Issue	Description	Date
P1	ISSUED FOR INFORMATION	25-01-13

Bar Scales

0 20 40 60 80m

1 : 800

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Client

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Scales	1 : 800	Drawn	MM
		Designed	MM
Grid	MGA	Checked	AMcL
Height Datum	AHD	Approved	

Project

WOOLOOWARE BAY TOWN CENTRE

Title

STORMWATER CATCHMENT PLAN

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V1

Appendix F

WMA Water Flood Assessment Report

BLUESTONE CAPITAL VENTURES
NO 1 PTY LTD

WOOLOOWARE BAY TOWN CENTRE REDEVELOPMENT – RETAIL SITE

FLOOD ASSESSMENT REPORT

JANUARY 2013

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WOOLOOWARE BAY TOWN CENTRE REDEVELOPMENT – RETAIL SITE

FLOOD ASSESSMENT REPORT
JANUARY 2013

Project **Woolooware Bay Town Centre Redevelopment – Retail Site**	**Project Number** **112077**
Client **Bluestone Capital Ventures No 1 Pty Ltd c/o AT&L**	**Client's Representative** Anthony McLandsborough
Authors **Dr Chin Hong Cheah** **Richard Dewar**	**Prepared by**
Date **29 January 2013**	**Verified by**

Revision	Description	Date
1	Draft Flood Assessment Report	18 January 2013
2	Final Flood Assessment Report	29 January 2013

WOOLOOWARE BAY TOWN CENTRE REDEVELOPMENT – RETAIL SITE

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EXECUTIVE SUMMARY

Background: Bluestone Capital Ventures No 1 Pty Ltd is proposing to re-develop the car park on the east side of the Cronulla Sharks Rugby League Club at 461 Captain Cook Drive, Woolooware as a retail centre. The surrounding land has been subject to flooding in the past (notably March 1975 but also more recently in 1983). As a result WMAwater (a specialist water engineering consulting company) was engaged to provide a flood assessment for the existing and post development scenarios.

Approach: A large amount of topographic, survey, pit and pipe and other data was collected and incorporated into the modelling approach. The study adopted best practice approaches using the DRAINS hydrologic model and the TUFLOW 1D/2D (one-dimensional and two-dimensional) hydraulic model to undertake the flood assessment.

Analysis: Design flood level and hazard information were produced for the 20 year ARI, 100 year ARI and Probable Maximum Floods for both existing and post development conditions. In addition analysis was undertaken to assess the possible impacts of a climate change induced sea level rise and rainfall intensity increase.

Floor Levels: The ground floor level for the retail development site is proposed to be at 4 mAHD, which is significantly higher than the 100 year ARI peak flood level of 2.46 mAHD and even the PMF peak flood level of 3.14 mAHD. With the exception of the main entrance off Captain Cook Drive whereby the crest level is proposed to be at 2.6 mAHD which is still above the 100 year ARI peak flood level, all other entrances are proposed at a similar level to the ground flood level (4 mAHD). Therefore, the site is not subject to any flood risk and is in compliance with Council's development controls for flooding.

Impact of Proposed Development: Based on the results generated using the DRAINS hydrologic model and TUFLOW hydraulic model developed for this flood assessment, the proposed retail development results in up to 20 mm off-site impact on the 1% AEP peak flood levels, with the affected area located at the entrance to the development and Captain Cook Drive. However, this impact is regarded as acceptable considering it is on the side of the roadway rather than at residential dwellings where it would increase flood damages.

Climate Change: Overall, the results show that an increase in peak flood level of up to a maximum of 0.2 m can be expected for the development site as a result of climate change and as such would not be an issue considering the proposed floor levels have been designed to be at a level above the PMF flood level plus any increase induced by climate change. However the relatively flat topography means that a small increase in peak level translates to a relatively large increase in flood extent.

Management of the Flood Problem: An evacuation route is proposed which serves as an access route from the site to high ground via Woolooware Road as well as providing access for emergency vehicles (i.e. SES, ambulance) to get to the site. As such, it is pertinent that the proposed new intersection is designed to be flood free up to the 100 year ARI event. A flood response plan should be prepared by the building management to minimise the risk associated with evacuations by providing information regarding evacuation routes, refuge areas, what to do/not to do during floods etc. Any evacuation undertaken would usually be under the direction of the lead agency, the State Emergency Services (SES). As such, it is necessary that SES be made aware of the new retail development and the local SES response planning can be prepared. Since the development proposed is above the PMF, there are no further requirements in regards to the construction as there are no flood liable structures. Nevertheless, reliable pedestrian and vehicle access to a place of refuge should be maintained.

1. INTRODUCTION

1.1. Background

WMAwater has been commissioned by Bluestone Capital Ventures No 1 Pty Ltd (c/o AT&L) to conduct a flood assessment for the proposed Woolooware Bay Town Centre development on Captain Cook Drive in Woolooware. The proposed development comprises residential, retail, landscaped open space, parking as well as entertainment areas. However this report only considers the proposed retail and car parking proposal on the east side of the Cronulla Sharks Rugby League Club. Proposed development in other related areas will be considered in separate reports.

The development site is located within the Woolooware Bay catchment adjacent to Captain Cook Drive and Woolooware Bay and plans describing the development proposal are provided in Appendix B. A glossary of flood related terms is provided in Appendix A.

1.2. Scope of Work

The flood assessment undertaken included the following:

- establishing hydrologic and hydraulic models to represent flood behaviour;
- running the 20Y and 100Y ARI events as well as the PMF;
- defining existing flooding behaviour (current on ground conditions pre-development);
- define proposed development conditions and model for the retial and car parking proposal only;
- incorporate climate change modelling as per NSW Government 2009 guidelines (in September 2012 the NSW Government repealed its sea level rise policy and Councils must now make their own decision);
- assess development impacts both with and without climate change for the range of flood events;
- advise on proposed floor levels in compliance with Council's DCP;
- develop mitigation works in case of impacts;
- assess provisional hazard for the existing and developed case; and
- consider flood safety including evacuation for events up to the PMF.

The report herein documents the assessment and evaluation of flooding in relation to the retail and club development only (assessment for the residential development will be provided in a separate report). The location of the development site is shown in Figure 1. This report does not include any water quality aspect, preparation of a stormwater management plan or erosion/sedimentation assessment.

2. BACKGROUND

2.1. Study Area

The proposed development site is located within the Woolooware Bay catchment which is part of the Sutherland Shire local government area. A significant catchment area of approximately 252 ha drains to the site as shown in Figure 2. Within the site itself, runoff from the playing fields include that of Toyota Park drains to an open, tidal channel. Land use in the catchment is predominately residential upstream with recreational parks and playing fields located downstream. The catchment slopes from south to north towards the Bay with the lower reaches typically flat and low lying.

2.2. Drainage System and Flood Mechanism

The catchment upstream of the development site and Woolooware Golf Course is drained primarily by a Council owned sub-surface pipe system, with natural earth drainage channels located downstream in the golf course. Floodwater discharges into Woolooware Bay either in the form of diffuse outflows or through the tidal channel located west of Toyota Park and the stormwater pipe adjacent to the Solander Playing Fields.

Captain Cook Drive, which is situated along the downstream end of the catchment, acts as a significant barrier to runoff from entering Woolooware Bay. Large quantities of floodwaters would flow from the Woolooware Golf Course onto Captain Cook Drive and subsequently onto the Toyota Park area. Another governing flood mechanism for this part of the catchment is tidal inundation due to its proximity to Woolooware Bay and flat topography.

2.3. Flood History

One of the most recent significant floods that occurred within the Woolooware Bay catchment is the 13th to 16th May 2003 event. This event has been well-documented with newspaper reports and correspondence received by Council record heavy damages to factories, houses and motor vehicles. Many of the community complaints were recorded on Council's customer response management system (CRMS).

