5	15.1	17.8	21.8	25	28.4	33	36.7	40.6
45 min	9.41	10.7	14.9	17.9	21	25.4	28.9	32.8	38.1	42.4	46.9
1 hour	10.8	12.2	16.9	20.3	23.7	28.4	32.1	36.5	42.3	47.1	52.1
1.5 hour	13	14.8	20.5	24.4	28.3	33.6	37.7	42.9	49.8	55.3	61.2
2 hour	15	17.1	23.6	28.1	32.5	38.4	42.9	48.7	56.6	62.9	69.5
3 hour	18.4	21	29.2	34.7	40	47	52.3	59.5	69.1	76.8	84.9
4.5 hour	22.6	26	36.3	43.2	49.9	58.6	65.2	74.2	86.2	95.8	106
6 hour	26.2	30.1	42.5	50.7	58.6	69	76.9	87.5	102	113	125
9 hour	31.9	36.9	52.6	63.1	73.3	86.9	97.3	111	129	143	158
12 hour	36.4	42.2	60.7	73.2	85.4	102	114	130	151	168	186
18 hour	43.1	50.2	72.8	88.4	104	125	141	161	187	207	229
24 hour	47.9	55.9	81.4	99.1	117	141	160	182	212	235	260
30 hour	51.6	60.1	87.7	107	126	153	174	199	231	256	284
36 hour	54.4	63.4	92.4	113	133	162	185	210	243	271	300
48 hour	58.7	68.2	98.9	121	143	173	198	223	259	288	319
72 hour	63.9	73.8	106	129	151	183	208	235	273	303	336
96 hour	67.3	77.3	110	132	155	186	212	239	278	310	343
120 hour	69.9	80.1	113	135	157	188	213	241	281	313	347
144 hour	72.2	82.6	115	138	160	190	214	242	284	316	350
168 hour	74.5	85.2	118	141	162	193	217	243	287	319	353

Table B 5: BoM 2016 IFD for Grid 5

Grid 5	**Annual Exceedance Probability (AEP) Rainfall intensity in mm/h**									
Duration	**63.20 %**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**	**1 in 200**	**1 in 500**	**1 in 1000**	**1 in 2000**
1 min	1.06	1.21	1.71	2.08	2.46	3.01	3.46	3.93	4.57	5.08	5.61
2 min	1.83	2.08	2.86	3.41	3.95	4.65	5.2	5.89	6.86	7.62	8.43
3 min	2.44	2.77	3.84	4.59	5.35	6.35	7.14	8.1	9.42	10.5	11.6
4 min	2.93	3.33	4.65	5.6	6.56	7.87	8.91	10.1	11.8	13.1	14.5
5 min	3.34	3.81	5.35	6.46	7.61	9.2	10.5	11.9	13.8	15.4	17
10 min	4.81	5.49	7.79	9.51	11.3	13.9	16.1	18.3	21.2	23.6	26.1
15 min	5.83	6.65	9.43	11.5	13.7	16.9	19.6	22.3	25.8	28.7	31.8
20 min	6.64	7.57	10.7	13.1	15.5	19.1	22.1	25.1	29.1	32.4	35.8
25 min	7.35	8.37	11.8	14.4	17.1	20.9	24.1	27.4	31.8	35.3	39.1
30 min	7.98	9.08	12.8	15.5	18.4	22.4	25.8	29.3	34	37.8	41.8
45 min	9.62	10.9	15.3	18.5	21.7	26.3	29.9	34	39.5	43.9	48.5
1 hour	11	12.6	17.5	21	24.6	29.5	33.4	37.9	44.1	49	54.2
1.5 hour	13.5	15.4	21.4	25.5	29.7	35.2	39.5	44.9	52.2	58	64.1
2 hour	15.6	17.8	24.8	29.5	34.2	40.4	45.2	51.3	59.6	66.3	73.3
3 hour	19.4	22.2	30.9	36.8	42.5	49.9	55.6	63.2	73.3	81.6	90.2
4.5 hour	24.1	27.7	38.8	46.2	53.4	62.7	69.8	79.4	92.1	102	113
6 hour	28	32.4	45.7	54.5	63	74.3	82.8	94.2	109	122	134
9 hour	34.5	40	57	68.4	79.5	94.2	105	120	139	155	171
12 hour	39.7	46.1	66.1	79.7	92.9	111	125	142	164	183	202
18 hour	47.4	55.2	79.8	96.8	114	136	154	176	204	227	251
24 hour	52.9	61.6	89.4	109	128	155	176	200	232	258	286
30 hour	57.1	66.4	96.5	118	139	169	192	220	255	283	313
36 hour	60.4	70.1	102	124	147	179	204	233	270	299	332
48 hour	65.2	75.4	109	133	158	192	219	249	289	320	354
72 hour	71.1	81.8	117	143	169	204	232	261	304	338	374
96 hour	75	85.8	122	147	174	209	237	266	309	345	382
120 hour	78.1	89.2	125	151	177	212	239	268	312	349	386
144 hour	81	92.4	129	155	180	215	242	271	314	352	390
168 hour	83.9	95.7	133	159	184	218	246	274	315	356	394

Table B 6: BoM 2016 IFD for Grid 6

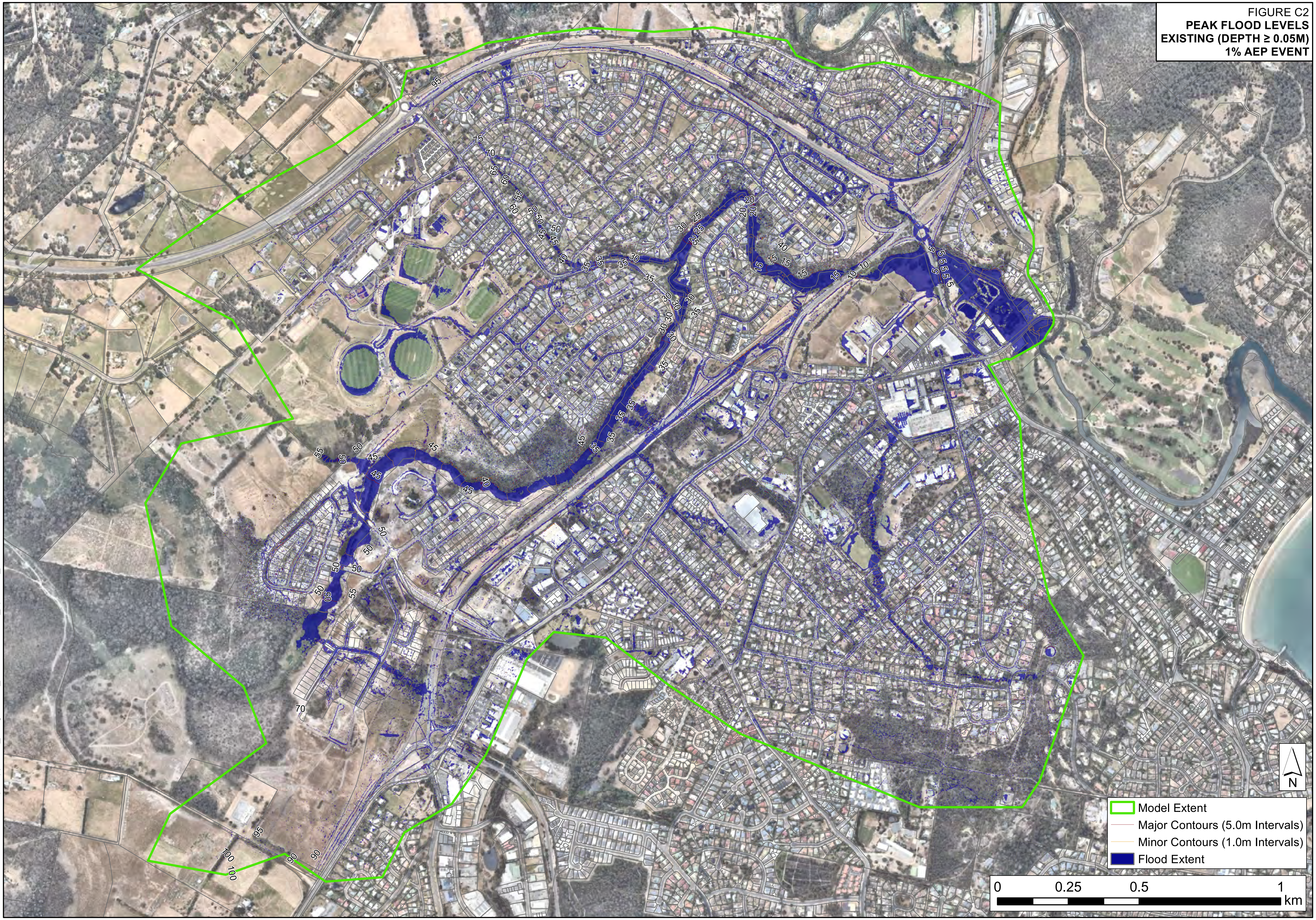
Grid 6	Annual Exceedance Probability (AEP) Rainfall intensity in mm/h									
Duration	**63.20 %**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**	**1 in 200**	**1 in 500**	**1 in 1000**	**1 in 2000**
1 min	1.07	1.22	1.72	2.09	2.48	3.02	3.48	3.96	4.59	5.11	5.65
2 min	1.84	2.09	2.87	3.42	3.96	4.67	5.21	5.92	6.88	7.64	8.46
3 min	2.45	2.78	3.85	4.61	5.36	6.37	7.16	8.14	9.46	10.5	11.6
4 min	2.94	3.34	4.67	5.62	6.58	7.9	8.95	10.2	11.8	13.1	14.5
5 min	3.36	3.82	5.37	6.49	7.64	9.24	10.5	12	13.9	15.5	17.1
10 min	4.83	5.51	7.82	9.55	11.4	14	16.2	18.4	21.4	23.8	26.3
15 min	5.85	6.67	9.47	11.6	13.8	17	19.7	22.4	26	28.9	32
20 min	6.67	7.6	10.8	13.1	15.6	19.2	22.2	25.2	29.3	32.6	36.1
25 min	7.38	8.4	11.9	14.4	17.1	21	24.2	27.5	32	35.5	39.3
30 min	8.01	9.12	12.8	15.6	18.4	22.5	25.9	29.4	34.2	38	42.1
45 min	9.66	11	15.4	18.5	21.8	26.4	30	34.2	39.7	44.1	48.8
1 hour	11.1	12.6	17.6	21.1	24.7	29.6	33.5	38.1	44.3	49.2	54.5
1.5 hour	13.6	15.4	21.5	25.6	29.8	35.4	39.7	45.2	52.5	58.4	64.6
2 hour	15.7	17.9	24.9	29.7	34.4	40.7	45.4	51.7	60	66.8	73.8
3 hour	19.5	22.3	31.2	37.1	42.8	50.3	56	63.7	74	82.4	91.1
4.5 hour	24.3	27.9	39.2	46.7	54	63.4	70.6	80.3	93.3	104	115
6 hour	28.3	32.7	46.2	55.2	63.9	75.2	83.9	95.5	111	123	136
9 hour	35	40.6	57.9	69.5	80.8	95.8	107	122	142	158	174
12 hour	40.3	46.8	67.3	81.1	94.7	113	127	145	168	187	206
18 hour	48.2	56.2	81.4	98.8	116	140	158	180	209	232	257
24 hour	53.9	62.8	91.4	111	131	159	180	205	238	265	293
30 hour	58.2	67.8	98.7	121	143	173	197	225	261	290	321
36 hour	61.6	71.6	104	127	151	183	209	239	277	307	340
48 hour	66.6	77.1	112	137	162	197	225	255	296	328	364
72 hour	72.7	83.7	120	146	173	209	238	268	312	347	384
96 hour	76.7	87.9	125	151	178	214	243	273	317	354	392
120 hour	79.9	91.3	128	155	181	217	245	276	320	358	397
144 hour	82.9	94.5	132	158	184	220	248	278	321	362	400
168 hour	85.9	97.9	136	162	188	223	251	281	323	365	404

APPENDIX C. Design Flood Mapping

Appendix C

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC01_PeakFloodLevel_5pctAEP_Event.mxd

FIGURE C2
PEAK FLOOD LEVELS
EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT

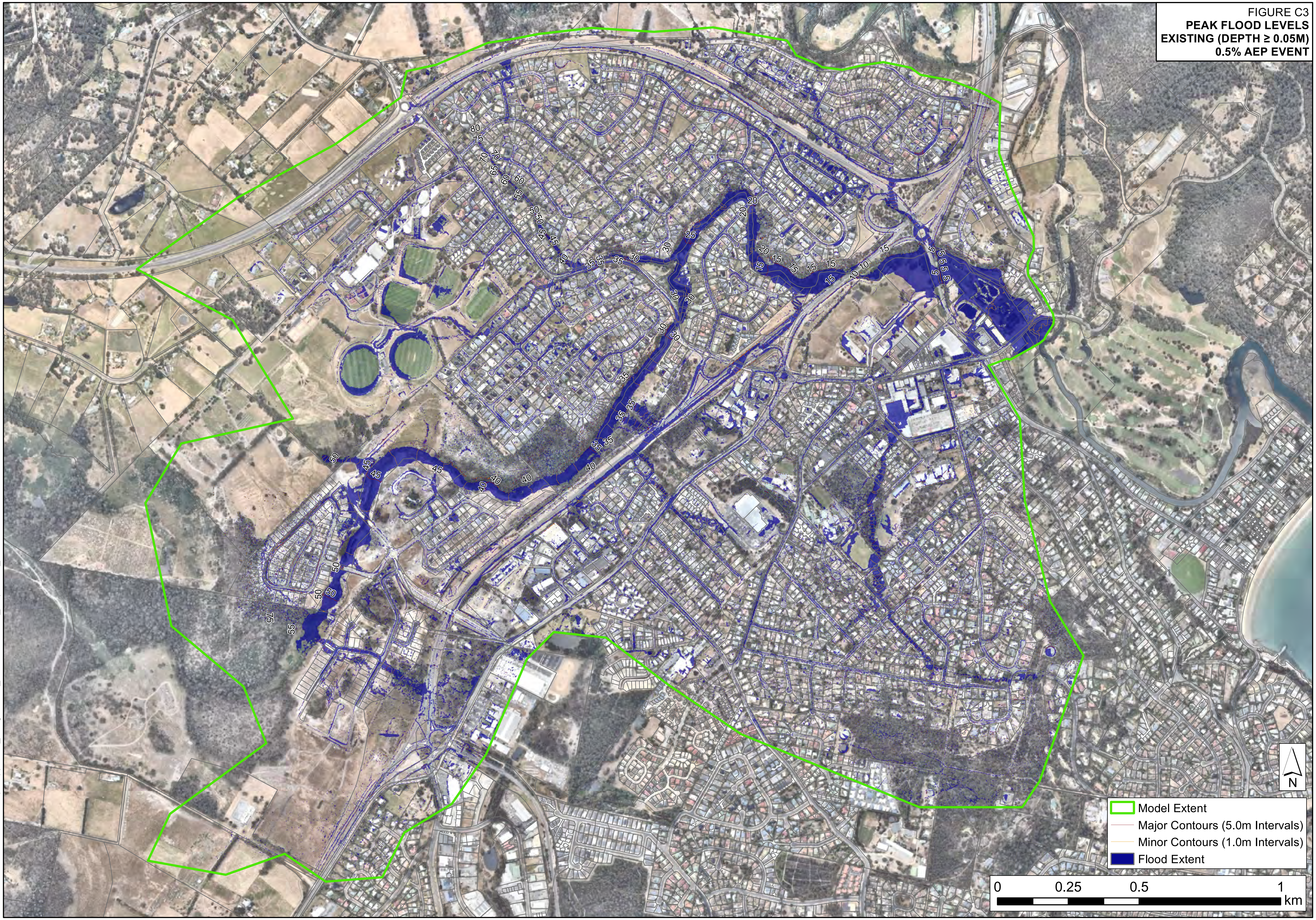


Model Extent
Major Contours (5.0m Intervals)
Minor Contours (1.0m Intervals)
Flood Extent

0 0.25 0.5 1 km

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC02_PeakFloodLevel_1pctAEP_Event.mxd

FIGURE C3
PEAK FLOOD LEVELS
EXISTING (DEPTH ≥ 0.05M)
0.5% AEP EVENT



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC03_PeakFloodLevel_0.5pctAEP_Event.mxd