Historic photos have been obtained that highlight the potential magnitude of flooding in the region. Photo 1 shows flooding of Captain Cook Drive at the intersection with Gannons Road for the March 1975 event. This event was known to have caused widespread flooding throughout Sydney and the rainfall was documented in Reference 1. It was estimated that based on rainfall and flood records at Miranda this event may have approached a 1 in 1000 ARI for a 12 hour duration and a 1 in 400 ARI for a 2 hour duration.

This event caused widespread flooding in Sans Souci, Kogarah and in others parts of the Sutherland Shire. It is likely that the rainfall intensities will have varied greatly across the area and at this locality the magnitude of the event cannot be accurately determined.

Photo 1: Flooding at the corner of Gannons Rd and Captain Cook Dr, dated March 1975 (courtesy of Ross Myers)

It should be noted that Captain Cook Drive has been raised since 1975 and other works for the adjacent sporting fields will have changed the topography and thus the resulting flood levels. It is likely that filling to create the sporting fields on the north (downstream) side of Captain Cook Drive will have increased flood levels upstream. At the time the assessment of flooding for a proposed development was not as sophisticated or as well understood as today.

Since 2003 there have been several events that have caused flooding of Captain Cook Drive causing traffic disruption (there is video on YouTube of the 12th March 2012 event). These events are likely to have a magnitude of less than 5 year ARI and possibly even more frequent. The magnitude of the rainfall event can only be accurately determined if there is a nearby pluviometer (records rainfall). Whilst there are pluviometers nearby the localised nature of the storms means that an accurate estimate is not always possible.

3. AVAILABLE DATA

3.1. Background

Various items of data as well as reports salient to the study have been collected and reviewed. Most reports and datasets were sourced from Sutherland Shire Council and supplemented by additional survey where required. The assistance of Sutherland Shire Council in this regard is gratefully acknowledged.

Reports were reviewed particularly for topographic/hydrologic parameters as well as observations of historical flood events. The key focus of the exercise was to collect data suitable for the model calibration process. This section provides a brief description of the various forms of data utilised in the study.

3.2. Previous Studies

The main studies reviewed as part of the present assessment included:

- Initial Subjective Assessment of Major Flooding, 2004 (Reference 2);
- Lower Georges River Floodplain Risk Management Study and Plan, 2011 (Reference 3); and
- Lower Georges River Stormwater Management Plan, 1999 (Reference 4).

The minor studies reviewed were:

- Stormwater drainage and water quality strategy for proposed re-zoning of the Sharks eastern site, 2002 (Reference 5); and
- Flood Study for proposed upgrading of Toyota Park for Cronulla Sutherland Leagues Club Ltd, 2007 (Reference 6).

Further descriptions of the above studies are provided in later sections where relevant to the current investigation.

3.3. Design Rainfall

Design rainfalls were obtained from the Bureau of Meteorology (BoM) and temporal patterns were obtained from Australian Rainfall and Runoff (Reference 7). The Intensity-Frequency-Duration (IFD) data for the catchment is provided in Table 1.

Table 1: IFD Data for Woolooware Bay Catchment

Intensity-Frequency-Duration Table

Location: 34.050S 151.150E NEAR.. Woolooware Issued: 22/11/2011

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

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	97.8	126	160	180	206	240	266
6Mins	91.6	118	150	168	193	225	250
10Mins	75.0	96.5	124	140	161	188	209
20Mins	54.9	71.1	92.7	105	122	144	160
30Mins	44.7	58.0	76.2	87.0	101	120	134
1Hr	30.3	39.4	52.1	59.8	69.6	82.7	92.7
2Hrs	19.7	25.7	33.9	38.9	45.3	53.7	60.3
3Hrs	15.2	19.7	26.0	29.7	34.6	41.0	45.9
6Hrs	9.69	12.5	16.4	18.6	21.6	25.5	28.5
12Hrs	6.22	8.03	10.4	11.8	13.7	16.1	17.9
24Hrs	4.03	5.20	6.75	7.65	8.85	10.4	11.6
48Hrs	2.57	3.32	4.32	4.90	5.68	6.68	7.46
72Hrs	1.91	2.47	3.21	3.63	4.20	4.95	5.52

(Raw data: 39.7, 8.02, 2.47, 84.21, 16.05, 4.96, skew=0.00, F2=4.29, F50=15.85)

Probable Maximum Precipitation (PMP) rainfall depths used to determine the Probable Maximum Flood (PMF) were obtained from Reference 8 using the generalised short-duration method. The maximum duration for which the method is applicable in the region is 6 hours. The parameters used for estimating the PMP are:

- Terrain classification: smooth;
- Adjustment for catchment elevation (EAF): 1;
- Moisture Adjustment Factor (MAF): 0.7; and
- Ellipses enclosing the catchment: A and B (refer to Reference 8 for further explanation of ellipsoid selection).

3.4. Design Tidal Levels

The proximity of the study area to Woolooware Bay means that flood behaviour is influenced by ocean storm and tidal effects. As a result flooding on site can be caused by intense rainfall over the catchment, elevated tidal levels (astronomic tide plus storm surge), or a combination of both.

Design tidal levels adopted for this study are listed in Table 2, based on analyses of tidal records from Fort Denison in Sydney Harbour. It was deemed that the results for Woolooware Bay would be similar.

Table 2: Design Tidal Levels

ARI (years)	Peak Level (mAHD)
20	1.38
50	1.42
100	1.45
Extreme or PMF	Not known but assumed as 1.50

3.5. Survey Data

Airborne Laser Scanning (ALS) data of the study area was obtained from Council to define the ground surface elevations. The ALS data was collected in October 2005 by AAMHATCH. The ALS provides ground level spot heights from which a Digital Terrain Model (DTM) can be constructed (refer to Figure 2). For well defined points mapped in areas of clear ground, the expected nominal point accuracies (based on a 68% confidence level) are ±0.15 m (vertical accuracy). When interpreting the above, it should be noted that the accuracy of the ground definition can be adversely affected by the nature and density of vegetation and/or the presence of steeply varying terrain. This data formed the foundation of the hydraulic model build process.

3.6. Other Spatial Information

A number of spatial datasets were also obtained from Council including:

- property cadastre layer;
- geo-referenced aerial photography of Woolooware Bay catchment; and
- various GIS layers relating to current land use, building outlines, water quality devices, major hydraulic structures and drainage infrastructure.

All GIS data have been provided in a MapInfo/ArcGIS compatible format and these layers were used to aid model schematisation for the hydrologic and hydraulic models.

3.7. Pit and Pipe Data

Council provided a database of the pit and pipe network within the catchment dated 7th December 2011 with physical details including:

- coordinates of each pit;
- linkage between pits;
- pipe type and dimensions; and
- pit details (type of pit, inlet type and dimensions, and depth to invert).

In addition, stormwater drainage data were obtained from RailCorp for the Sutherland – Cronulla railway line. In addition the entire drainage network was re-surveyed by a registered surveyor during the period of February to August 2012 and any new information included. A plan view of the pit and pipe network is shown in Figure 3.

4. HYDROLOGIC MODELLING

4.1. Overview

Hydrologic models suitable for design flood estimation are described in AR&R 1987 (Reference 7). These models or techniques range from simple procedures to estimate peak flows (such as Probabilistic Rational Method) to more complex rainfall-runoff routing models that provide estimates of complete flow hydrographs. In current Australian engineering practice, examples of the more commonly used runoff routing models for rural catchments include RORB, RAFTS and WBNM (Watershed Bounded Network Model). For urban catchments with a significant pit and pipe network DRAINS is the most commonly used hydrologic model in NSW. DRAINS is specifically designed to simulate flow into kerb inlet pits in roads and through and underground pipe network. All these models allow the rainfall depth to vary both spatially and temporally over the catchment, and have parameters governing runoff volume/shape that can be calibrated against recorded data.

For the above reason hydrologic modelling of the Woolooware Bay catchment was carried out using DRAINS. The total catchment represented by the DRAINS model draining across the subject site to Woolooware Bay is 252 ha (note parts of the catchment shown on Figure 3 do not drain to the subject site). Catchment areas for the hydrologic models were delineated using the available topographic information obtained from Council (i.e. DEM generated from the ALS data), aerial photos, drainage network data and field inspection. A total of 112 sub-catchments were found to contribute to the subject site. The sub-catchments are shown in Figure 2. The sub-catchment layout ensures that where hydraulic controls exist, these are accounted for and are able to be appropriately incorporated into the hydraulic routing.

4.2. DRAINS

4.2.1. Introduction

DRAINS (Reference 10) is a hydrologic/hydraulic model that can simulate the full storm hydrograph and is capable of describing the flow behaviour of a catchment and pipe system for real storm events, as well as statistically based design storms. It is designed for analysing urban or partly urban catchments where artificial drainage elements have been installed. The DRAINS model is broadly characterised by the following features:

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

DRAINS generates a full hydrograph of surface flows arriving at each pit and routes these through the pipe network or overland, combining them where appropriate. Used in conjunction with a 2D (two-dimensional) hydraulic model, the benefit of DRAINS is that it produces a flow hydrograph at all modelled pits (in this instance for each modelled sub-catchment) that can then be input into the 2D model.[1]

4.2.2. Input Data

For the hydrologic modelling, the sub-catchment details were defined and collated into a spreadsheet for input to DRAINS. Sub-catchment areas were obtained based on the ALS survey and assuming that properties drain to the street and flow in the street is along the gutters and in one direction only (i.e. does not sub-divide at an intersection). The delineation of these sub catchment areas is shown on Figure 2. For each sub-catchment area the proportion of pervious (grassed), impervious (paved), supplementary area (paved area not directly connected to pipe system) was determined from field and aerial photographic inspections. For residential areas (include roads) a relatively high value of % imperviousness was adopted to reflect the likely low infiltration capacity of suburban yards and open space areas.

4.2.3. Adopted Model Parameters

Losses from a paved or impervious area are considered to comprise only an initial loss (an amount sufficient to wet the pavement and fill minor surface depressions). Losses from grassed areas are comprised of an initial loss and a continuing loss. The continuing loss was calculated from an infiltration equation curve incorporated into DRAINS and is based on the estimated representative soil type and antecedent moisture condition. It was assumed that the soil in the catchment has a slow infiltration rate potential and the antecedent moisture condition was assumed saturated. The latter was justified by the fact that the peak rainfall burst can typically occur within a longer event that has a duration lasting days. The adopted parameters for the design runs are summarised in Table 3.

Table 3: Adopted DRAINS Model Parameters

RAINFALL LOSSES	
Paved (Impervious) Area Depression Storage (Initial Loss)	1.0 mm
Supplementary Area Depression Storage (Initial Loss)	1.0 mm
Grassed (Pervious) Area Depression Storage (Initial Loss)	5.0 mm
SOIL TYPE	3
Slow infiltration rates. This parameter, in conjunction with the AMC, determines the continuing loss	
ANTECEDENT MOISTURE CONDITIONS	3
Description	Rather wet
Total Rainfall in 5 Days Preceding the Storm	12.5 to 25 mm

[1] The DRAINS model developed for this study does not account for surface controls or storages (e.g. embankments, local depressions) hence direct comparison of flows with those predicted by the hydraulic model is not possible, with the exception of smaller events whereby overland flow is not dominant.

5. HYDRAULIC MODELLING

5.1. Overview

A key objective of this study is to define the flood behaviour within the existing study area in terms of flood levels, flows and velocities. An integrated one-dimensional/two-dimensional (1D/2D) hydraulic model was used to achieve this using the TUFLOW (Reference 11) hydrodynamic modelling package.

TUFLOW software is widely used for pipe and overland flow hydraulic simulation of unsteady flow systems throughout Australia and the UK. TUFLOW is a finite difference numerical model for the solution of the depth averaged shallow water flow equations (Reference 11). The model is capable of dynamically simulating complex overland flow regimes and interactions with sub-surface drainage systems. 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 overland and pipe flow.

For the hydraulic analysis of complex overland flow paths, a combined 1D/2D model such as TUFLOW provides several key advantages when compared to a 1D only model. For example, in comparison to a purely 1D approach, a combined 1D/2D approach can:

- provide localised detail of any topographic and/or structural features that may influence flood behaviour;
- better facilitate the identification of potential overland flow paths and flood problem areas;
- dynamically model the interaction between the drainage system and the complex overland flow paths, including surcharging effects; and
- inherently represent the available flood storage within the 2D model geometry, which is particularly important for assessment of the detention basin performance.

In comparison to previous studies, a 2D model can better define the spatial variations in flood behaviour across the study area. Information such as flow velocity, flood levels and hydraulic hazard can be readily mapped across the model extent. This information can be easily integrated into a GIS-based environment for result presentation.

5.2. Model Extents

The TUFLOW hydraulic model established in this study extends to the whole of the Woolooware Bay catchment. The TUFLOW model incorporated both major subsurface drainage features and overland flow paths within the model extent. The two components were dynamically linked such that the model accounted for the interactions between the drainage system and overland flow behaviour. The TUFLOW model layout is shown on Figure 3.

5.3. Digital Terrain Model

Overland flow paths in the hydraulic model were defined using a DTM. The DTM was compiled largely from the ALS datasets. The DTM was sampled on a regular grid of 3 m square cells for use in the model. This level of detail was deemed to allow sufficient resolution of drainage features while still retaining practical computational run-times for an area of this scale (model runs take approximately 6 hours for a 1 hour rainfall event).

Two DTMs were established to represent the following conditions:

- Existing conditions; and
- Post-development conditions – retail development and upgrade of Captain Cook Drive/Woolooware Road intersection (refer Appendix B).

The DTM for post-development conditions was developed based on concept drawings provided by AT&L (refer Appendix B).

5.4. Breaklines

A number of significant hydraulic features which are likely to impact on the flow behaviour exist within the Woolooware Bay catchment. These hydraulic features are particularly important due to the flat nature of the topography at the downstream parts of the catchment. Breaklines were used throughout the study area in order to define hydraulic controls not well represented in the 3 m DTM used to inform the model grid.

The key benefit of using breaklines is that high resolution height data for significant hydraulic features (such as road kerbs) can be utilised in conjunction with the coarser 3 m DTM for modelling purposes.

Significant hydraulic features found in the study area include:

- road kerbs and gutters;
- major road embankments; and
- Sutherland to Cronulla railway line.

5.5. Key Model Parameters

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

The majority of the catchment consists of urban dwellings with the remainder as generally golf courses and recreational parks located at the downstream parts of the study area. The corresponding Manning's 'n' roughness values adopted for modelling purposes are shown in Table 4. These values have been adopted based on site inspections and also from neighbouring flood studies (i.e. Reference 12). It is important to note that all buildings have been "nulled" or removed from the model grid, hence there is no need to use higher Manning's 'n' for those areas since the "blockage" effect presented by those buildings has already been accounted for. It was assumed that the flood storage presented by the buildings is insignificant.