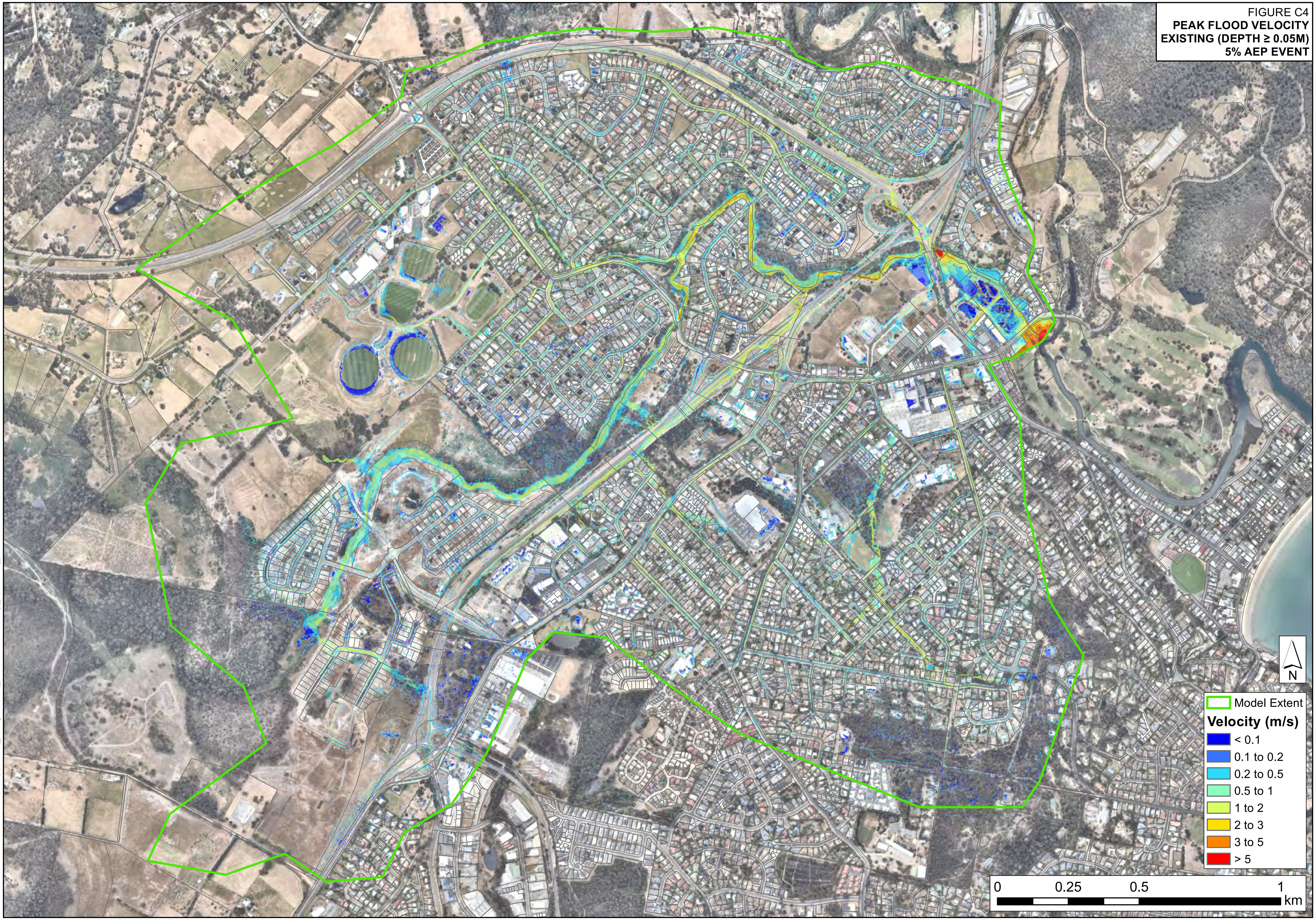
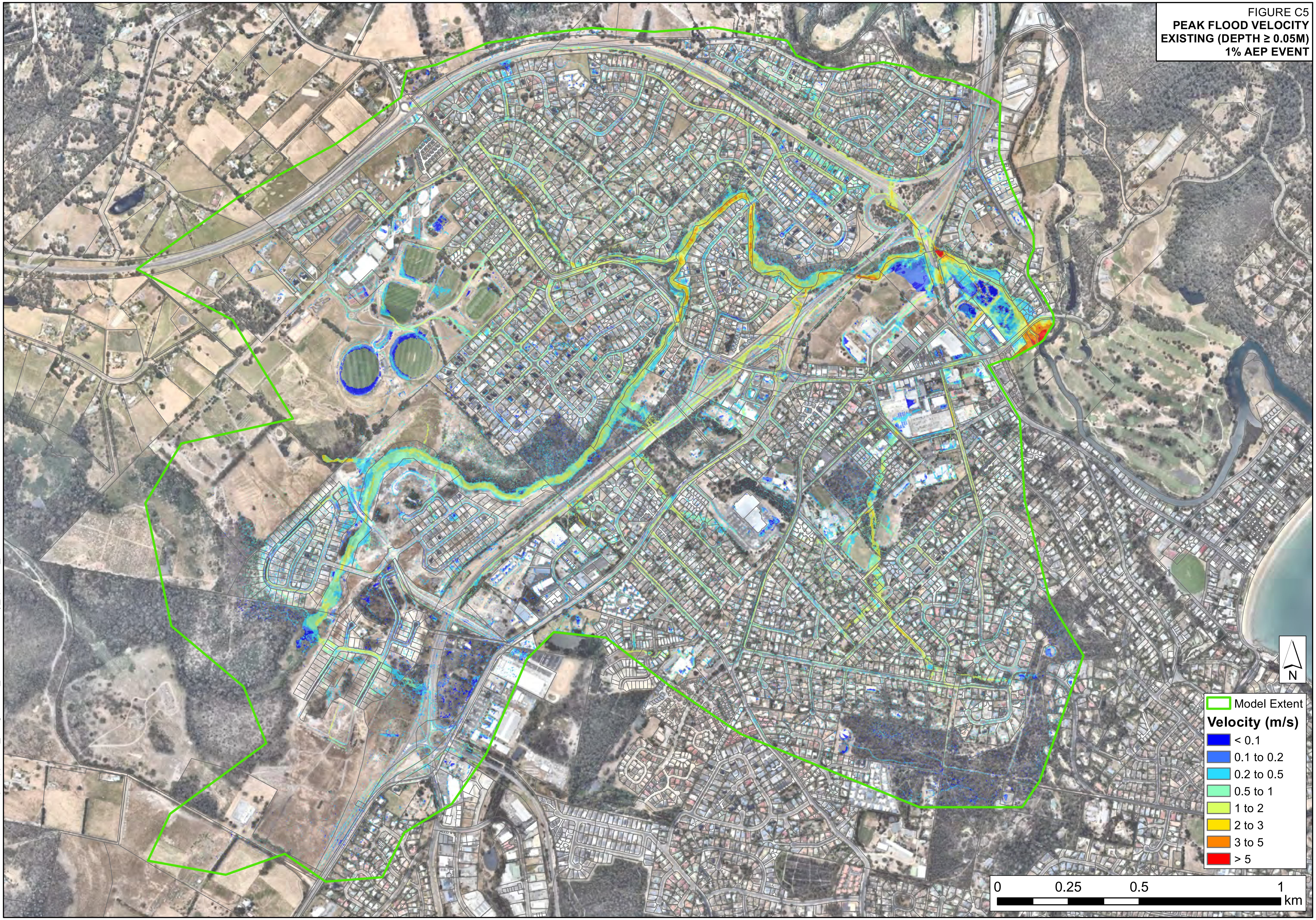
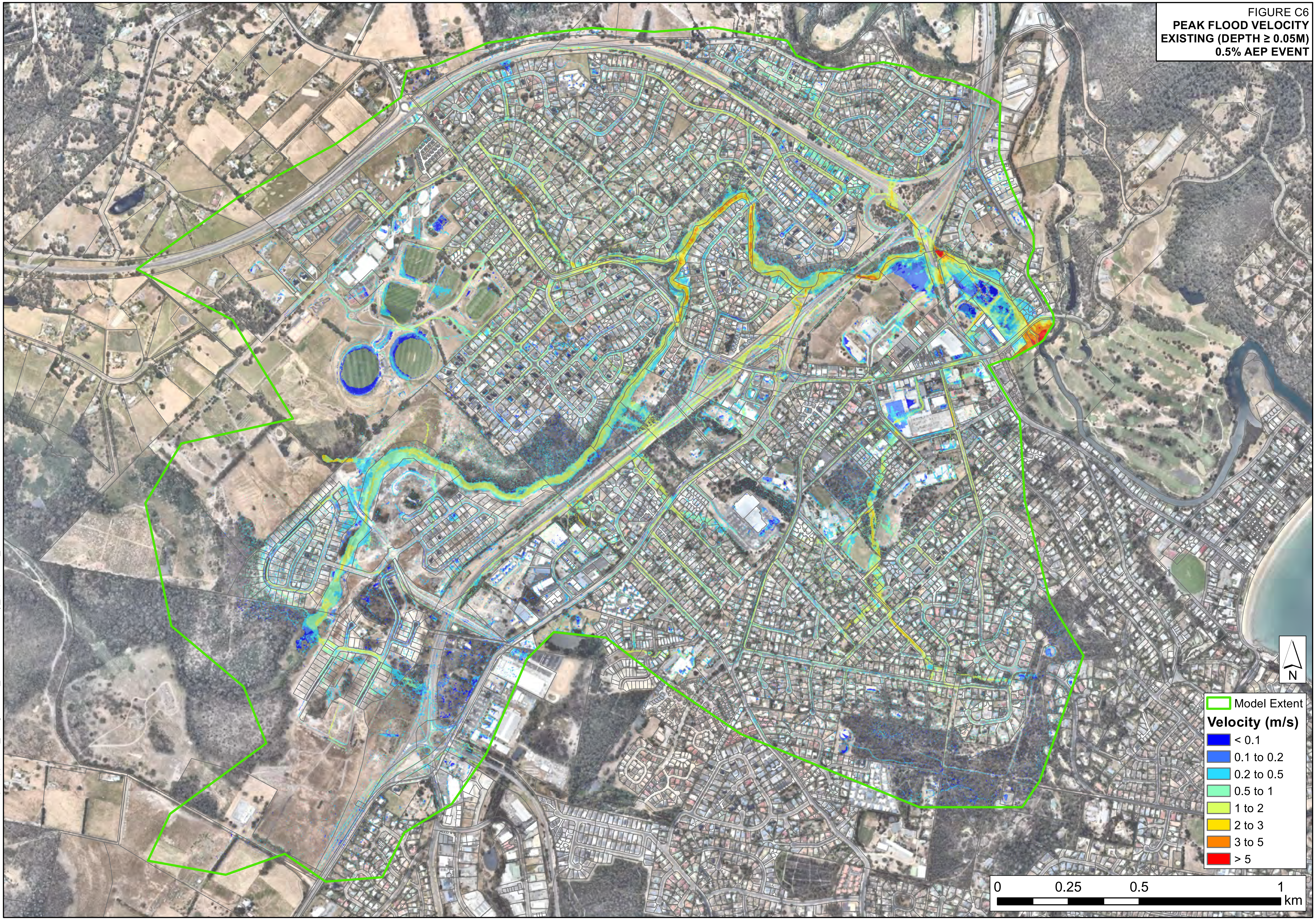


FIGURE C4
PEAK FLOOD VELOCITY
EXISTING (DEPTH ≥ 0.05M)
5% AEP EVENT
Model Extent
Velocity (m/s)
< 0.1
0.1 to 0.2
0.2 to 0.5
0.5 to 1
1 to 2
2 to 3
3 to 5
> 5
0
0.25
0.5
1
km
N
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC04_PeakFloodVelocity_5pctAEP_Event.mxd



J:Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC05_PeakFloodVelocity_1pctAEP_Event.mxd



J:Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC06_PeakFloodVelocity_0.5pctAEP_Event.mxd

FIGURE C7
PEAK FLOOD DEPTHS
EXISTING (DEPTH ≥ 0.05M)
5% AEP EVENT
Model Extent
Peak Flood Depth (m)
0.05 - 0.10
0.10 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
0
0.25
0.5
1
km
N
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC07_PeakFloodDepth_5pctAEP_Event.mxd

FIGURE C8
PEAK FLOOD DEPTHS
EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC08_PeakFloodDepth_1pctAEP_Event.mxd


FIGURE C9
PEAK FLOOD DEPTHS
EXISTING (DEPTH ≥ 0.05M)
0.5% AEP EVENT
N
Model Extent
Peak Flood Depth (m)
0.05 - 0.10
0.10 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
0
0.25
0.5
1
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC09_PeakFloodDepth_0.5pctAEP_Event.mxd

FIGURE C10
HYDRAULIC HAZARD
EXISTING (DEPTH ≥ 0.05M)
5% AEP EVENT

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC10_Hydraulic_Hazard_5pctAEP_Event.mxd


FIGURE C11
HYDRAULIC HAZARD
EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT
Model Extent
Hydraulic Hazard
H1 - No constraints
H2 - Unsafe for small vehicles
H3 - Unsafe for all vehicles, children and the elderly
H4 - Unsafe for all people and all vehicles
H5 - Unsafe for all people and all vehicles. Buildings require special engineering design and construction
H6 - Unconditionally dangerous
0
0.25
0.5
1
km
N
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC11_Hydraulic_Hazard_1pctAEP_Event.mxd

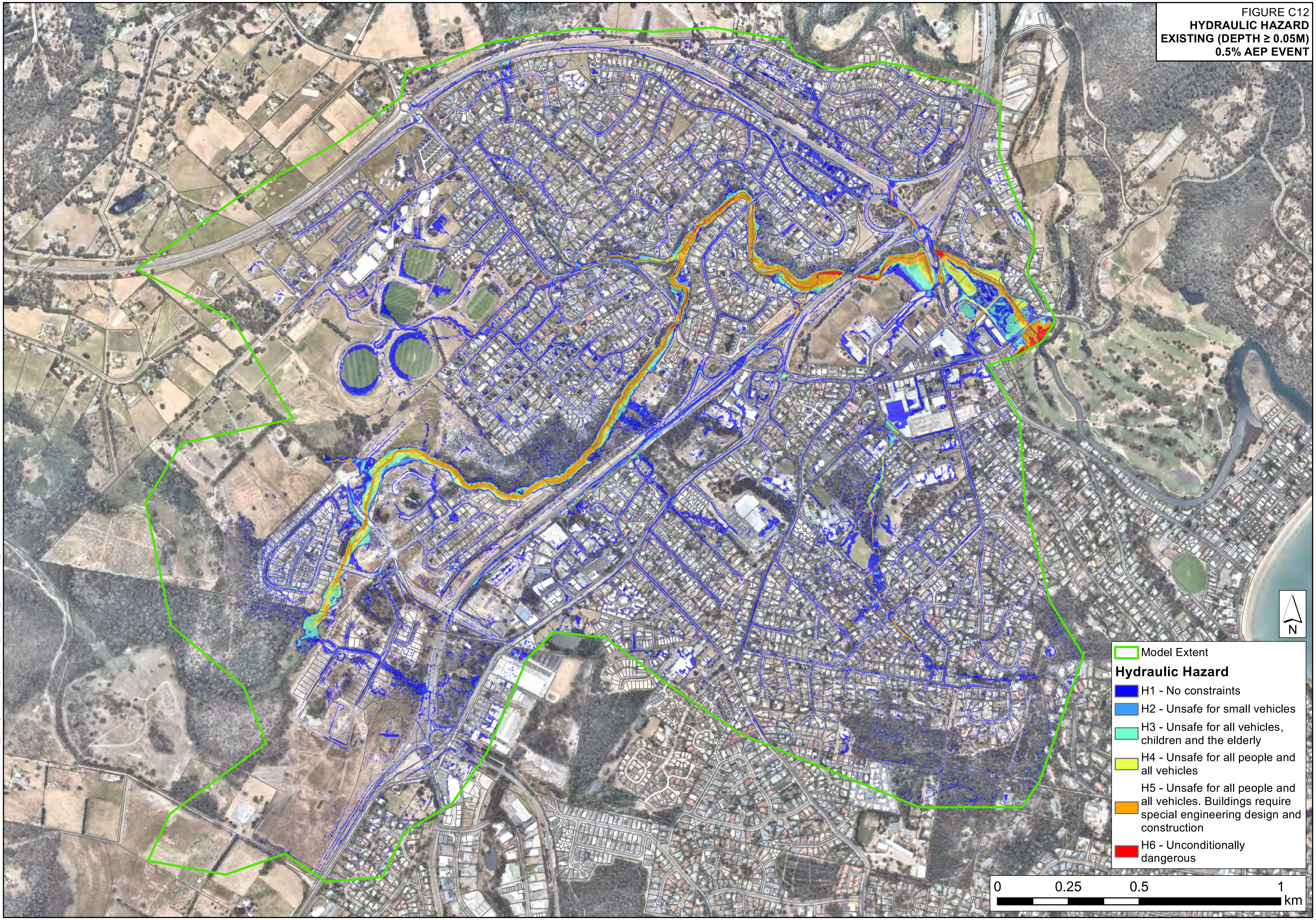


FIGURE C12
HYDRAULIC HAZARD
EXISTING (DEPTH ≥ 0.05M)
0.5% AEP EVENT

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC12_Hydraulic_Hazard_0.5pctAEP_Event.mxd

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC13_HydraulicCategories_5pctAEP_Event.mxd