Table 4: Summary of Manning's 'n' Values

Surface	Adopted Manning's 'n'
Default	0.03
Regular Channel	0.03
Channel with Deep Pool and Weed	0.05
Channel Bank with Heavy Growth/Mangrove	0.1
Roads	0.015
Urban/Dwellings	0.03*
Commercial/Industrial	0.03*
Railway	0.05
Light Vegetation/Golf Course	0.035
Medium Vegetation	0.06

* Buildings have been removed from model grid, hence lower Manning's n can be adopted

The 2D numerical scheme for the TUFLOW model includes an allowance for sub-grid scale turbulence and eddies, features that are too small to be modelled directly. These physical processes result in energy loss and can affect the flow behaviour. Within the TUFLOW model the effects of these sub-grid processes are modelled by the introduction of an eddy viscosity formulation, where energy losses are applied either as a constant term or according to the Smagorinsky formulation, in proportion to the flow velocity and the 2D cell edge length. For this assessment, a combination of the constant and Smagorinsky eddy viscosity formulations were used, with coefficients of 0.1 and 0.2 respectively as recommended in the TUFLOW manual (Reference 11).

5.6. Pits and Pipes Network

Pit and pipe information as described in Section 3.7 was used to create a 1D drainage network in TUFLOW. Pipes of all sizes were included in the TUFLOW model though smaller pipes are generally prone to blockages during storms due to leaves and debris. Temporary blockage may also occur during a storm as the pit entry may be restricted by a vehicle parking over the grate or leaves/silt/branches filling the inlet.

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

The approach adopted for this study has been to assume 50% blockage at all culverts and pipes, which was consistent with the blockage factor adopted for a neighbouring flood study

(Reference 12). This approach has been adopted to take into account blockage caused by debris (cars, fencing, vegetation) being swept into drainage structures. A site inspection of the box culverts discharging to the open tidal channel west of Toyota Park also affirmed that 50% was a reasonable factor to be adopted in modelling drainage blockage for this study area.

5.7. Boundary Conditions

Local catchment inflows within the TUFLOW hydraulic model extent were derived from the DRAINS hydrologic model. Runoff hydrographs from the hydrologic model (a hydrograph for each of the 112 sub-catchments) are applied at pit locations or low-lying areas within each sub-catchment and used as inflows into the 2D model.

The downstream boundary of the study area is Woolooware Bay and natural variability of water level is expected in the downstream catchment areas from both tidal and catchment flows. For design flood estimation a level in Woolooware Bay is required for calculation of water levels and pipe discharges in the lower parts of the catchment. This tailwater boundary is shown in Figure 3.

5.7.1. Adopted Tailwater Levels

As noted previously, in addition to runoff from the catchment, the downstream areas of the catchment and the tidal channel west of Toyota Park are influenced by backwater effects from high water levels in Woolooware Bay. These two distinct mechanisms produce flooding in the study area but may not result from the same storm. It is acknowledged however that this may not necessarily be the case and that ocean influences may occur in conjunction with rainfall events. Consideration must therefore be given to account for the joint probability of coincident flooding from both catchment runoff and backwater effects from Woolooware Bay.

A full joint probability analysis is beyond the scope of the present study. Recommended in Reference 9 is the ‘peak envelope’ approach that adopts the highest of the predicted levels from the two mechanisms to estimate design flood levels. The same document also advised that a 100 year ARI ocean event in conjunction with a 100 year ARI rainfall event would likely to produce flood levels greater than the 100 year ARI.

Table 5 sets out the joint probabilities of the ocean and rainfall design events. Thus a 100 year ARI event is an envelope of the 100 year ARI ocean event (1.45 mAHD peak ocean level combined with a 20 year ARI rainfall event) and a 100 year ARI rainfall event (20 year ARI 1.38 mAHD peak ocean level combined with a 100 year ARI rainfall event). For the 20 year ARI event the same ocean and rainfall conditions are used for the ocean and rainfall event scenarios.

Table 5: Adopted Co-incidence of Ocean and Rainfall Events

OCEAN Envelope		**DESIGN FLOOD EVENT (ARI)**	**RAINFALL Envelope**	
Peak Design Ocean Event (ARI) and level (mAHD)	Co incident Design Rainfall Event (ARI)		Design Rainfall Event (ARI)	Co incident Design Ocean Event (ARI) and level (mAHD)
PMF (1.5)	100 year	**Extreme**	PMF	100 year (1.45)
100 year (1.45)	20 year	**100 year**	100 year	20 year (1.38)
50 year (1.42)	20 year	**50 year**	50 year	20 year (1.38)
20 year (1.38)	20 year	**20 year**	20 year	20 year (1.38)

6. DESIGN FLOOD ASSESSMENT

6.1. Critical Duration

Critical storm duration analysis is undertaken to determine the storm duration that produces the greatest flood levels (at the subject site) for the given design event. A range of storm durations were modelled for the 100 year ARI event and it was found that the critical duration is 1 hour for the development site (see Figure 4). Therefore, for all design events, excluding the PMF, the 1 hour duration was used to determine the peak flood levels.

A similar process was undertaken for the PMF with various PMP durations (15 minutes to 6 hours) modelled so that peak flood levels and associated rainfall durations could be identified. The 45-minute duration PMP was determined to be the critical duration for the development site and was thus used to determine peak flood levels.

6.2. Overview of Results

A number of maps have been produced to display the flood affected regions for the various design events. It should be noted that inundation patterns and/or peak flood levels shown for design events are based on best available estimates of flood behaviour within the catchment. Inundation from local overland flow may vary depending on the actual rainfall event and local influences (parked cars, change in topography, earth works etc.). Tabulated results (Table 6) are provided for ease of comparison between flood events at locations of interest around the study area and these are shown on Figure 1.

Two scenarios are considered and modelled herein: existing conditions and post-development conditions. For the latter, the proposed retail development and upgrade of the intersection at Captain Cook Dr/Woolooware Rd were incorporated into the model. The flood impacts resulting from the proposed works are assessed by comparing the model results between the two scenarios.

Table 6: Design Storm Peak Flood Levels (mAHD) for Existing and Post-Development Conditions

	Existing Conditions			Post-Development Conditions		
LOCATION	**20Y ARI**	**100Y ARI**	**PMF**	**20Y ARI**	**100Y ARI**	**PMF**
1 Captain Cook Dr/Woolooware Rd Intersection East	1.65	1.74	2.31	+0.03	+0.03	+0.11
2 Captain Cook Dr /Woolooware Rd Intersection West	2.28	2.32	2.67	+0.06	+0.05	+0.08
3 Retail Site Entrance	2.41	2.46	3.14	-0.02	-	-0.02
4 Captain Cook Dr	2.39	2.46	3.13	-	-	-
5 Tidal Channel	2.02	2.38	3.06	-	-	-
6 Fitness First Open Drain	1.51	1.53	1.80	-	-	-0.14

Notes: Difference calculated relative to Existing Conditions (i.e. a positive number indicates an increase in flood level compared to existing conditions)

A summary of the results is provided as follows:

- Peak flood depths and levels for all design flood events for existing conditions, Figure 5 - Figure 7;
- Peak flood depths and levels for all design flood events for post-development conditions, Figure 8 - Figure 10;
- Peak flood velocities for all design flood events for post-development conditions, Figure 11 - Figure 13;
- Flood impact map for the proposed development for the 100 year ARI event, Figure 14;
- Flood extents for various climate change scenarios for the 100 year ARI and PMF events, Figure 15 - Figure 16; and
- Provisional flood hazard categorisation for existing conditions and post-development conditions, Figure 17 - Figure 22.

As discussed in Section 5.7.1, a 'peak envelope' approach that adopts the highest of the predicted levels from the two flooding mechanisms, i.e. catchment runoff and tidal inundation, has been used to estimate the design flood levels.

6.3. Existing Flood Behaviour

Floodwaters originating from Woolooware Golf Course exceed the capacity of the twin box culverts beneath the roadway and overtop Captain Cook Drive for all the modelled design events. Figure 5 to Figure 7 indicate that the majority of the proposed retail development site remains flood free for all design flood events. The exception is at the low spot of the entrance to the Cronulla Sharks Rugby League Club off Captain Cook Drive whereby in the 100 year ARI event inundation of up to 0.3 m occurs (refer to Figure 6). At this entrance off Captain Cook Drive the site was identified as low provisional (based on depth and velocity) for events up to and including the 100 year ARI event (refer to Figure 18).

6.4. Floor Levels

The ground floor level for the retail development site is proposed to be at 4 mAHD, which is significantly higher than the 100 year ARI peak flood level of 2.46 mAHD and even the PMF peak flood level of 3.14 mAHD. With the exception of the main entrance off Captain Cook Drive whereby the crest level is proposed to be at 2.6 mAHD which is still above the 100 year ARI peak flood level, all other entrances are proposed at a similar level to the ground flood level (4 mAHD). Therefore, the site is not subject to any flood risk and is in compliance with Council's development controls for flooding.

6.5. Flood Impact Assessment for Development

Associated with the development are the earthworks proposed for the entrance to the retail development as well as the realignment and upgrade of the existing local drainage system from a 375 mm diameter pipe to a proposed 450 mm diameter pipe which will convey more floodwaters to the tidal channel west of Toyota Park. With the proposed works, however, adverse flood impact of up to 20 mm was found for the 100 year ARI flood event (for an area <50 m^2), as shown in Figure 14. This impact is regarded as acceptable considering it is on the side of the roadway rather than at residential dwellings where it would increase flood damages.

An impact assessment was also carried out for the Captain Cook Drive/Woolooware Road intersection upgrade whereby the existing roundabout will be modified to a signalised intersection. Table 6 indicates that the 100 year ARI peak flood levels on the grassed area east and west of the intersection will increase by up to 30 mm and 50 mm respectively as a result of the intersection upgrade. However, with Woolooware Rd raised slightly from the current crest level, inundation depths of less than 100 mm will occur for the 100 year ARI event which provides a safer passage for vehicles during flooding.

6.6. Climate Change

Climate change modelling has also been carried out as per the NSW Government guidelines issued in 2009 (Reference 13). However it should be noted that in September 2012 the NSW Government repealed mandatory compliance with the 0.4 m sea level rise by the year 2050 and 0.9 m sea level rise by the year 2100. Councils in NSW must now make their own decisions regarding the assessment of sea level rise. Sutherland Shire Council has made no formal statement that it is adopting a sea level rise assessment different to Reference 13.

A rainfall intensity increase of 10%, 20% and 30% was assessed as well as sea level rise scenarios by the year 2050 and 2100. The following runs were modelled:

- 100 year ARI event with 10%, 20% and 30% increase in rainfall;
- 100 year ARI event with sea level rise to predicted year 2050 levels (+0.4 m);
- 100 year ARI event with sea level rise to predicted year 2100 levels (+0.9 m);
- 100 year ARI event with sea level rise to predicted year 2100 levels (+0.9 m) plus 10% rainfall increase; and
- PMF event with sea level rise to predicted year 2100 levels (+0.9 m).

The results are tabulated in Table 7 for the 100 year ARI event and Table 8 for the PMF event. Flood extents of the various climate change scenarios are shown in Figure 15 and Figure 16. Overall, the results show that an increase in peak flood level of up to a maximum of 0.2 m can be expected for the development site and as such would not be an issue considering the proposed floor levels have been designed to be at a level above the PMF flood level plus any increase induced by climate change. However the relatively flat topography means that a small increase in peak level translates to a relatively large increase in flood extent.

Table 7: Impacts on 100 year ARI Peak Flood Levels (mAHD) for Climate Change Scenarios

Location	Base Case	Rainfall Increase +10%	Rainfall Increase +20%	Rainfall Increase +30%	Sea Level Rise 2050 Scenario (+0.4m)	Sea Level Rise 2100 Scenario (+0.9m)	Rainfall Increase +10% & Sea Level Rise 2100 Scenario (+0.9m)
1 Captain Cook Dr/Woolooware Rd Intersection East	1.77	+0.05	+0.09	+0.11	+0.06	+0.51	+0.51
2 Captain Cook Dr /Woolooware Rd Intersection West	2.37	+0.02	+0.03	+0.04	-	+0.03	+0.04
3 Retail Site Entrance	2.46	+0.05	+0.10	+0.15	+0.02	+0.14	+0.18
4 Captain Cook Dr	2.46	+0.04	+0.09	+0.15	+0.02	+0.14	+0.18
5 Tidal Channel	2.37	+0.09	+0.16	+0.22	+0.06	+0.22	+0.26
6 Fitness First Open Drain	1.53	-	+0.03	+0.03	+0.26	+0.75	+0.75

Notes: Difference calculated relative to Base Case (i.e. a positive number indicates an increase in flood level compared to non climate change scenario)

Table 8: Impacts on PMF Peak Flood Levels (mAHD) for Climate Change Scenarios

Location	Base Case	Sea Level Rise 2100 Scenario (+0.9m)
1 Captain Cook Dr/Woolooware Rd Intersection East	2.42	+0.05
2 Captain Cook Dr /Woolooware Rd Intersection West	2.75	-
3 Retail Site Entrance	3.12	-
4 Captain Cook Dr	3.12	-
5 Tidal Channel	3.05	+0.02
6 Fitness First Open Drain	1.66	+0.69

Notes: Difference calculated relative to Base Case (i.e. a positive number indicates an increase in flood level compared to non climate change scenario)

7. MANAGEMENT OF THE FLOOD PROBLEM

7.1. Access and Safety during Floods

Flood levels for the PMF event were assessed for the site as this relates to evacuation and overall risk to lives. It also informs the emergency plan, as the PMF flood levels largely determine whether the strategy for response to flooding should be to sit tight or to evacuate immediately.

For both existing and post-development conditions, a peak flood level of 3.2 mAHD is expected at the site in the PMF event. As this is lower than the proposed ground floor car park level (4.0 mAHD), emergency egress during a flood event is not necessary. Nevertheless, an evacuation route as indicated in Figure 21 and Figure 22 is proposed which serves as an access route from the site to high ground via Woolooware Road as well as providing access for emergency vehicles (i.e. SES, ambulance) to get to the site. As such, it is pertinent that the proposed new intersection is designed to be flood free up to the 100 year ARI event. With the existing topography of the intersection, shallow inundation depths (<100 mm) can still be expected as indicated on Figure 9 for the 100 year ARI event.

7.2. Flood Response Plan

A flood response plan should be prepared by the building management to minimise the risk associated with evacuations by providing information regarding evacuation routes, refuge areas, what to do/not to do during floods etc. Any evacuation undertaken would usually be under the direction of the lead agency, the State Emergency Services (SES). As such, it is necessary that SES be made aware of the new retail development and the local SES response planning can be prepared.

Since the development proposed is above the flood planning level, there are no further requirements in regards to the construction as there are no flood liable structures. Nevertheless, reliable pedestrian and vehicle access to a place of refuge should be maintained. Recommendations for an evacuation route have been provided in the previous section.