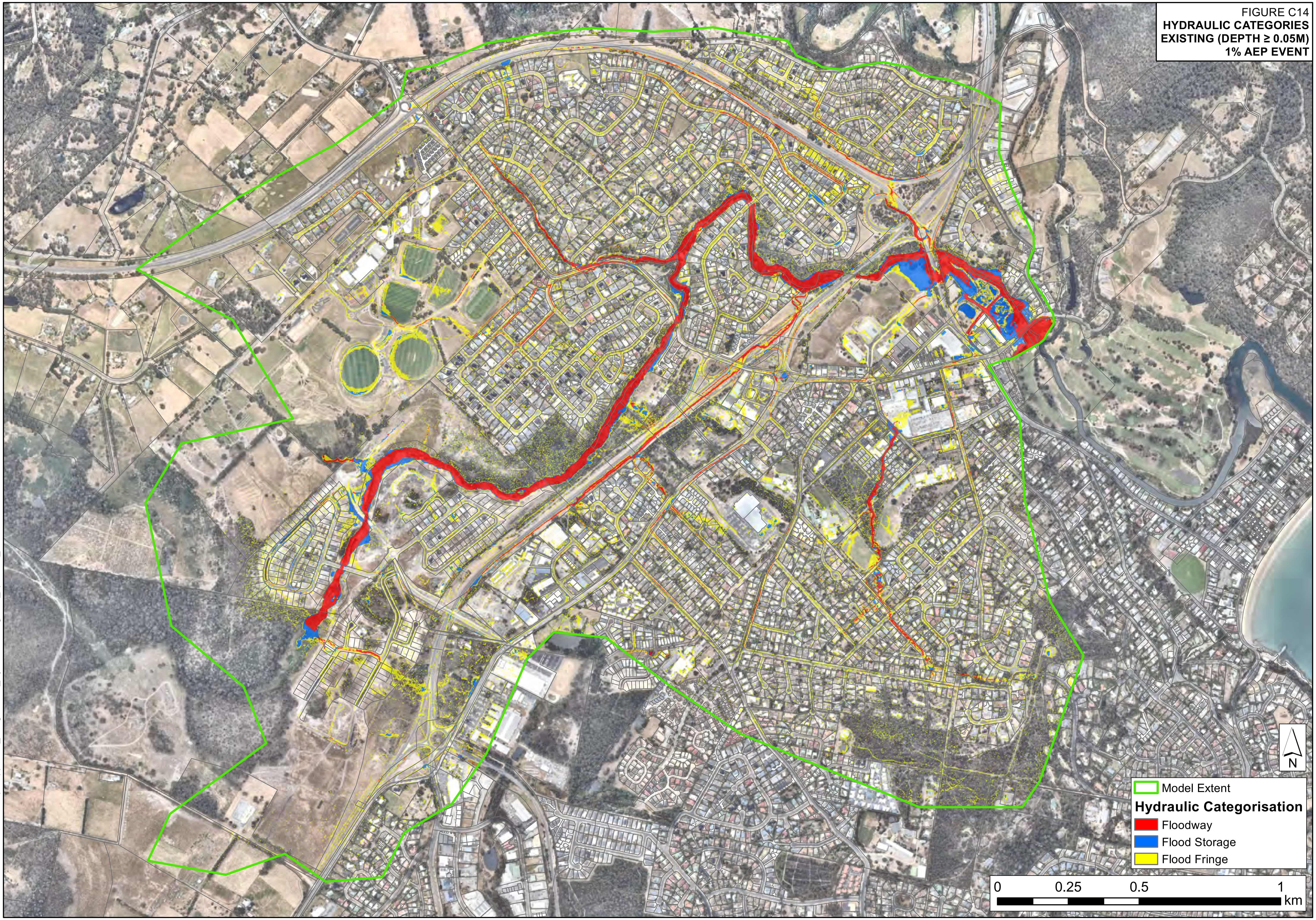


FIGURE C14
HYDRAULIC CATEGORIES
EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT
N
Model Extent
Hydraulic Categorisation
Floodway
Flood Storage
Flood Fringe
0
0.25
0.5
1
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC14_HydraulicCategories_1pctAEP_Event.mxd

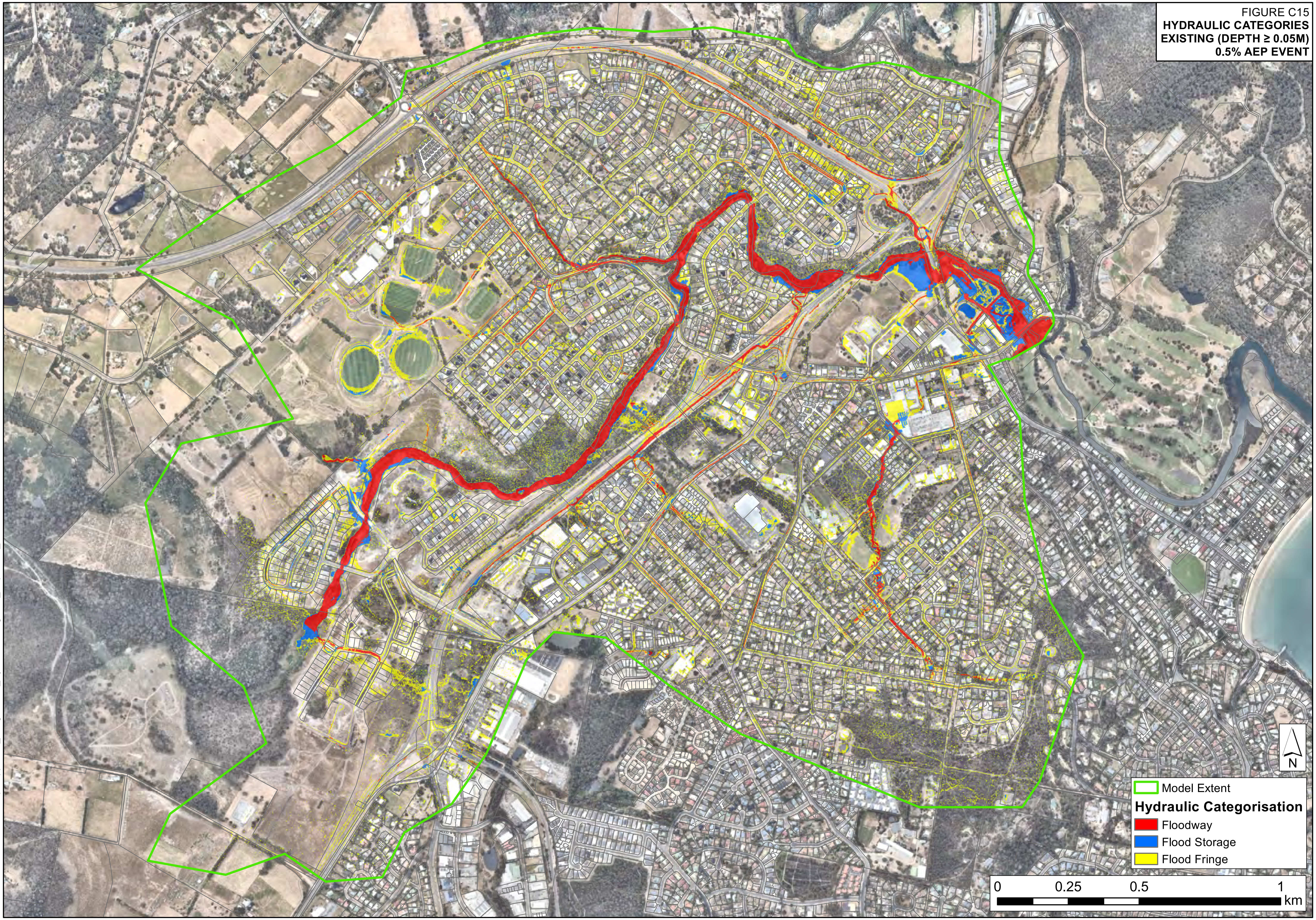
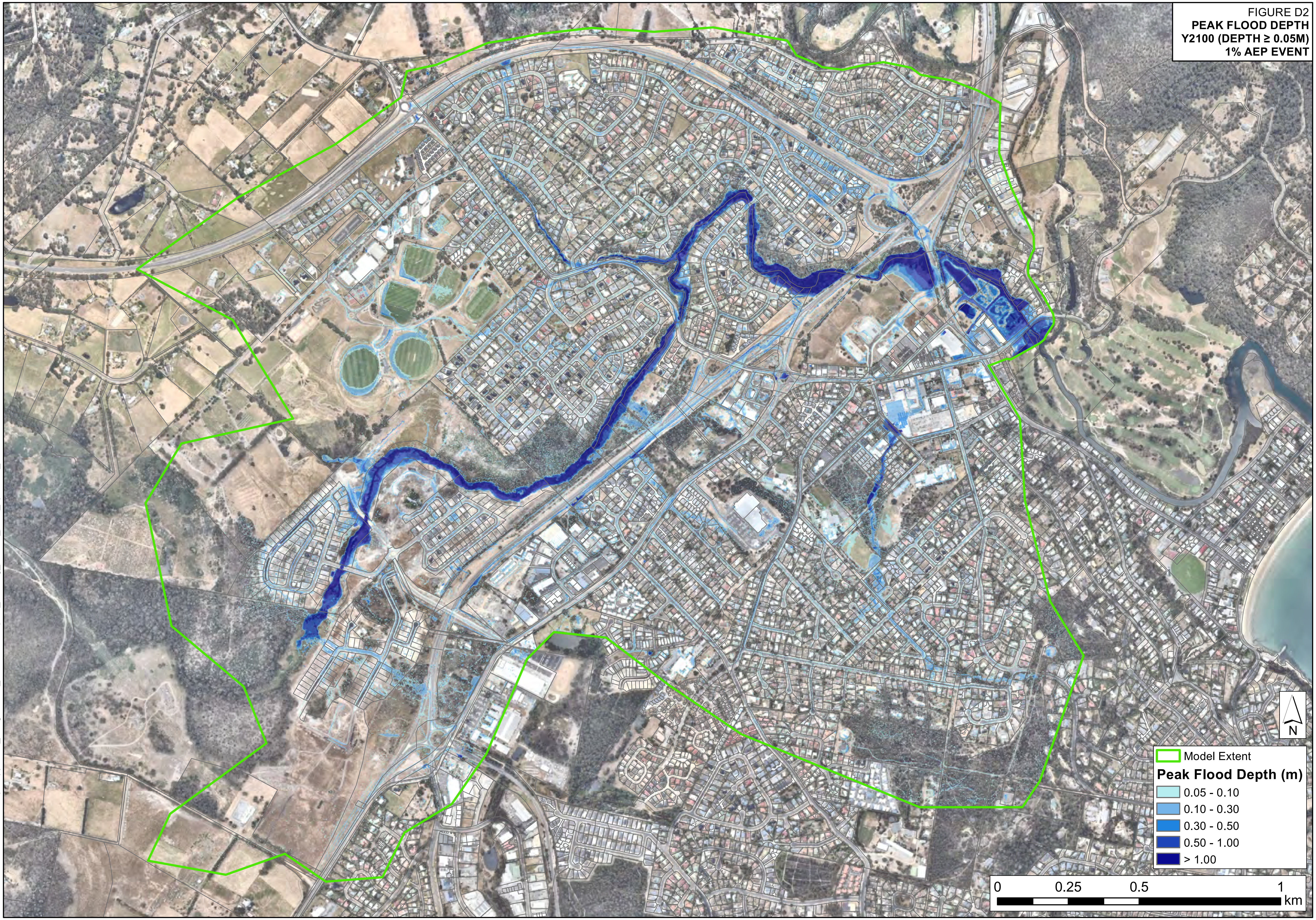


FIGURE C15
HYDRAULIC CATEGORIES
EXISTING (DEPTH ≥ 0.05M)
0.5% AEP EVENT
Model Extent
Hydraulic Categorisation
Floodway
Flood Storage
Flood Fringe
0
0.25
0.5
1
km
N
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureC15_HydraulicCategories_0.5pctAEP_Event.mxd

APPENDIX D. Sensitivity Flood Mapping

Appendix D


FIGURE D1
PEAK FLOOD DEPTH
Y2050 (DEPTH ≥ 0.05M)
1% AEP EVENT
N
Model Extent
Peak Flood Depth (m)
0.05 - 0.10
0.10 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
0
0.25
0.5
1
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureD01_PeakFloodDepths_Existing_climate_2050_1pctAEP_Event.mxd



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureD02_PeakFloodDepths_Existing_climate_2100_1pctAEP_Event.mxd

FIGURE D3
PEAK FLOOD DEPTH
20% ROUGHNESS DECREASE (DEPTH ≥ 0.05M)
1% AEP EVENT

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureD03_PeakFloodLeveldepth_1pctAEP_Event.mxd


FIGURE D4
PEAK FLOOD DEPTH
20% ROUGHNESS INCREASE (DEPTH ≥ 0.05M)
1% AEP EVENT
Model Extent
Peak Flood Depth (m)
0.05 - 0.10
0.10 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
0
0.25
0.5
1
km
N
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureD04_PeakFloodLeveldepth_1pctAEP_Event.mxd

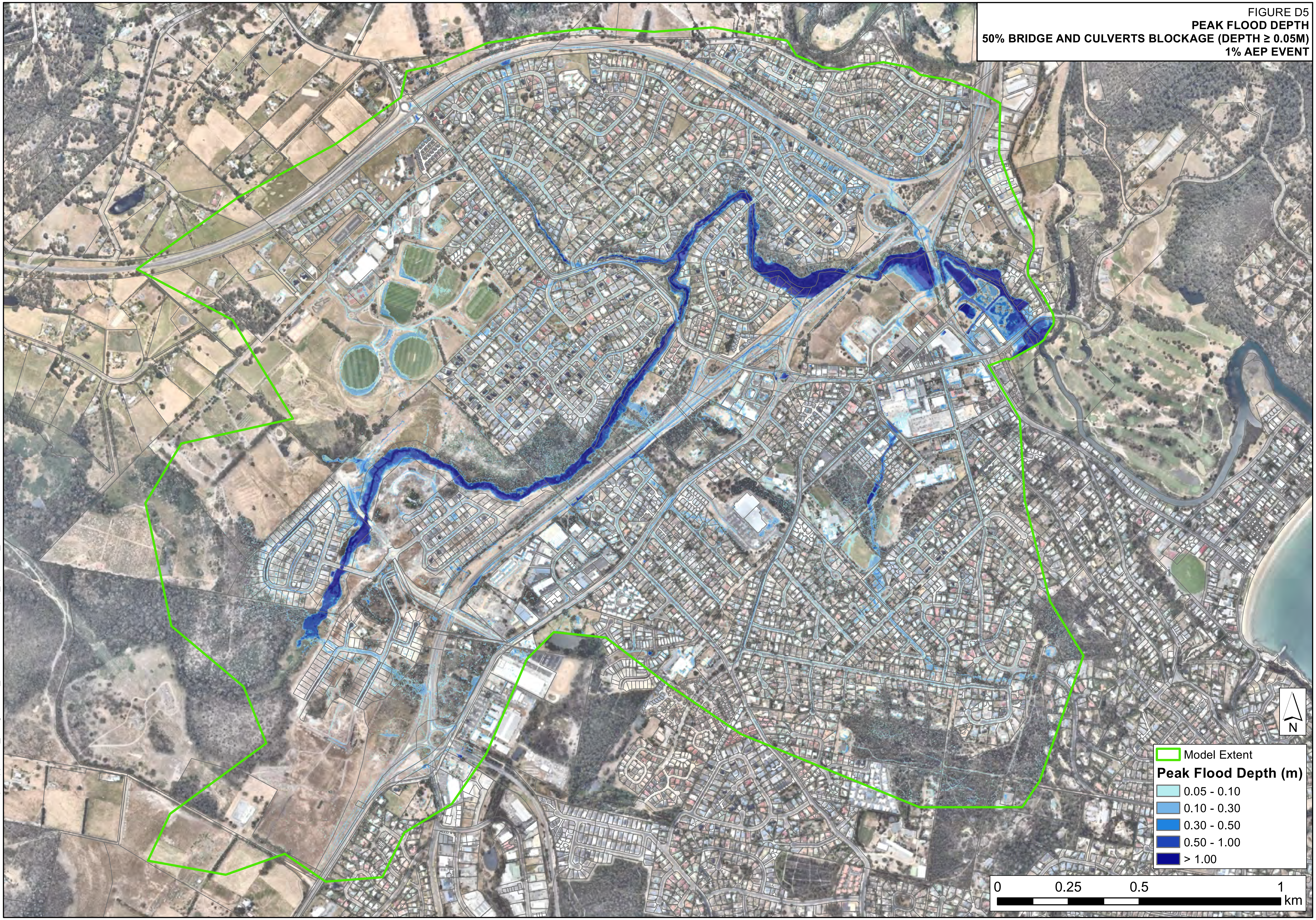
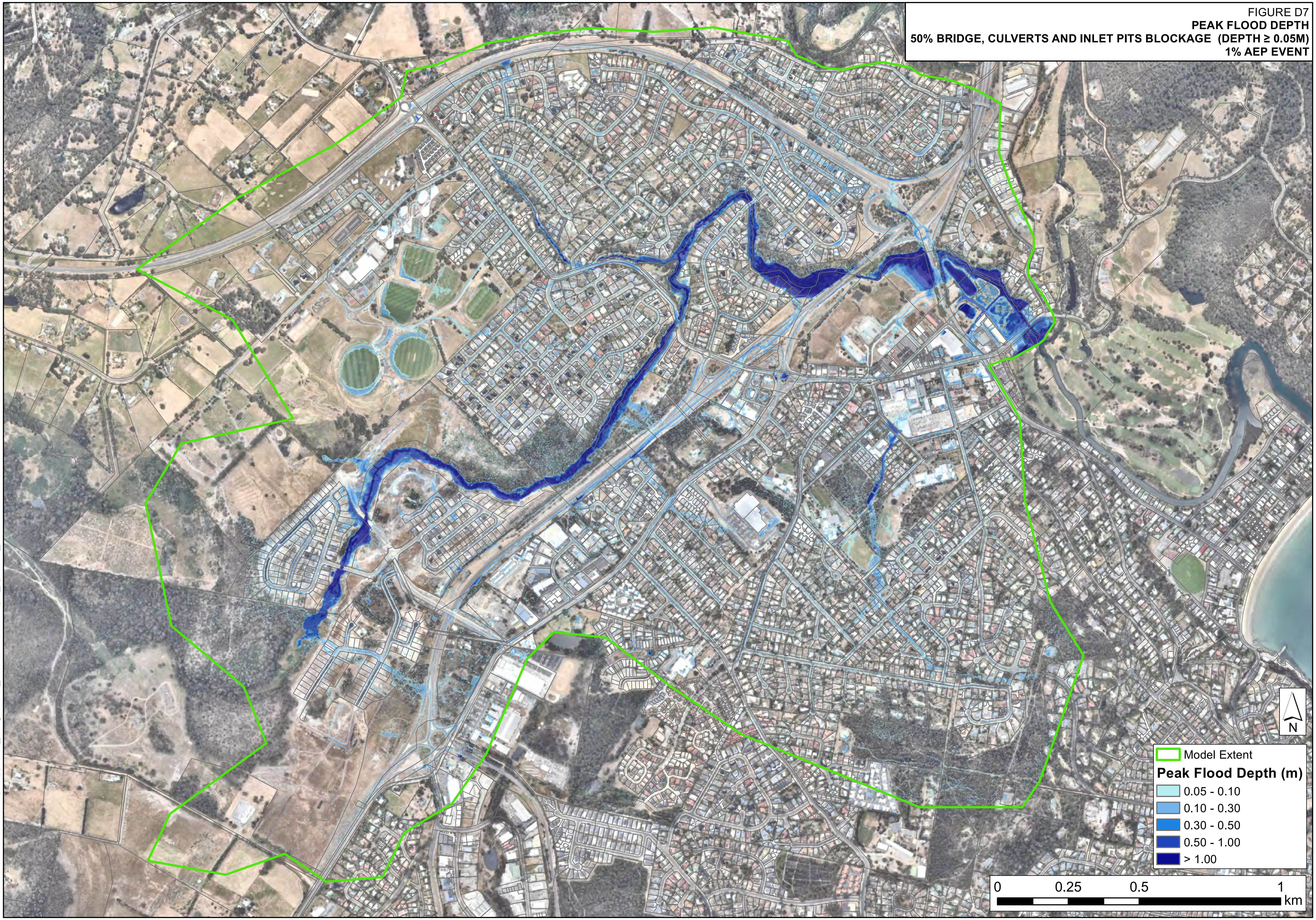


FIGURE D5
PEAK FLOOD DEPTH
50% BRIDGE AND CULVERTS BLOCKAGE (DEPTH ≥ 0.05M)
1% AEP EVENT
N
Model Extent
Peak Flood Depth (m)
0.05 - 0.10
0.10 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
0
0.25
0.5
1
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureD05_PeakFloodLeveldepth_1pctAEP_Event.mxd

FIGURE D6
PEAK FLOOD DEPTH
50% INLET PITS BLOCKAGE (DEPTH ≥ 0.05M)
1% AEP EVENT

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureD06_PeakFloodLeveldepth_1pctAEP_Event.mxd

FIGURE D7
PEAK FLOOD DEPTH
50% BRIDGE, CULVERTS AND INLET PITS BLOCKAGE (DEPTH ≥ 0.05M)
1% AEP EVENT



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_D\FigureD07_PeakFloodLeveldepth_1pctAEP_Event.mxd

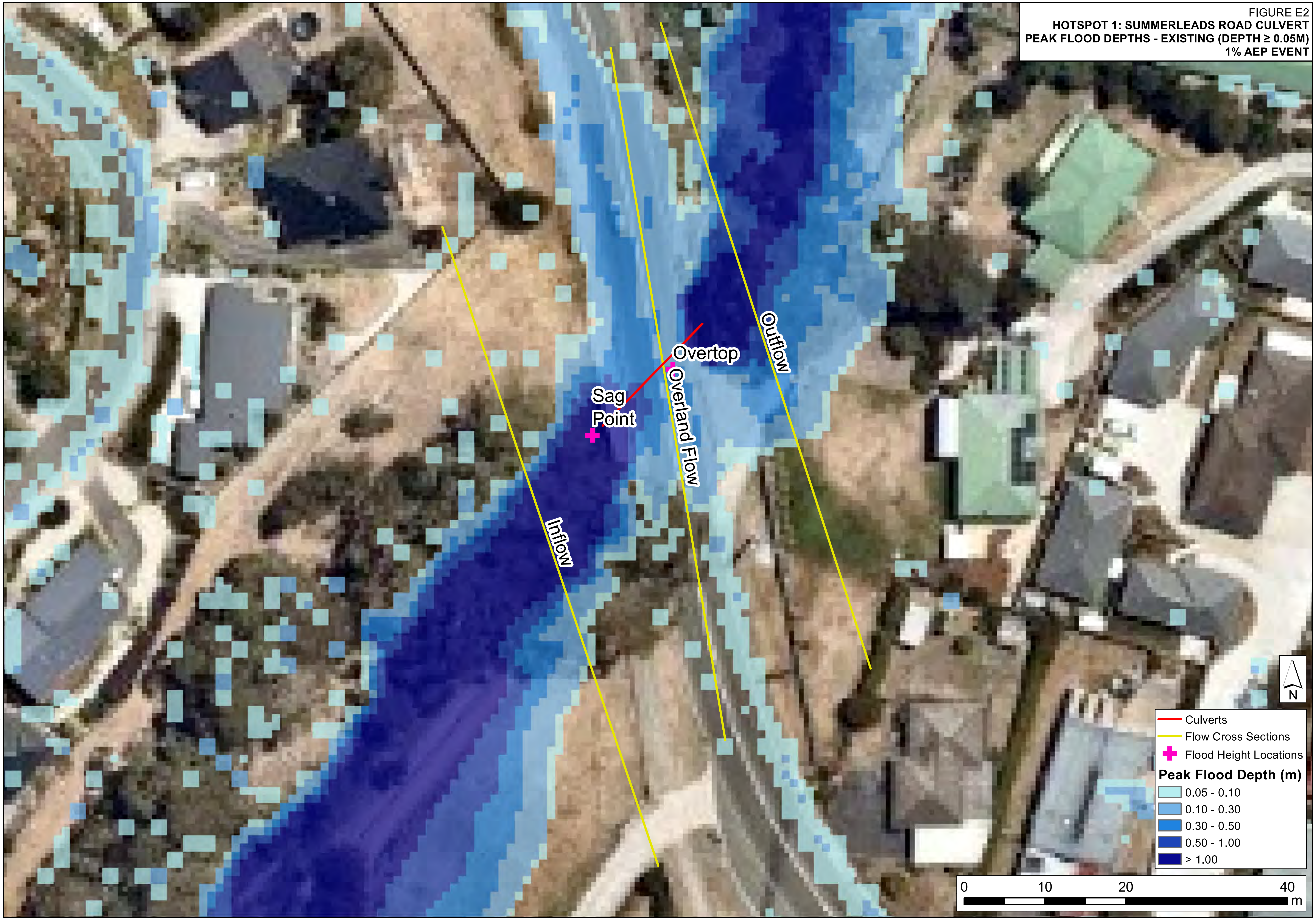
APPENDIX E. Hot Spot Mapping

Appendix E

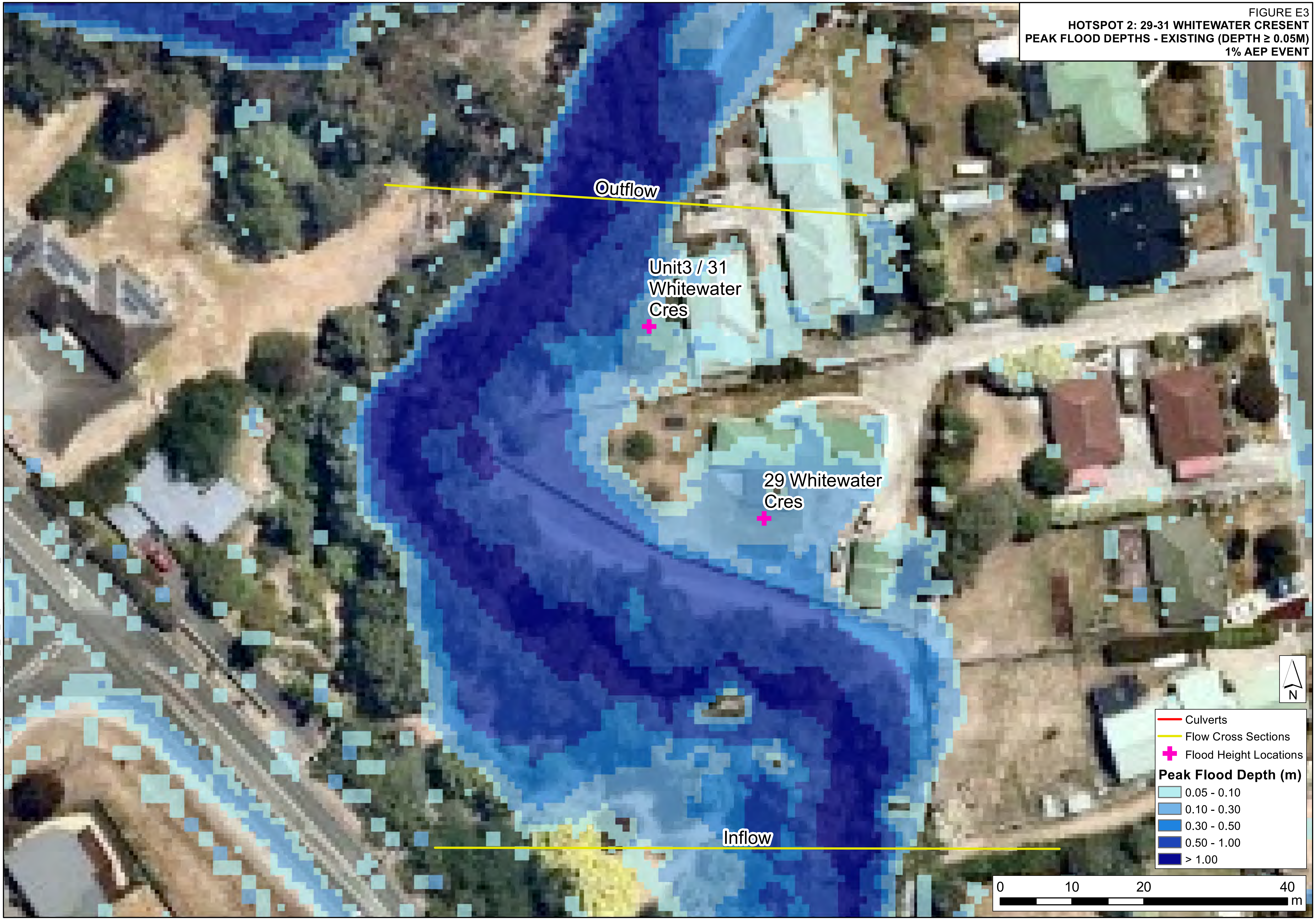
FIGURE E1
HOTSPOT LOCATIONS
PEAK FLOOD DEPTHS - EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE1_Hotspot_Locations.mxd

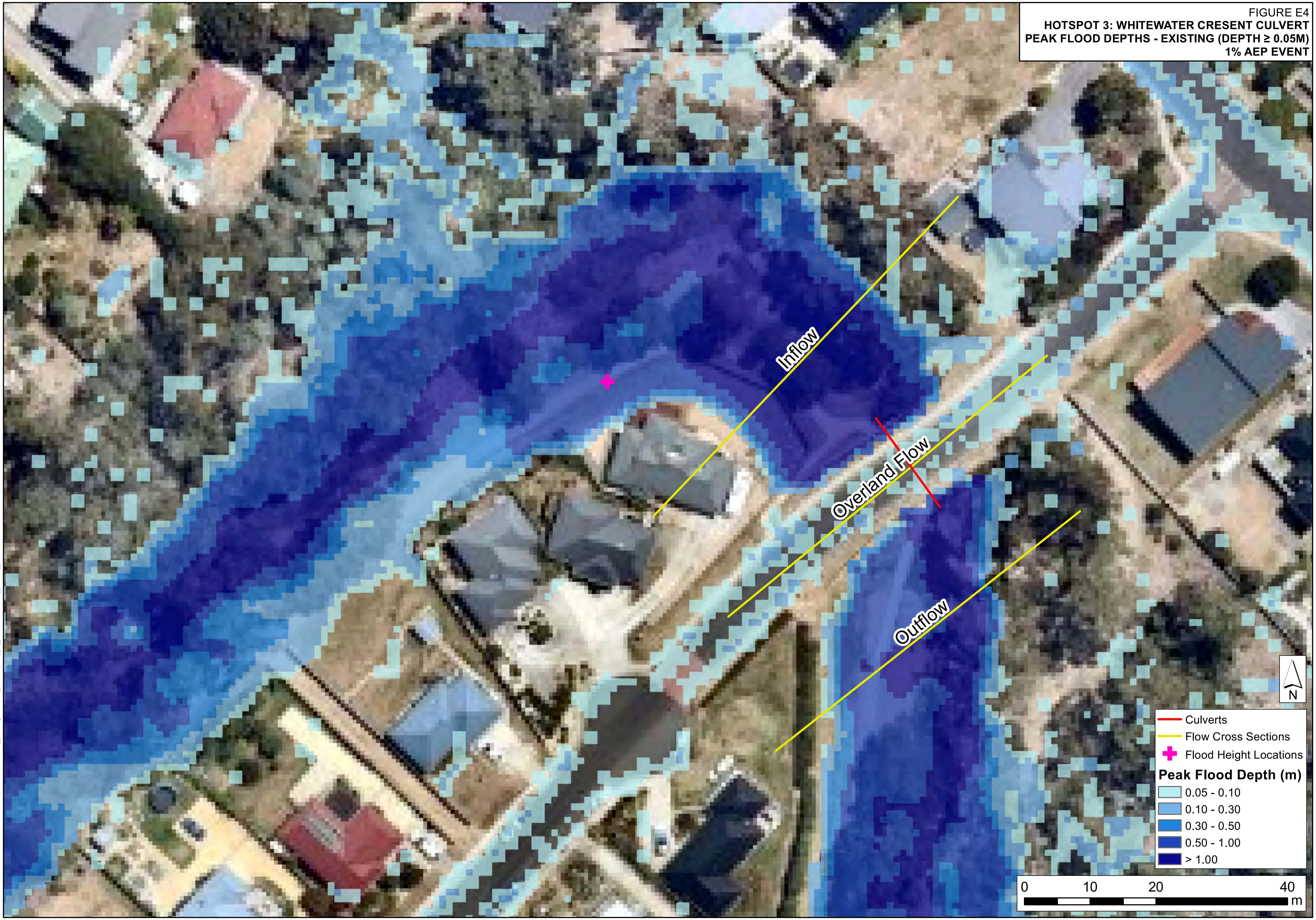
FIGURE E2
HOTSPOT 1: SUMMERLEADS ROAD CULVERT
PEAK FLOOD DEPTHS - EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT



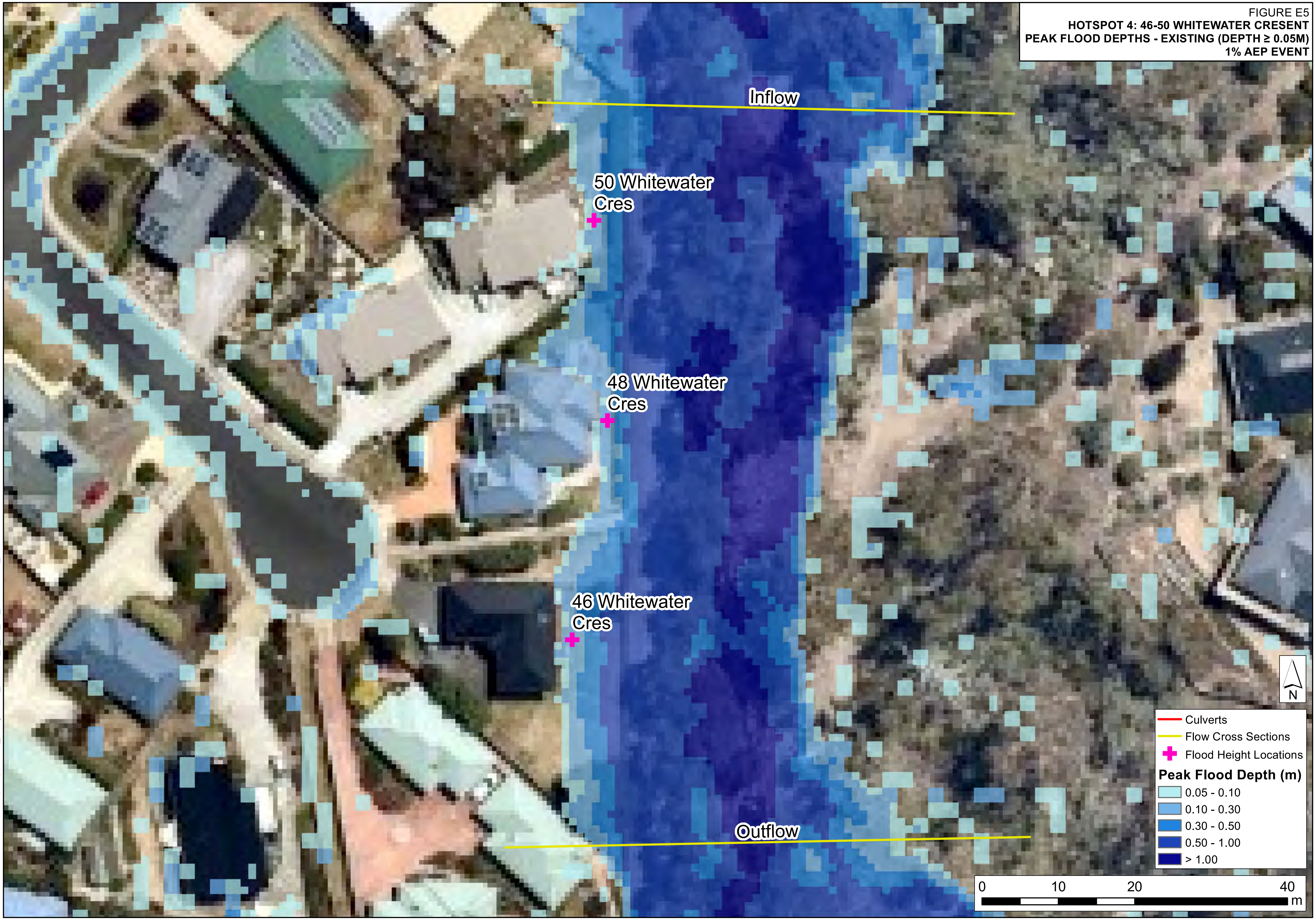
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE2_Hotspot_1_Summerleas_Rd_Culvert.mxd