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Figures

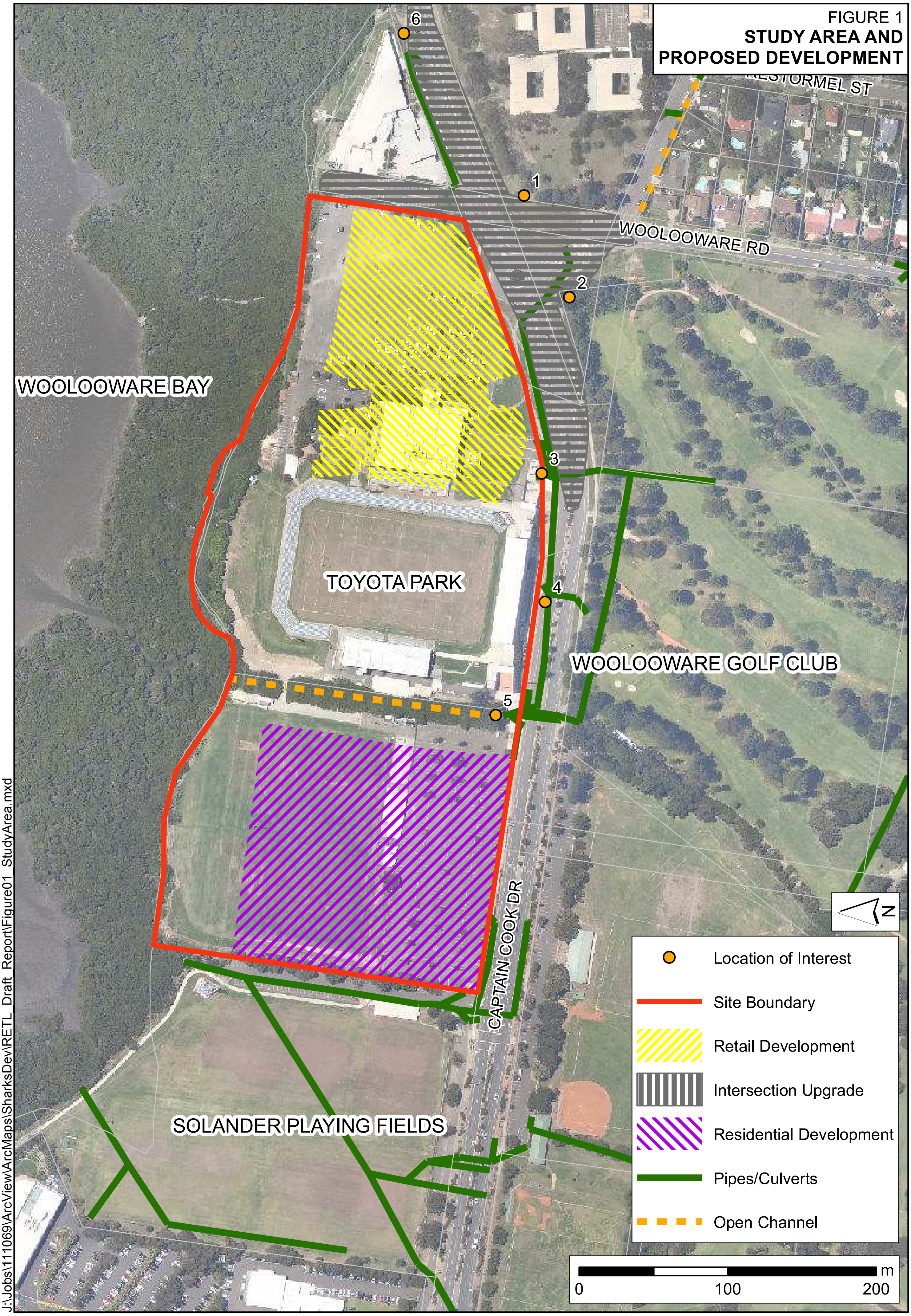
FIGURE 1
STUDY AREA AND
PROPOSED DEVELOPMENT
KESTORMEL ST
WOOLOOWARE RD
WOOLOOWARE BAY
TOYOTA PARK
WOOLOOWARE GOLF CLUB
CAPTAIN COOK DR
SOLANDER PLAYING FIELDS
1
2
3
4
5
6
N
Location of Interest
Site Boundary
Retail Development
Intersection Upgrade
Residential Development
Pipes/Culverts
Open Channel
0
100
200
m
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FIGURE 2
EXISTING TOPOGRAPHY AND SUBCATCHMENTS