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE3_Hotspot_2_29_31_Whitewater_Cres.mxd

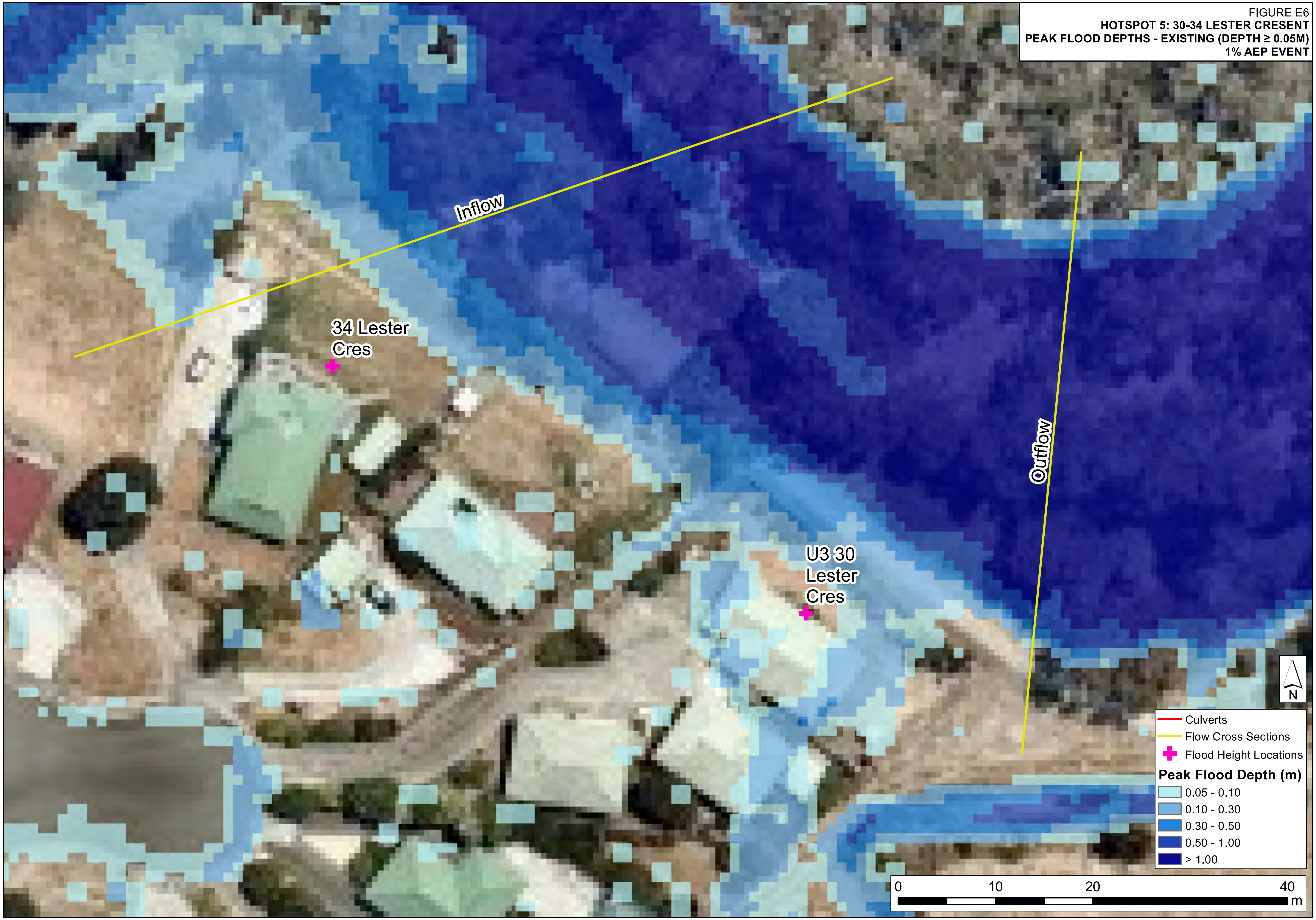


J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE4_Hotspot_3_Whitewater_Cres_Culvert.mxd



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE5_Hotspot_4_46_50_Whitewater_Cres.mxd

FIGURE E6
HOTSPOT 5: 30-34 LESTER CRESENT
PEAK FLOOD DEPTHS - EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE6_Hotspot_5_30_34_Lester_Cres.mxd

J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE7_Hotspot_6_Southern_Outlet_Culvert.mxd

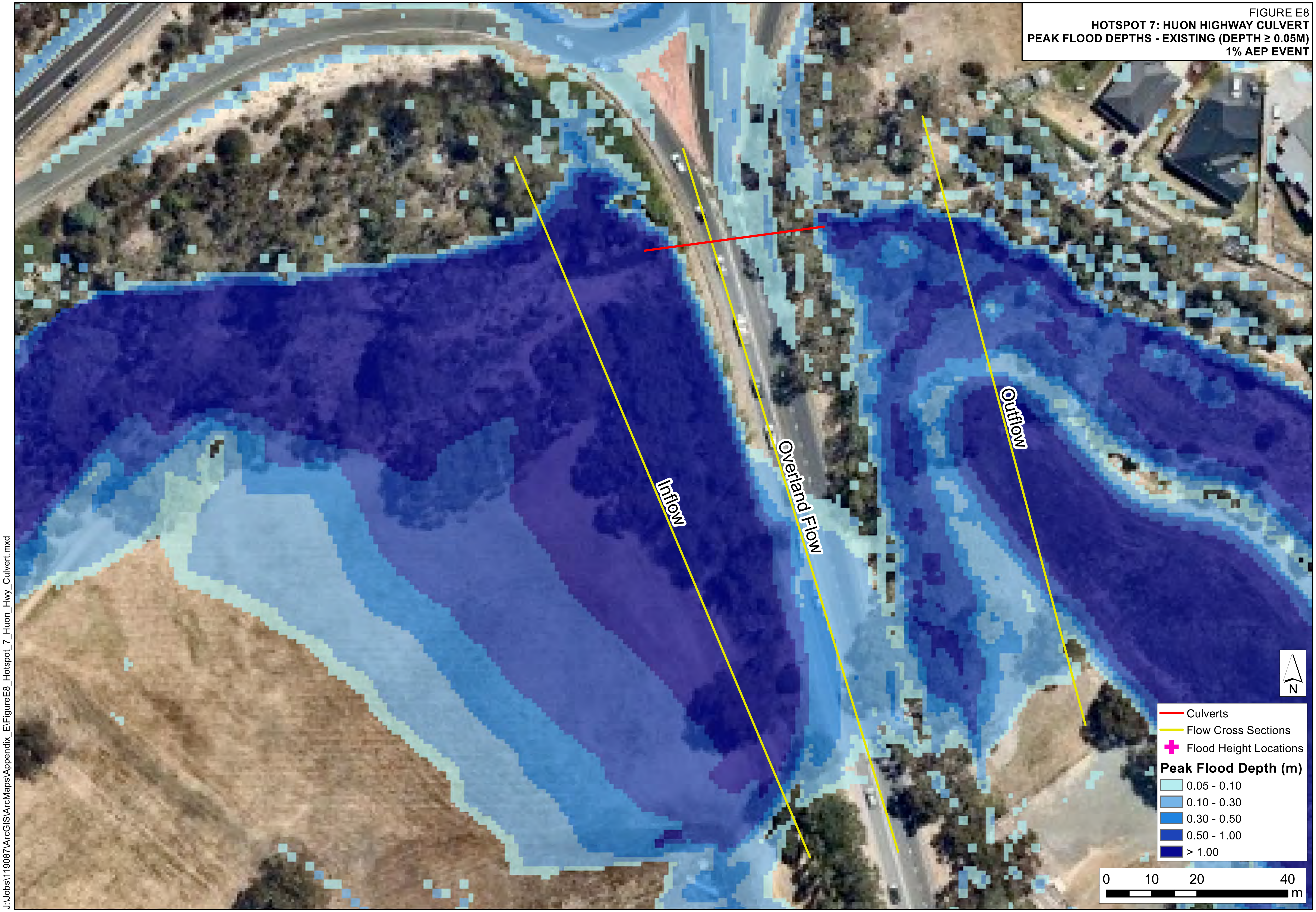
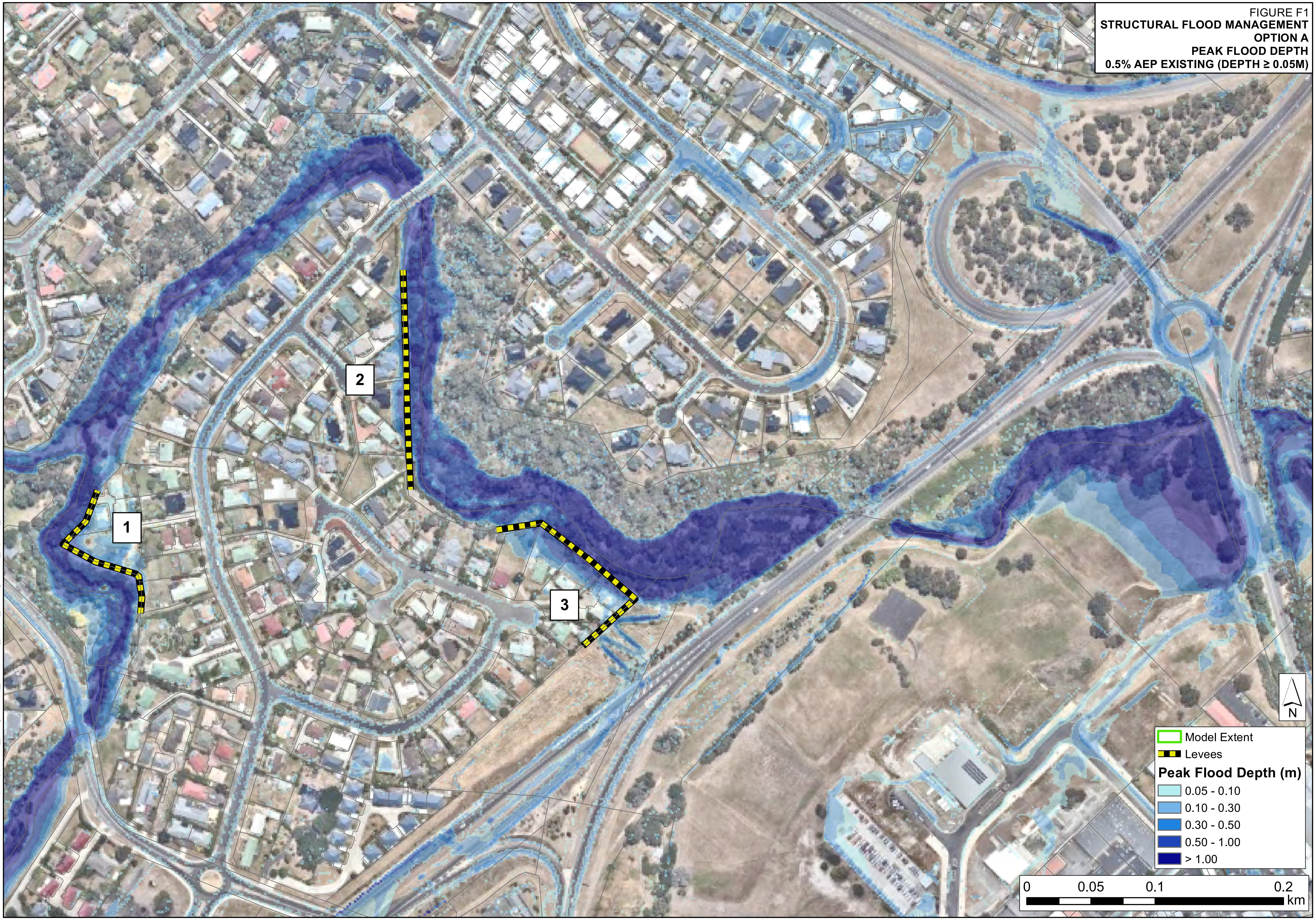


FIGURE E8
HOTSPOT 7: HUON HIGHWAY CULVERT
PEAK FLOOD DEPTHS - EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT
Inflow
Overland Flow
Outflow
N
Culverts
Flow Cross Sections
Flood Height Locations
Peak Flood Depth (m)
0.05 - 0.10
0.10 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
0 10 20 40
m
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_E\FigureE8_Hotspot_7_Huon_Hwy_Culvert.mxd

APPENDIX F. Flood Mitigation Mapping

Appendix F



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF01_Structural_Flood_Management_Option_A.mxd

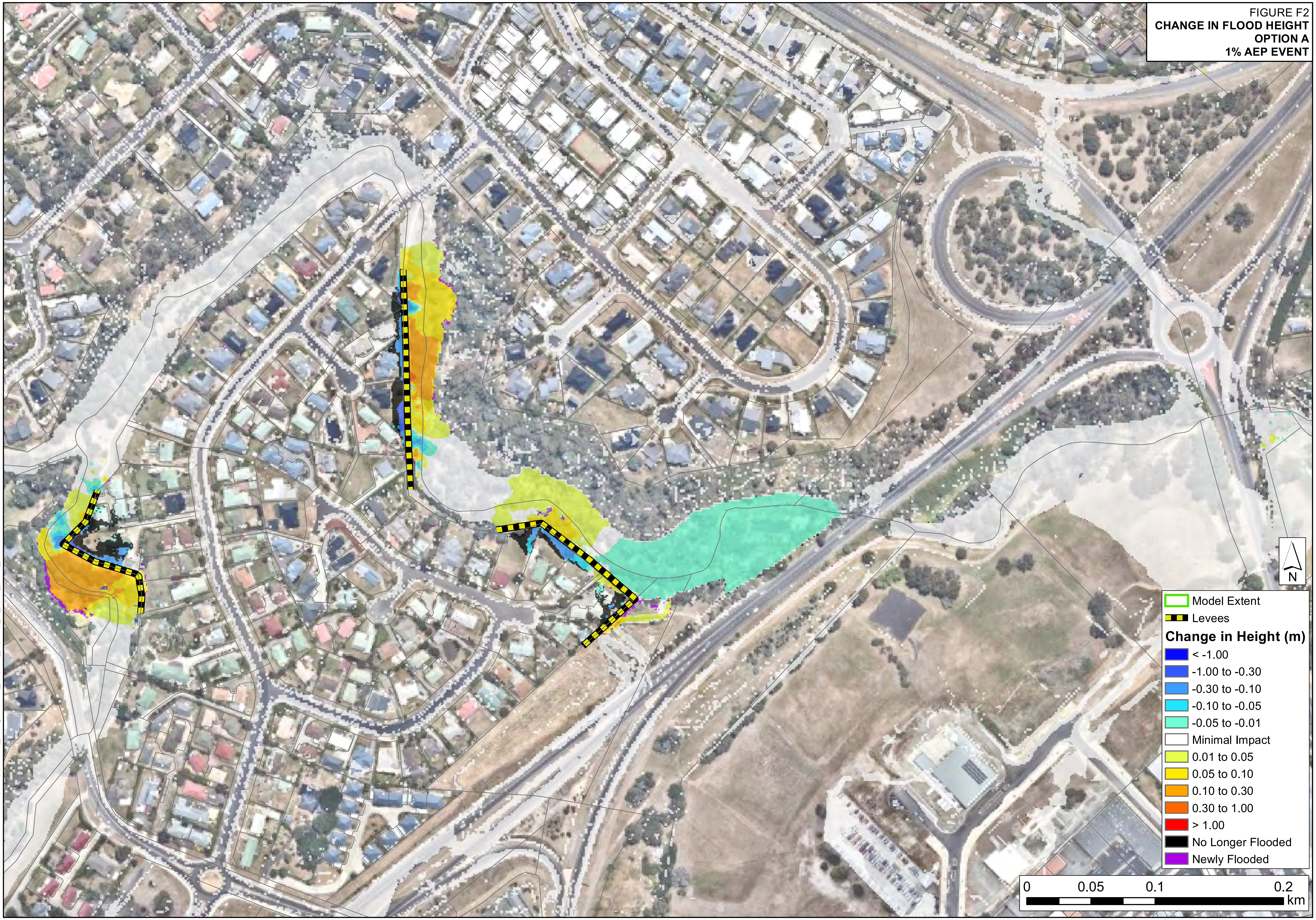
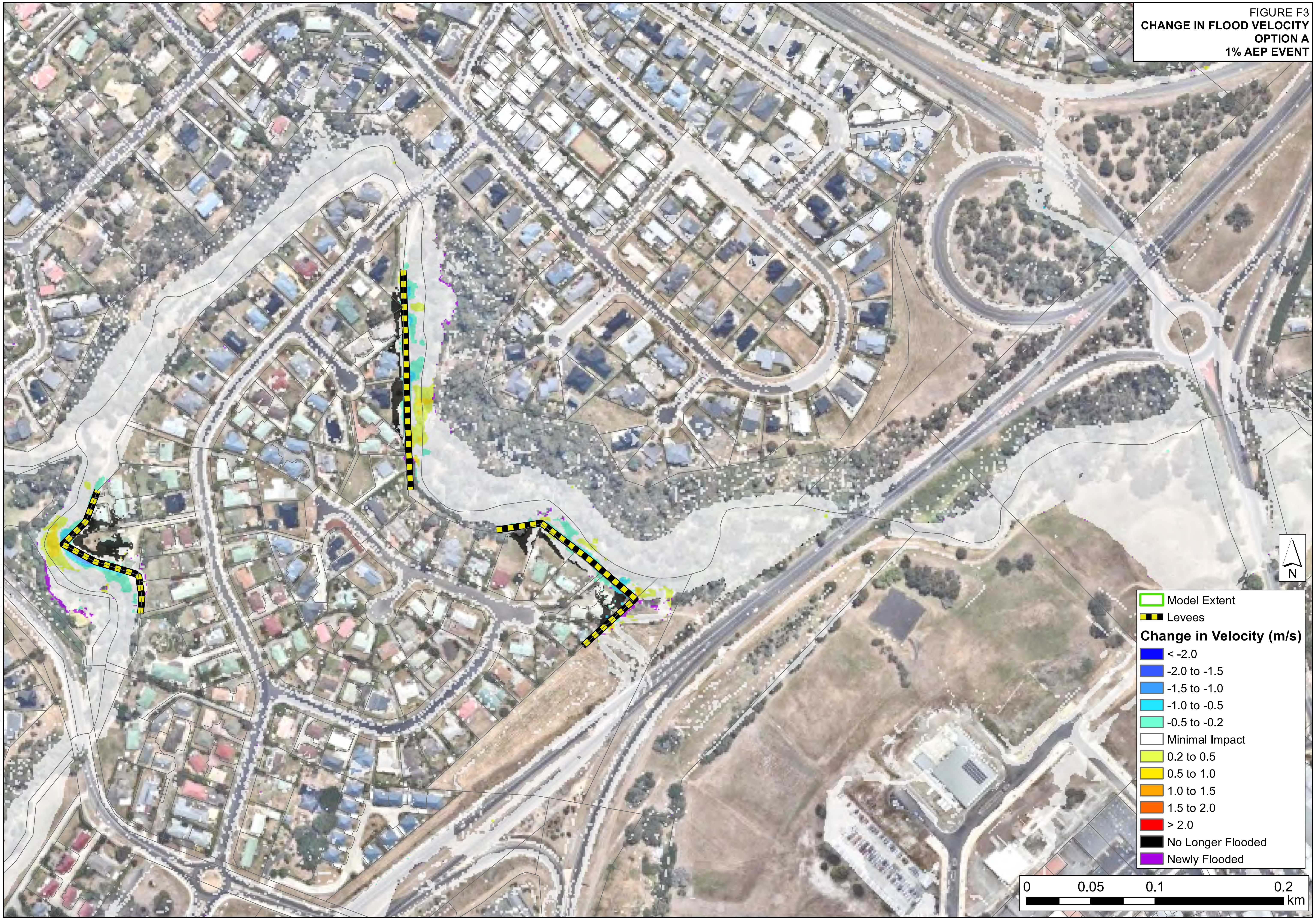


FIGURE F2
CHANGE IN FLOOD HEIGHT
OPTION A
1% AEP EVENT
Model Extent
Levees
Change in Height (m)
< -1.00
-1.00 to -0.30
-0.30 to -0.10
-0.10 to -0.05
-0.05 to -0.01
Minimal Impact
0.01 to 0.05
0.05 to 0.10
0.10 to 0.30
0.30 to 1.00
> 1.00
No Longer Flooded
Newly Flooded
N
0 0.05 0.1 0.2
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF02_Impact_Option_A_1pct_AEP_Event.mxd



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF03_Impact_Velocity_Option_A_1pct_AEP_Event.mxd

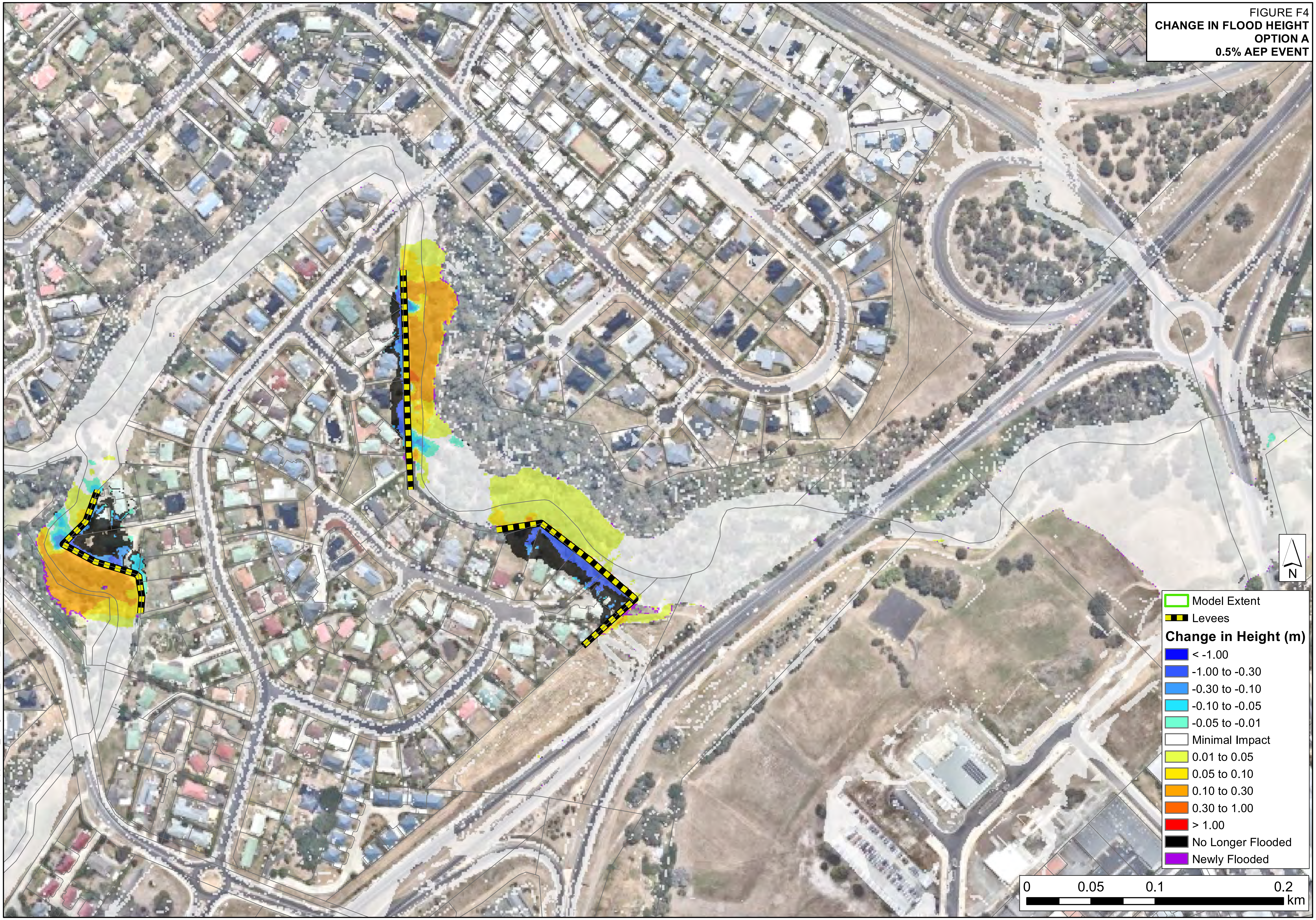


FIGURE F4
CHANGE IN FLOOD HEIGHT
OPTION A
0.5% AEP EVENT
N
Model Extent
Levees
Change in Height (m)
< -1.00
-1.00 to -0.30
-0.30 to -0.10
-0.10 to -0.05
-0.05 to -0.01
Minimal Impact
0.01 to 0.05
0.05 to 0.10
0.10 to 0.30
0.30 to 1.00
> 1.00
No Longer Flooded
Newly Flooded
0 0.05 0.1 0.2
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF04_Impact_Option_A_0.5pct_AEP_Event.mxd

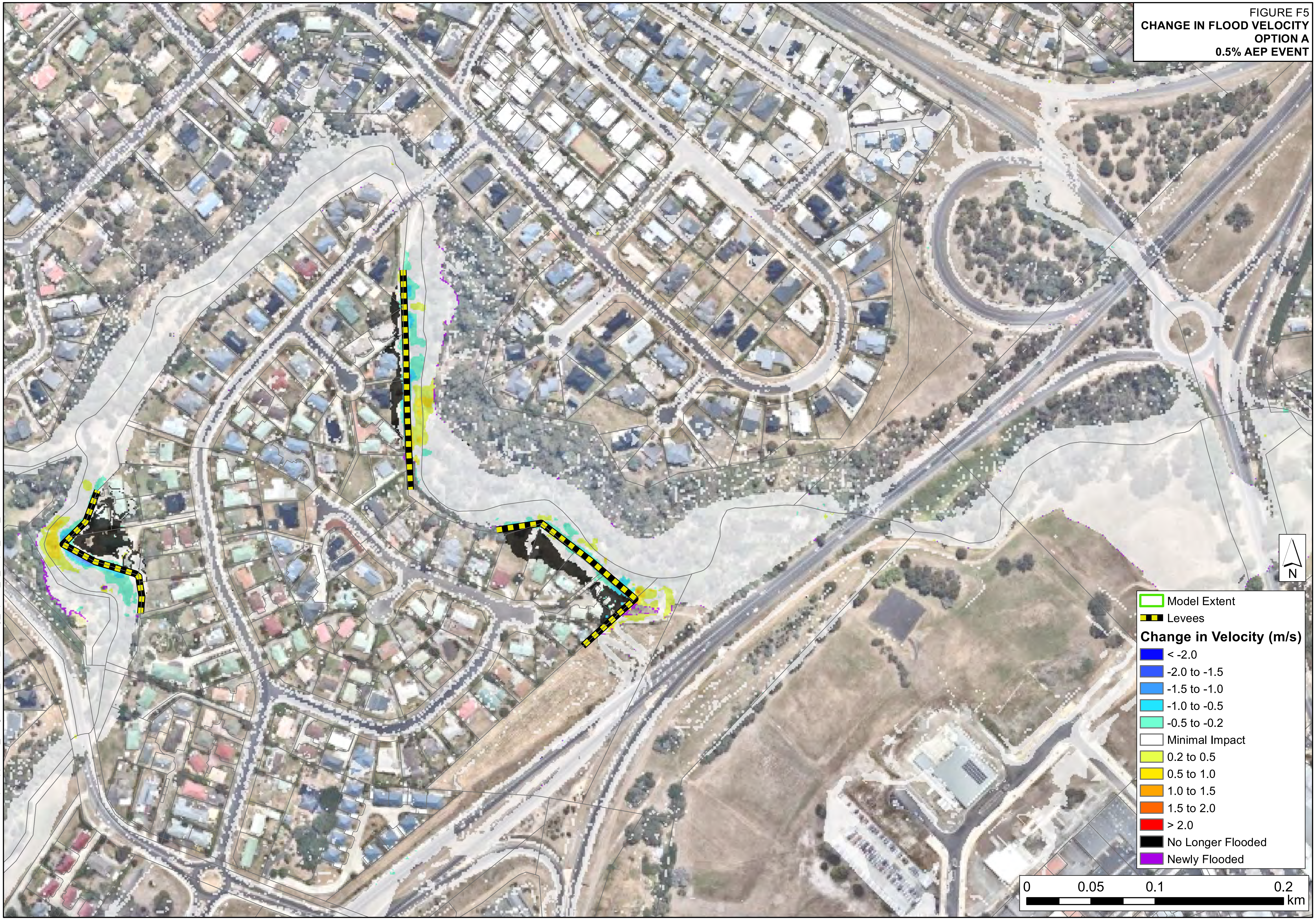
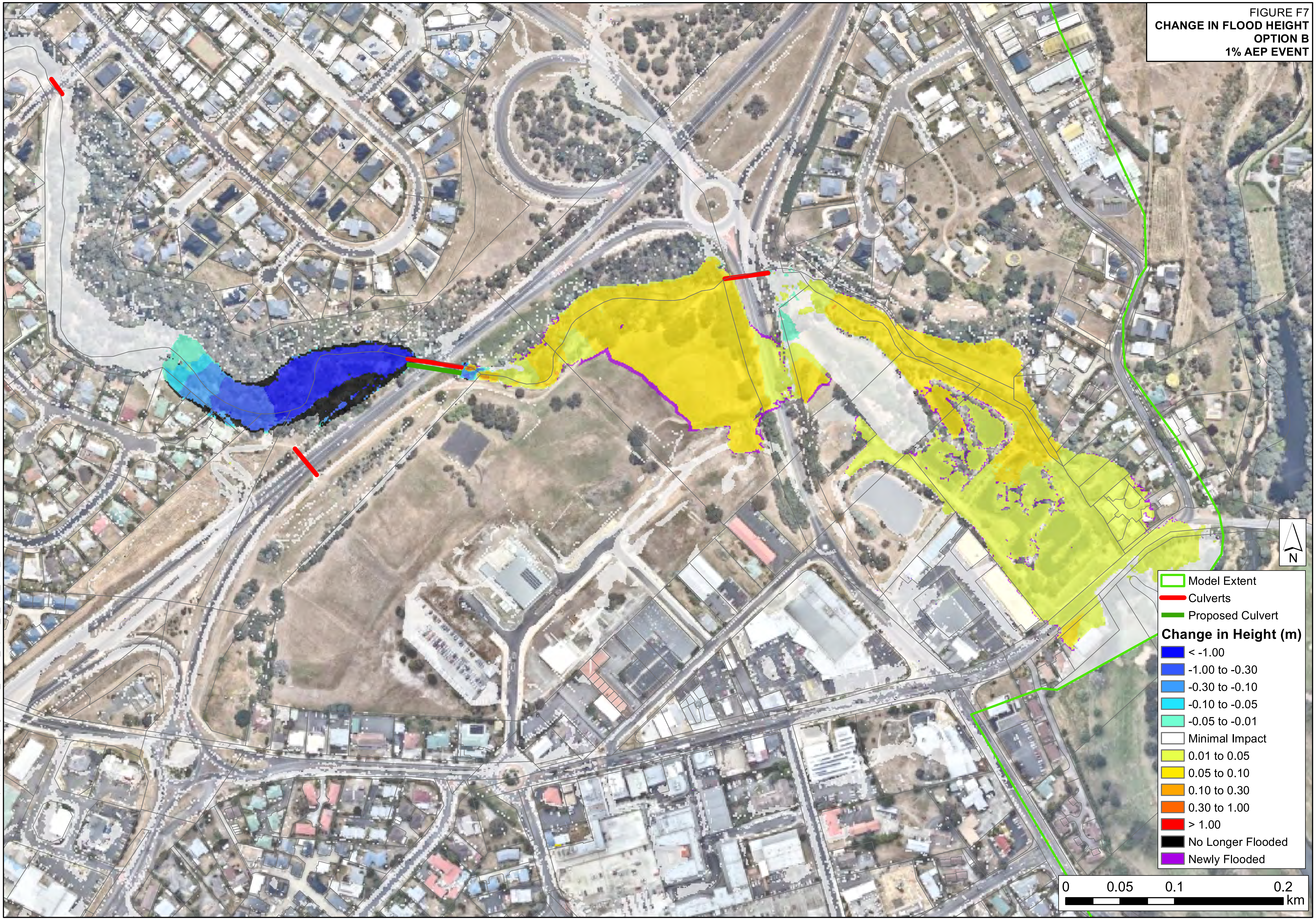


FIGURE F5
CHANGE IN FLOOD VELOCITY
OPTION A
0.5% AEP EVENT
Model Extent
Levees
Change in Velocity (m/s)
< -2.0
-2.0 to -1.5
-1.5 to -1.0
-1.0 to -0.5
-0.5 to -0.2
Minimal Impact
0.2 to 0.5
0.5 to 1.0
1.0 to 1.5
1.5 to 2.0
> 2.0
No Longer Flooded
Newly Flooded
0 0.05 0.1 0.2
km
N
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF05_Impact_Velocity_Option_A_0.5pct_AEP_Event.mxd