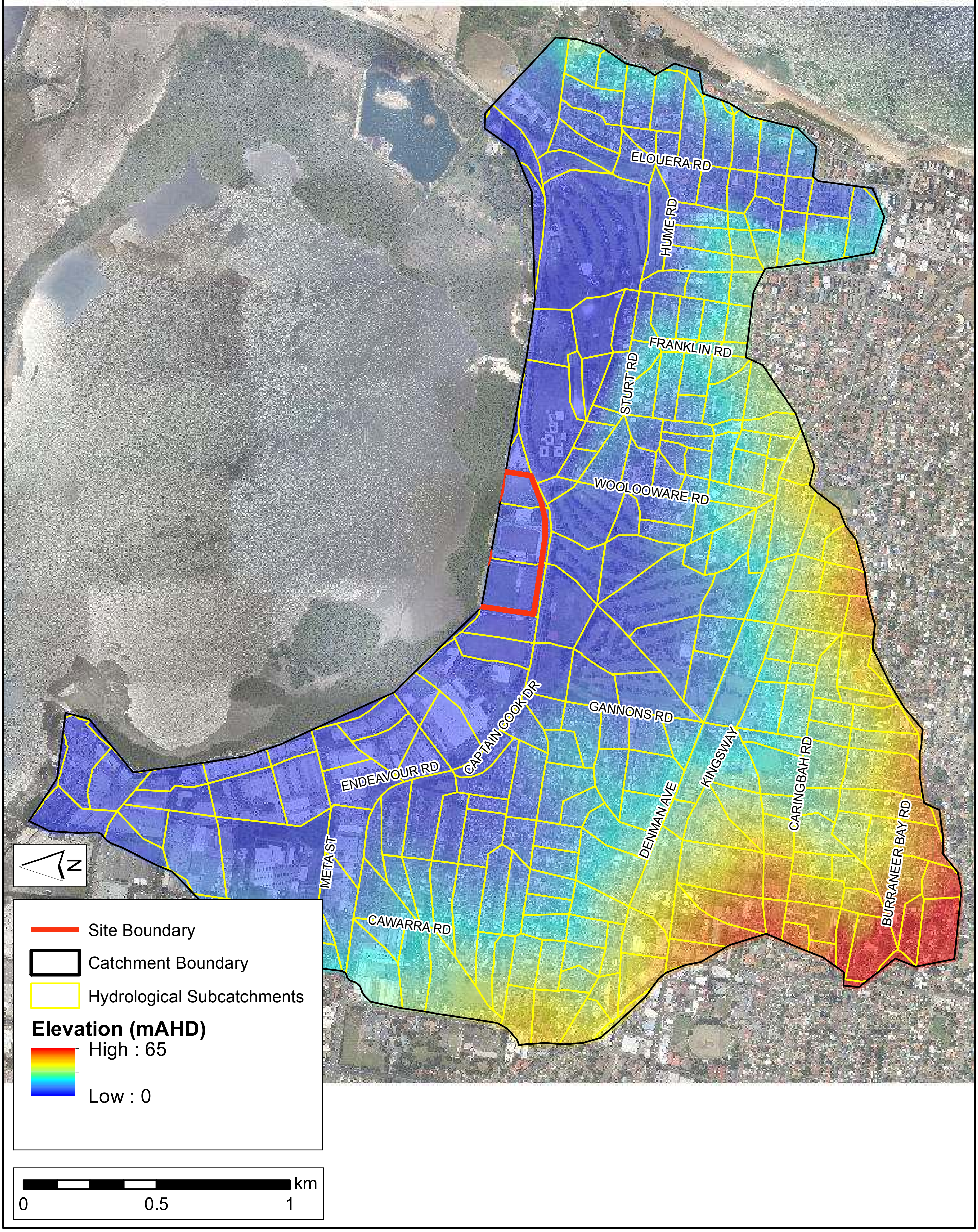

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FIGURE 3
DRAINAGE NETWORK & HYDRAULIC MODEL LAYOUT

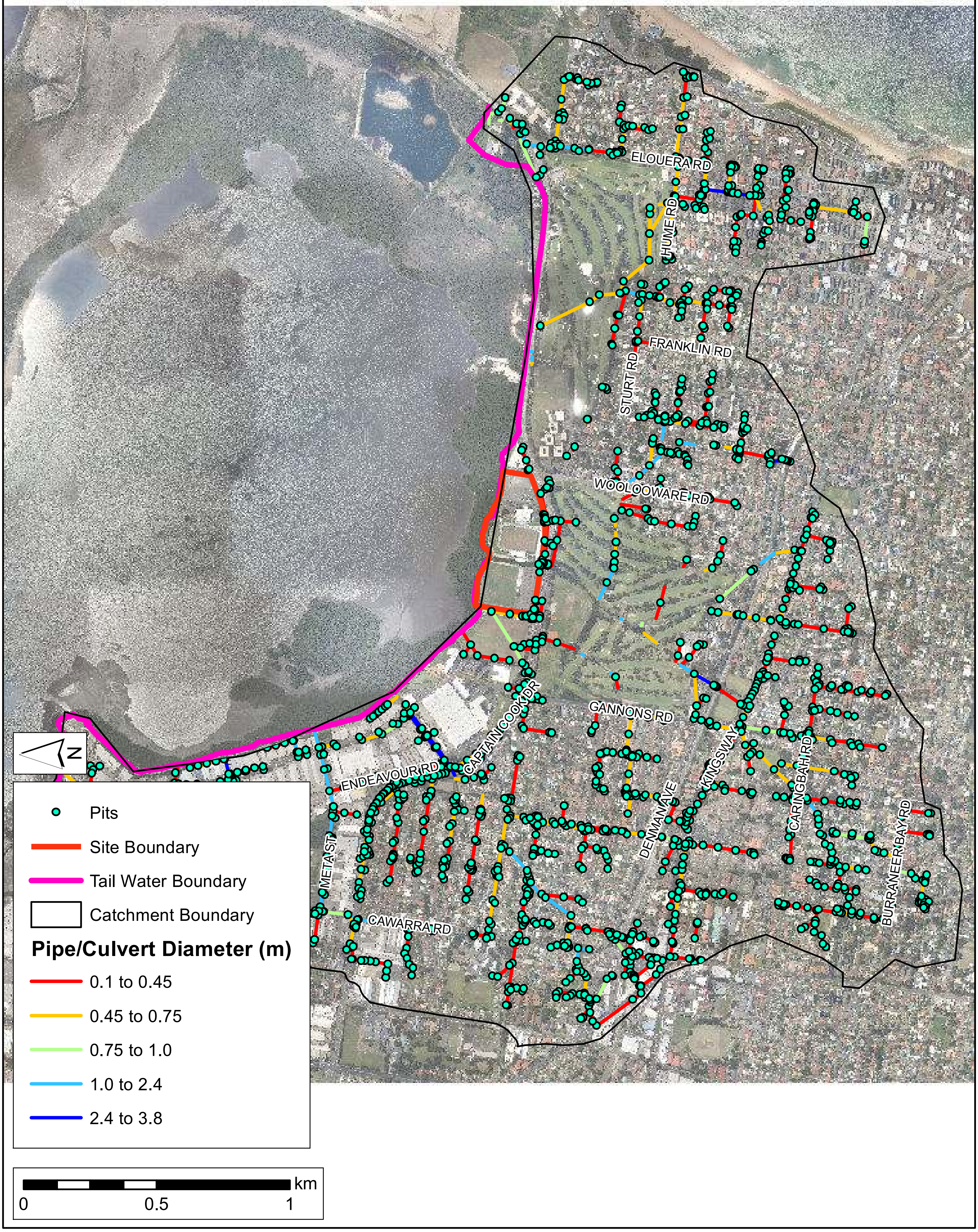

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FIGURE 4
CRITICAL DURATION MAP
100Y ARI EVENT