FIGURE F6
STRUCTURAL FLOOD MANAGEMENT OPTION B PEAK FLOOD DEPTH 0.5% AEP EXISTING (DEPTH ≥ 0.05M)

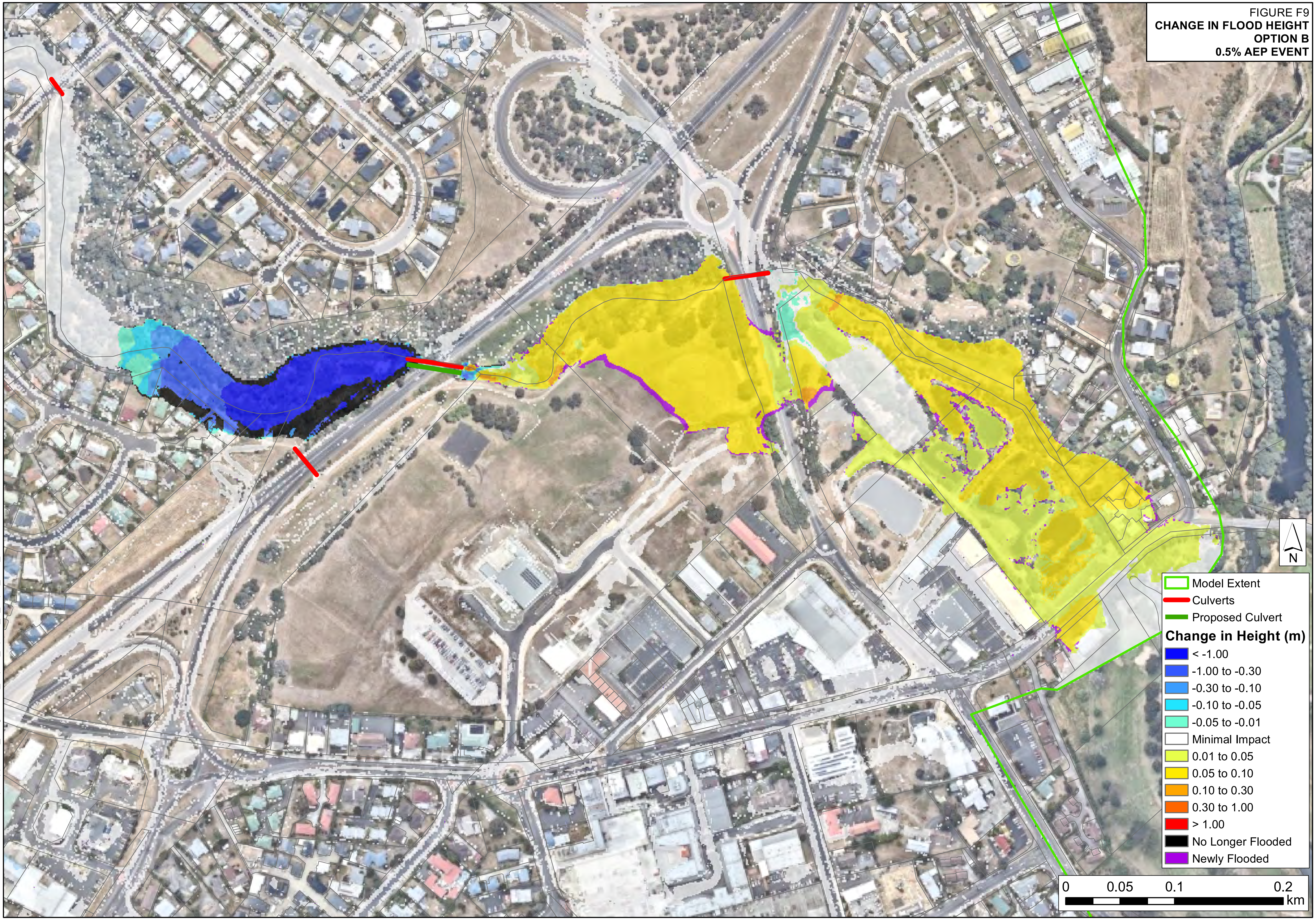
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF06_Structural_Flood_Management_Option_B.mxd

FIGURE F7
CHANGE IN FLOOD HEIGHT
OPTION B
1% AEP EVENT

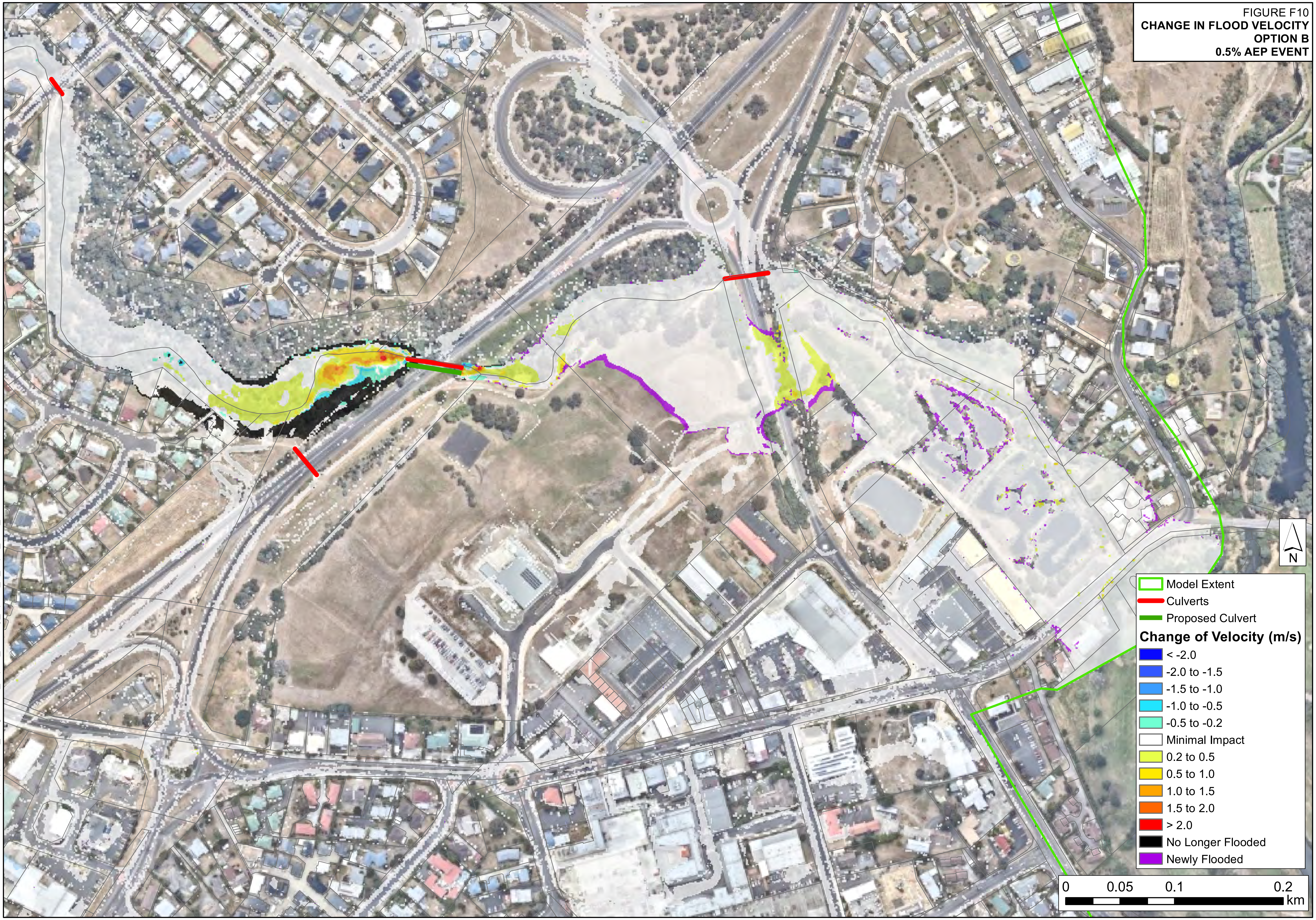


J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF07_Impact_Option_B_1pct_AEP_Event.mxd


FIGURE F8
CHANGE IN FLOOD VELOCITY
OPTION B
1% AEP EVENT
Model Extent
Culverts
Proposed Culvert
Change in Velocity (m/s)
< -2.0
-2.0 to -1.5
-1.5 to -1.0
-1.0 to -0.5
-0.5 to -0.2
Minimal Impact
0.2 to 0.5
0.5 to 1.0
1.0 to 1.5
1.5 to 2.0
> 2.0
No Longer Flooded
Newly Flooded
N
0
0.05
0.1
0.2
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF08_Impact_Velocity_Option_B_1pct_AEP_Event.mxd



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF09_Impact_Option_B_05pct_AEP_Event.mxd



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF10_Impact_Velocity_Option_B_05pct_AEP_Event.mxd

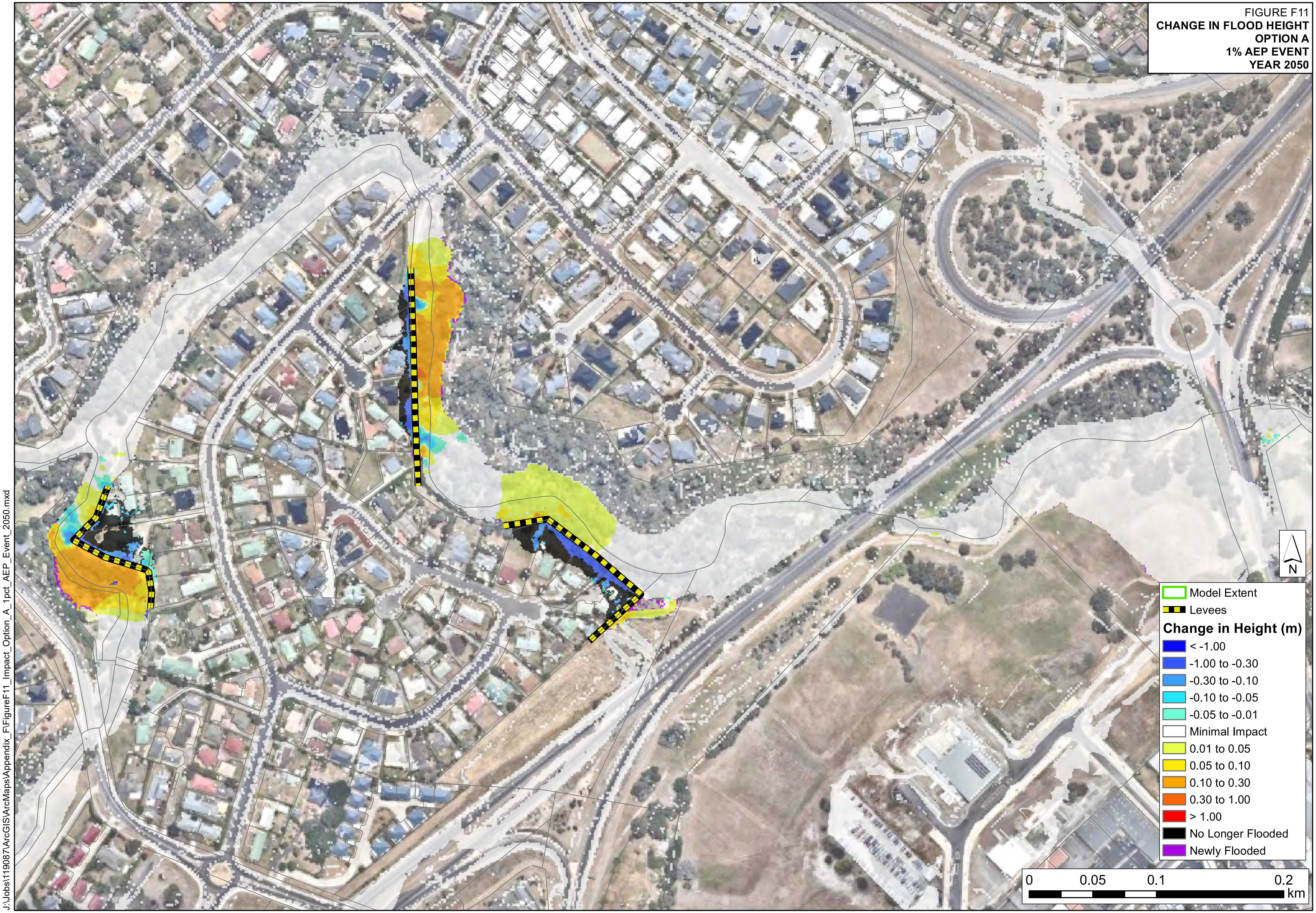


FIGURE F11
CHANGE IN FLOOD HEIGHT
OPTION A
1% AEP EVENT
YEAR 2050
N
Model Extent
Levees
Change in Height (m)
< -1.00
-1.00 to -0.30
-0.30 to -0.10
-0.10 to -0.05
-0.05 to -0.01
Minimal Impact
0.01 to 0.05
0.05 to 0.10
0.10 to 0.30
0.30 to 1.00
> 1.00
No Longer Flooded
Newly Flooded
0 0.05 0.1 0.2
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF11_Impact_Option_A_1pct_AEP_Event_2050.mxd

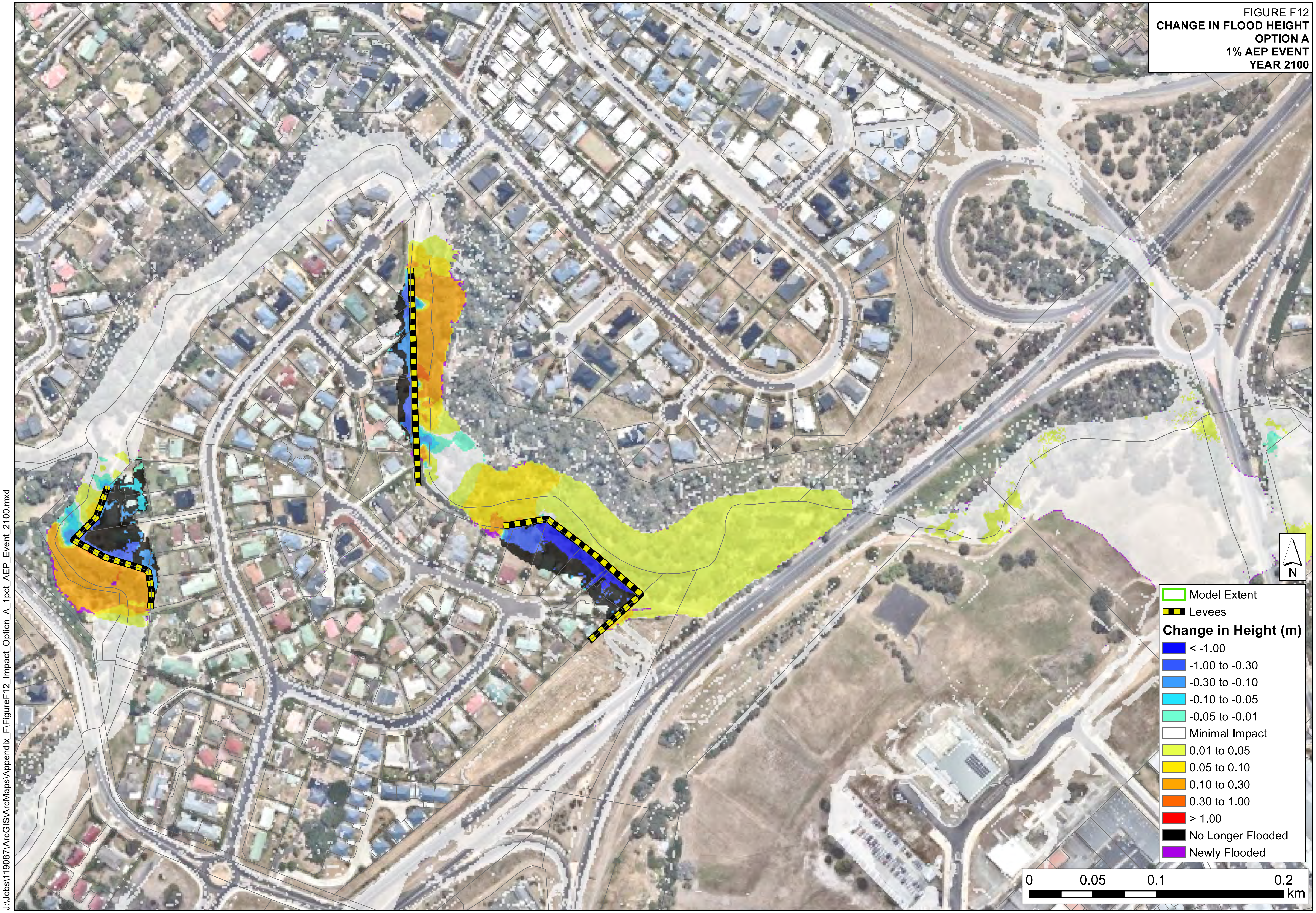


FIGURE F12
CHANGE IN FLOOD HEIGHT
OPTION A
1% AEP EVENT
YEAR 2100
Model Extent
Levees
Change in Height (m)
< -1.00
-1.00 to -0.30
-0.30 to -0.10
-0.10 to -0.05
-0.05 to -0.01
Minimal Impact
0.01 to 0.05
0.05 to 0.10
0.10 to 0.30
0.30 to 1.00
> 1.00
No Longer Flooded
Newly Flooded
0
0.05
0.1
0.2
km
N
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF12_Impact_Option_A_1pct_AEP_Event_2100.mxd

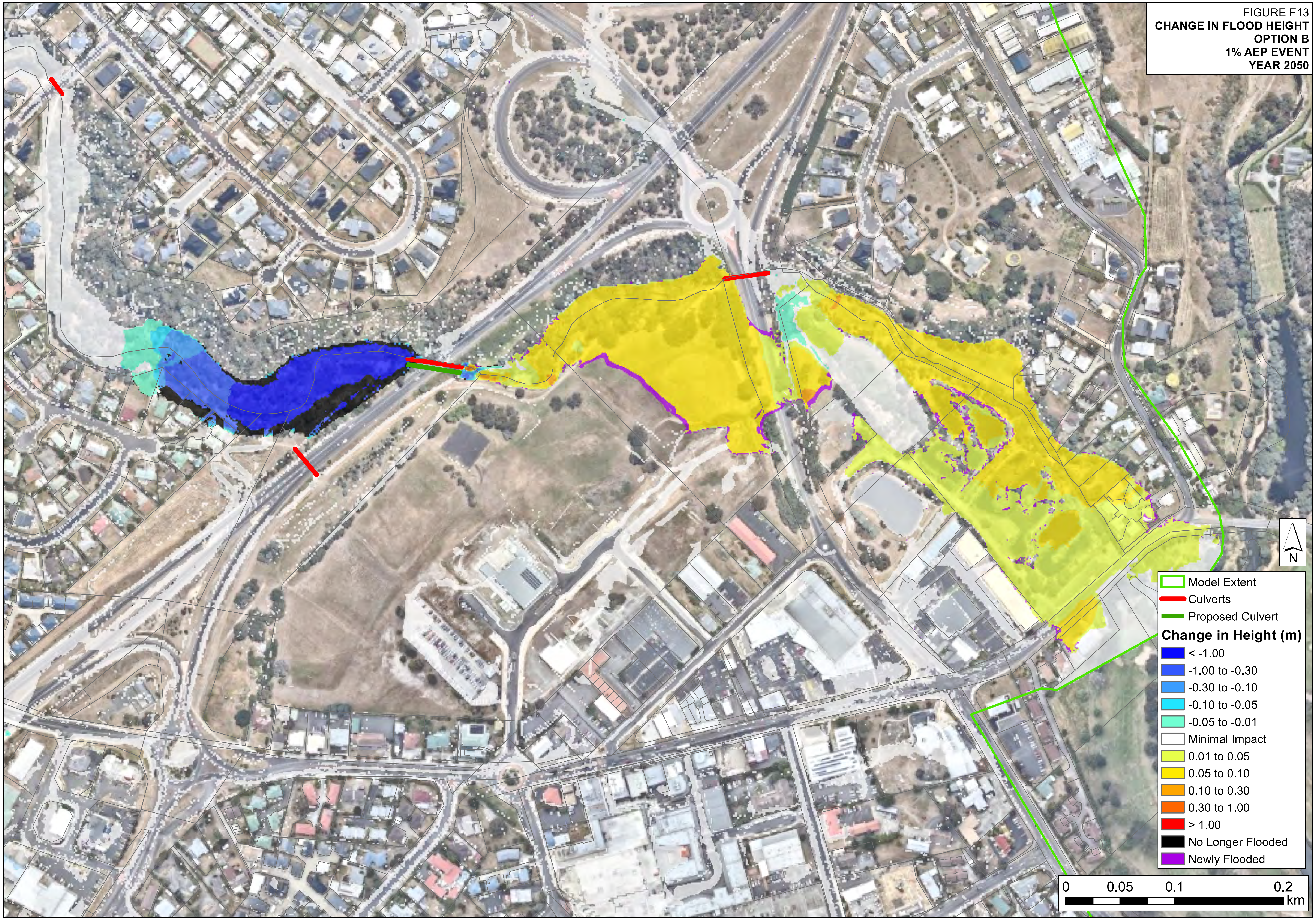


FIGURE F13
CHANGE IN FLOOD HEIGHT
OPTION B
1% AEP EVENT
YEAR 2050
Model Extent
Culverts
Proposed Culvert
Change in Height (m)
< -1.00
-1.00 to -0.30
-0.30 to -0.10
-0.10 to -0.05
-0.05 to -0.01
Minimal Impact
0.01 to 0.05
0.05 to 0.10
0.10 to 0.30
0.30 to 1.00
> 1.00
No Longer Flooded
Newly Flooded
N
0
0.05
0.1
0.2
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF13_Impact_Option_B_1pct_AEP_Event_2050.mxd

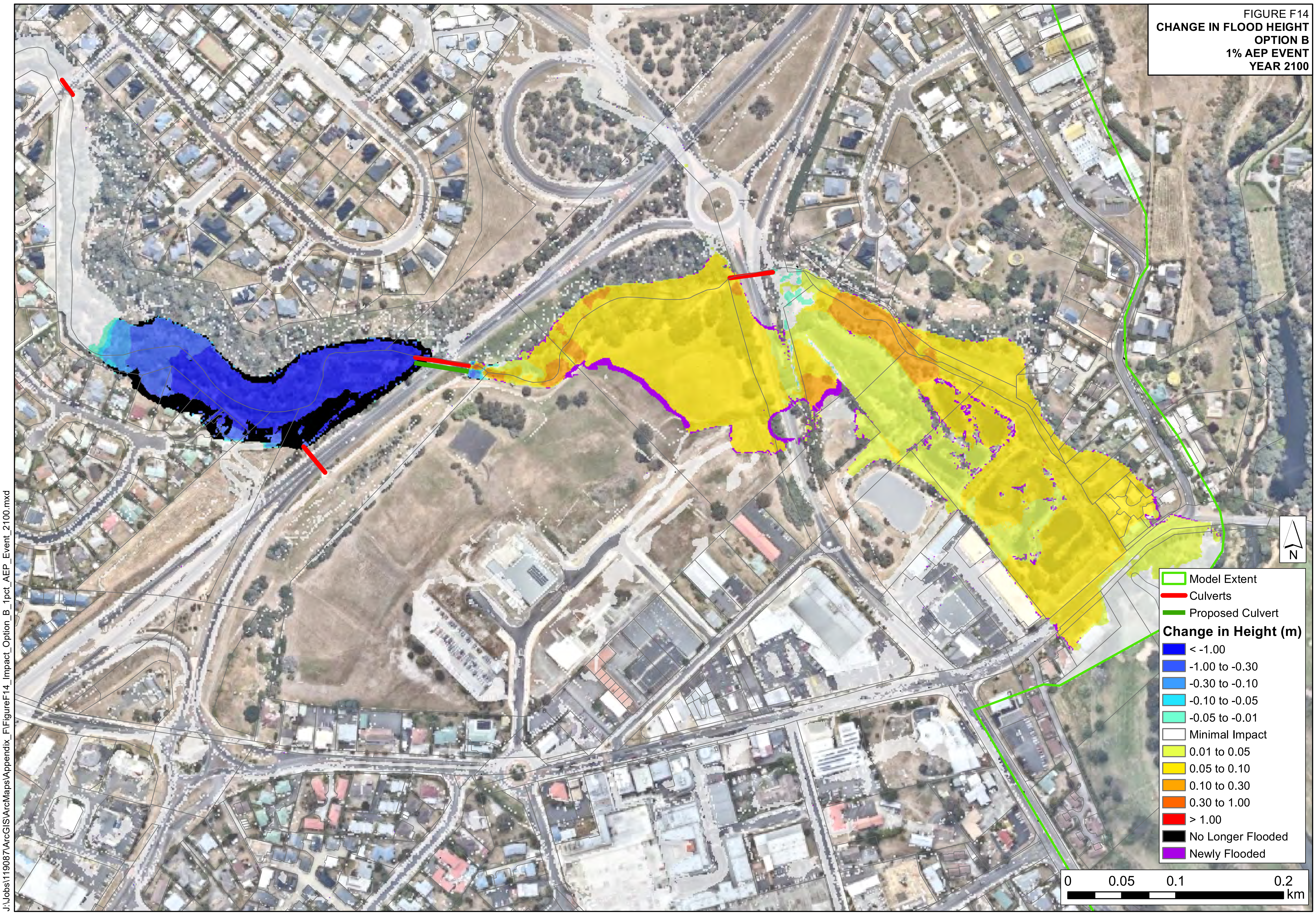
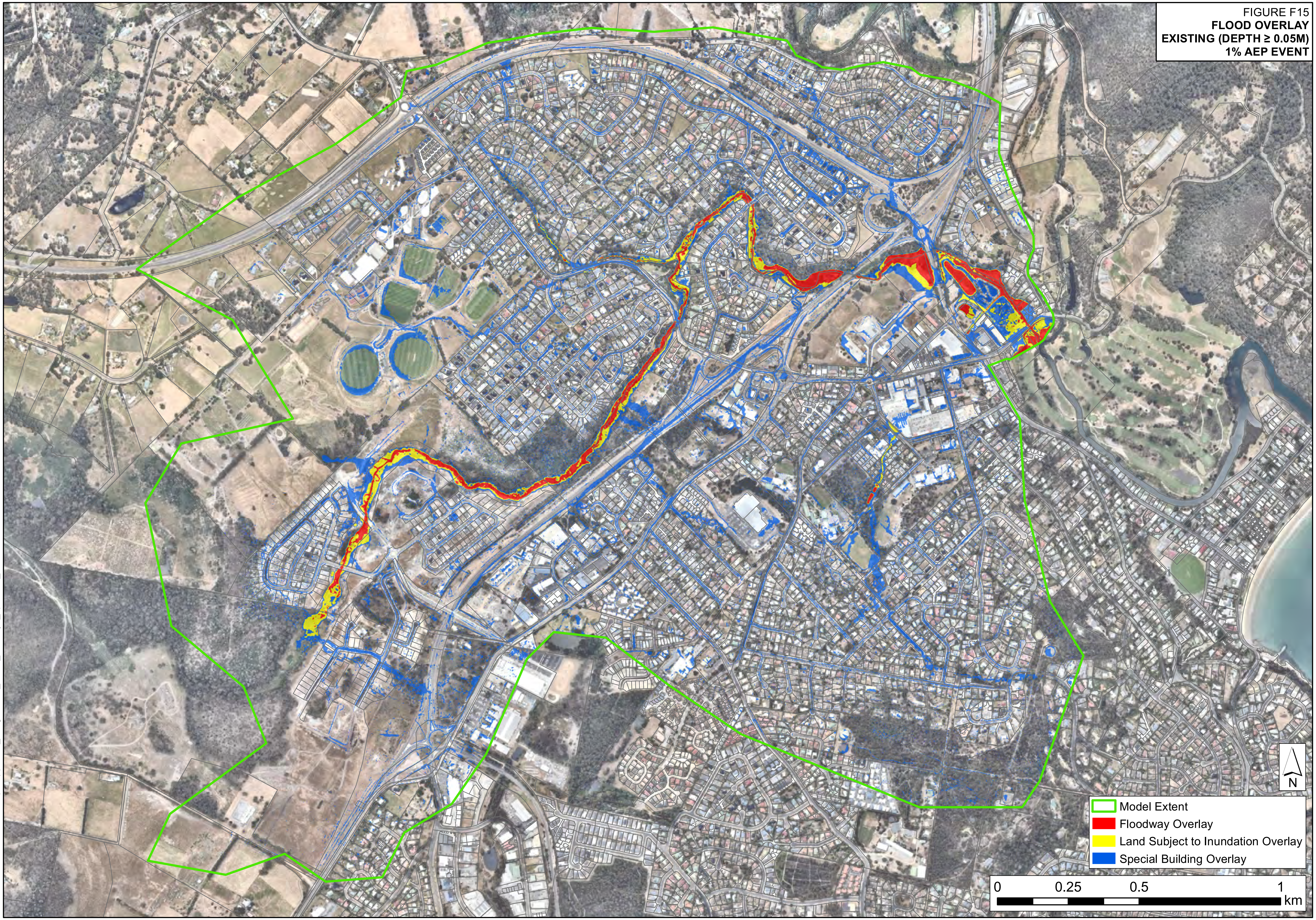


FIGURE F14
CHANGE IN FLOOD HEIGHT
OPTION B
1% AEP EVENT
YEAR 2100
Model Extent
Culverts
Proposed Culvert
Change in Height (m)
< -1.00
-1.00 to -0.30
-0.30 to -0.10
-0.10 to -0.05
-0.05 to -0.01
Minimal Impact
0.01 to 0.05
0.05 to 0.10
0.10 to 0.30
0.30 to 1.00
> 1.00
No Longer Flooded
Newly Flooded
N
0 0.05 0.1 0.2
km
J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF14_Impact_Option_B_1pct_AEP_Event_2100.mxd

FIGURE F15
FLOOD OVERLAY
EXISTING (DEPTH ≥ 0.05M)
1% AEP EVENT



J:\Jobs\119087\ArcGIS\ArcMaps\Appendix_F\FigureF15_Flood_Overlays_1pctAEP_Event.mxd

APPENDIX G. Costing

Appendix G

Item No	Description		QTY	Unit	Rate	Amount
A	**Levee works**					
1	Site establishment, Construction Management Plan and Environmental management Plan		1	Item	$20,000.00	$20,000.00
2	**Construction of Levee No.1 (1.5m High)**					
	(i) Construction of Earthwork Levee inclusive of compaction requirements, stripping and stockpiling of topsoil, excavation for berms and side batters, placing, watering and compacting approved clay filling, side batters, compaction testing, inclusive of stockpiling of all surplus spoil, construction waste and rubbish.		1150	m^3	$40.00	$46,000.00
	PROVISIONAL ITEM (ii) Re-instatement of Landscaping	Water Side	575	m^2	$10.00	$5,750.00
		Land Side	477	m^2	$10.00	$4,770.00
	PROVISIONAL ITEM (iii) Demolition of existing shared paths inclusive of material disposal		142	m	$50.00	$7,100.00
	PROVISIONAL ITEM (iv) 125mm thick concrete footpath inclusive of SL72 mesh reinforcement 50mm/20mm Class 2 FCR bedding and construction joints.		285	m^2	$150.00	$42,750.00
	PROVISIONAL ITEM (v) Tree Removal		1	Item	$7,000.00	$7,000.00
	Total Cost of Levee No. 1					$113,370.00
3	**Construction of Levee No.2 (1.7m High)**					
	(i) Construction of Earthwork Levee inclusive of compaction requirements, stripping and stockpiling of topsoil, excavation for berms and side batters, placing, watering and compacting approved clay filling, side batters, compaction testing, inclusive of stockpiling of all surplus spoil, construction waste and rubbish.		1680	m^3	$40.00	$67,200.00
	PROVISIONAL ITEM (ii) Re-instatement of Landscaping	Water Side	778	m^2	$10.00	$7,780.00
		Land Side	645	m^2	$10.00	$6,450.00
	PROVISIONAL ITEM (iii) Demolition of existing shared paths inclusive of material disposal		170	m	$50.00	$8,500.00
	PROVISIONAL ITEM (iv) 125mm thick concrete footpath inclusive of SL72 mesh reinforcement 50mm/20mm Class 2 FCR bedding and construction joints.		340	m^2	$150.00	$51,000.00
	PROVISIONAL ITEM (v) Tree Removal		1	Item	$10,000.00	$10,000.00
	Total Cost of Levee No. 2					$150,930.00
4	**Construction of Levee No.3 (0.85m High)**					
	(i) Construction of Earthwork Levee inclusive of compaction requirements, stripping and stockpiling of topsoil, excavation for berms and side batters, placing, watering and compacting approved clay filling, side batters, compaction testing, inclusive of stockpiling of all surplus spoil, construction waste and rubbish.		610	m^3	$40.00	$24,400.00
	PROVISIONAL ITEM (ii) Re-instatement of Landscaping	Water Side	294	m^2	$10.00	$2,940.00
		Land Side	244	m^2	$10.00	$2,440.00

	PROVISIONAL ITEM (iii) Demolition of existing shared paths inclusive of material disposal		128	m	$50.00	$6,400.00
	PROVISIONAL ITEM (iv) 125mm thick concrete footpath inclusive of SL72 mesh reinforcement 50mm/20mm Class 2 FCR bedding and construction joints.		257	m^2	$150.00	$38,550.00
	PROVISIONAL ITEM (v) Tree Removal		1	Item	$7,000.00	$7,000.00
	Total Cost of Levee No. 3					$81,730.00
	TOTAL LEVEE WORKS					**$366,030.00**
B	PROVISIONAL ITEMS AND CONTINGENCIES					
5	PROVISIONAL ITEM Contingencies (30%)		1	Item		$109,809.00
6	PROVISIONAL ITEM Design, documentation, contract administration and supervision of works (10%)		1	Item		$36,603.00
	TOTAL CONTINGENCIES					$146,412.00
A	**LEVEE WORKS**					**$366,030.00**
B	**PROVISIONAL ITEMS AND CONTINGENCIES**					**$146,412.00**

SUBTOTAL:	$512,442.00
GST:	$51,244.20
TOTAL AMOUNT (Including GST):	$563,686.20