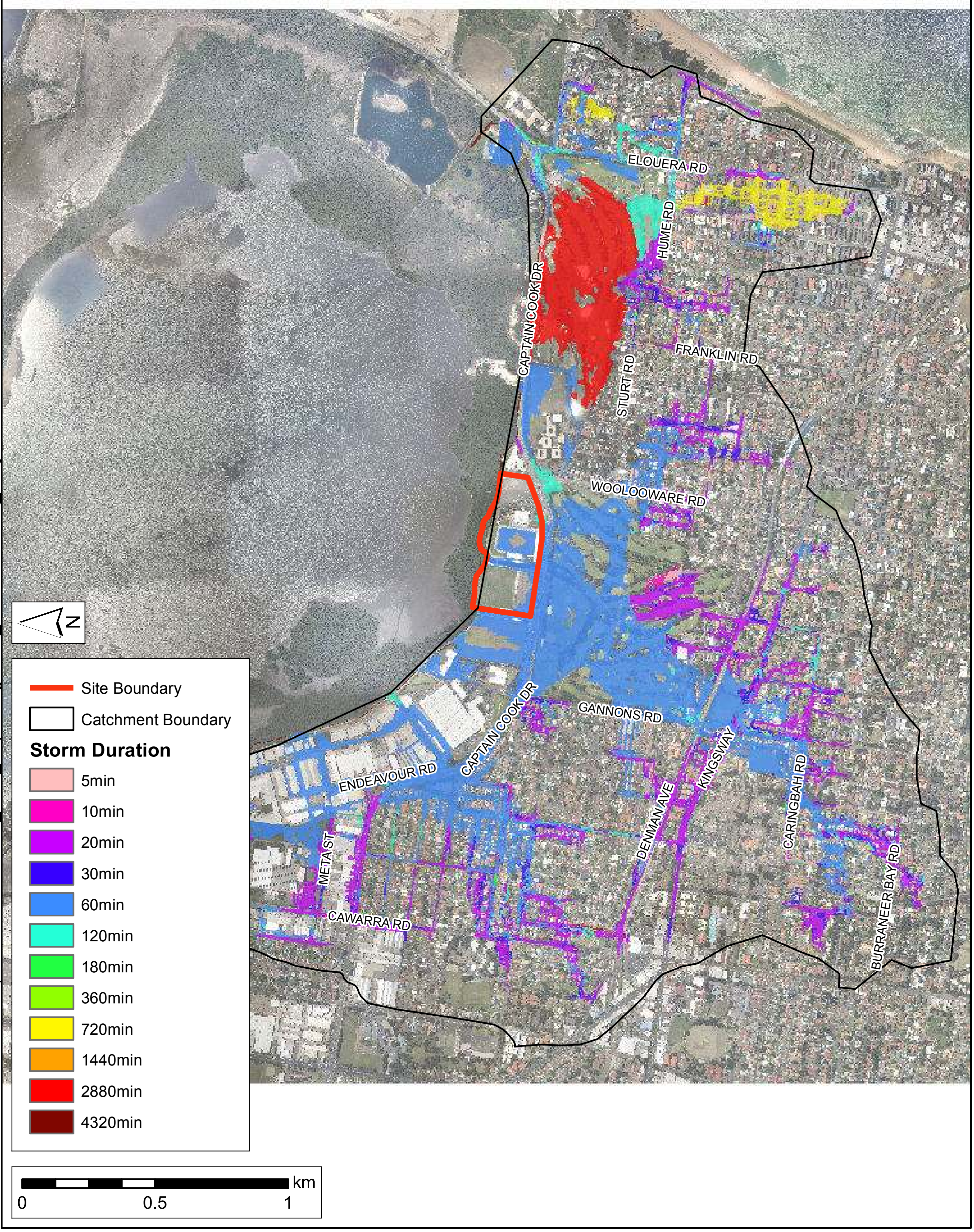

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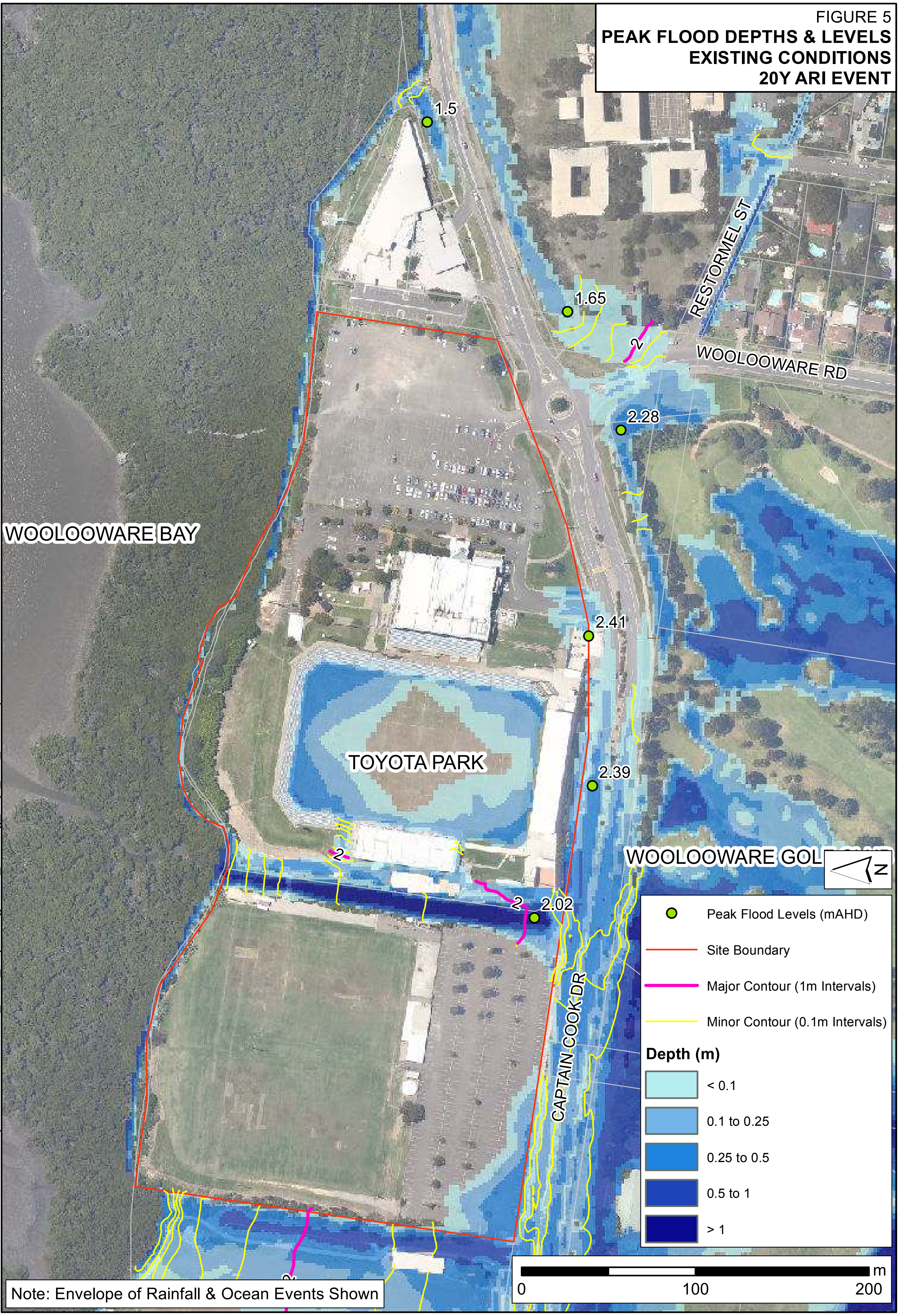
FIGURE 5
**PEAK FLOOD DEPTHS & LEVELS
EXISTING CONDITIONS
20Y ARI EVENT**



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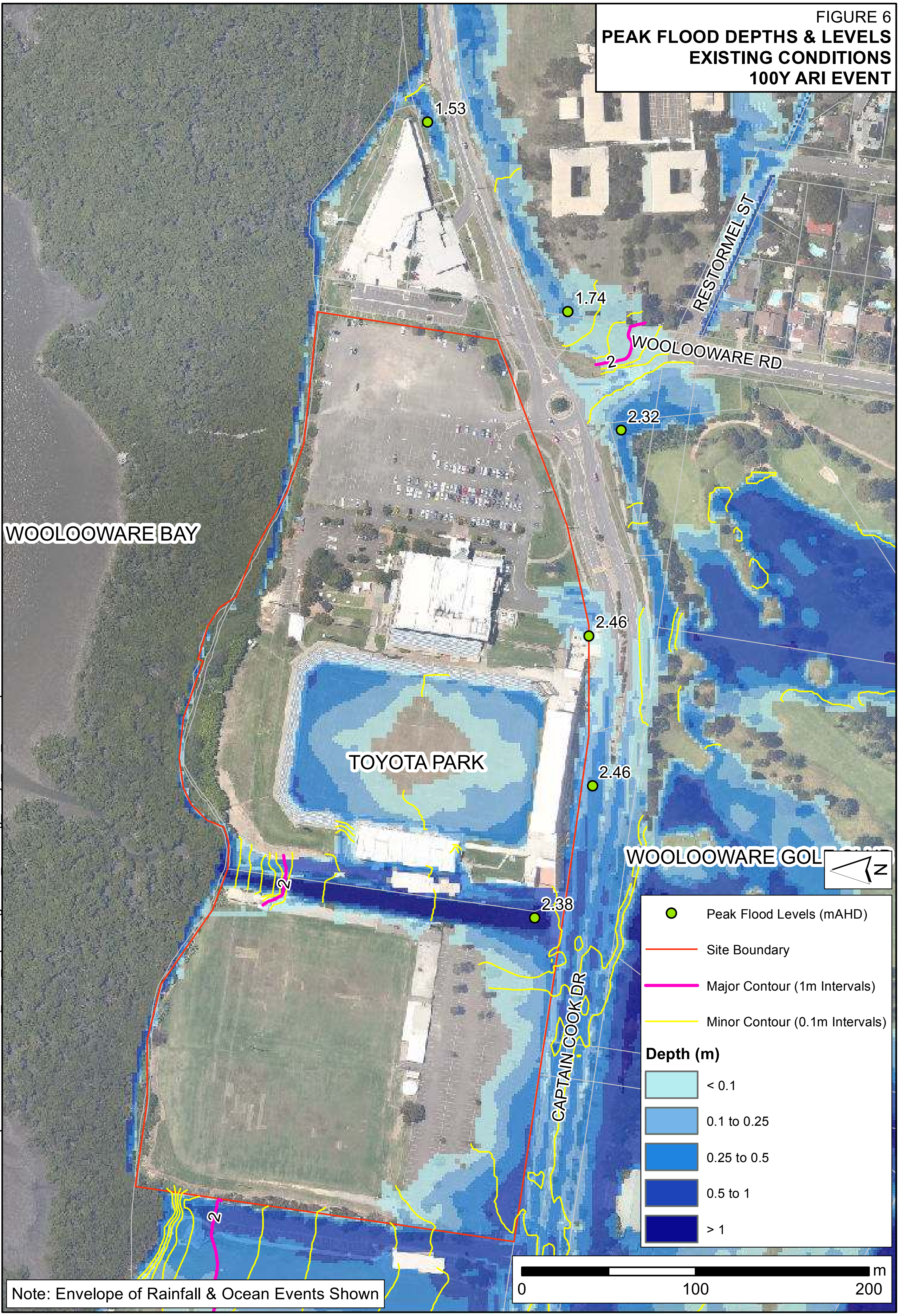

FIGURE 6
PEAK FLOOD DEPTHS & LEVELS
EXISTING CONDITIONS
100Y ARI EVENT

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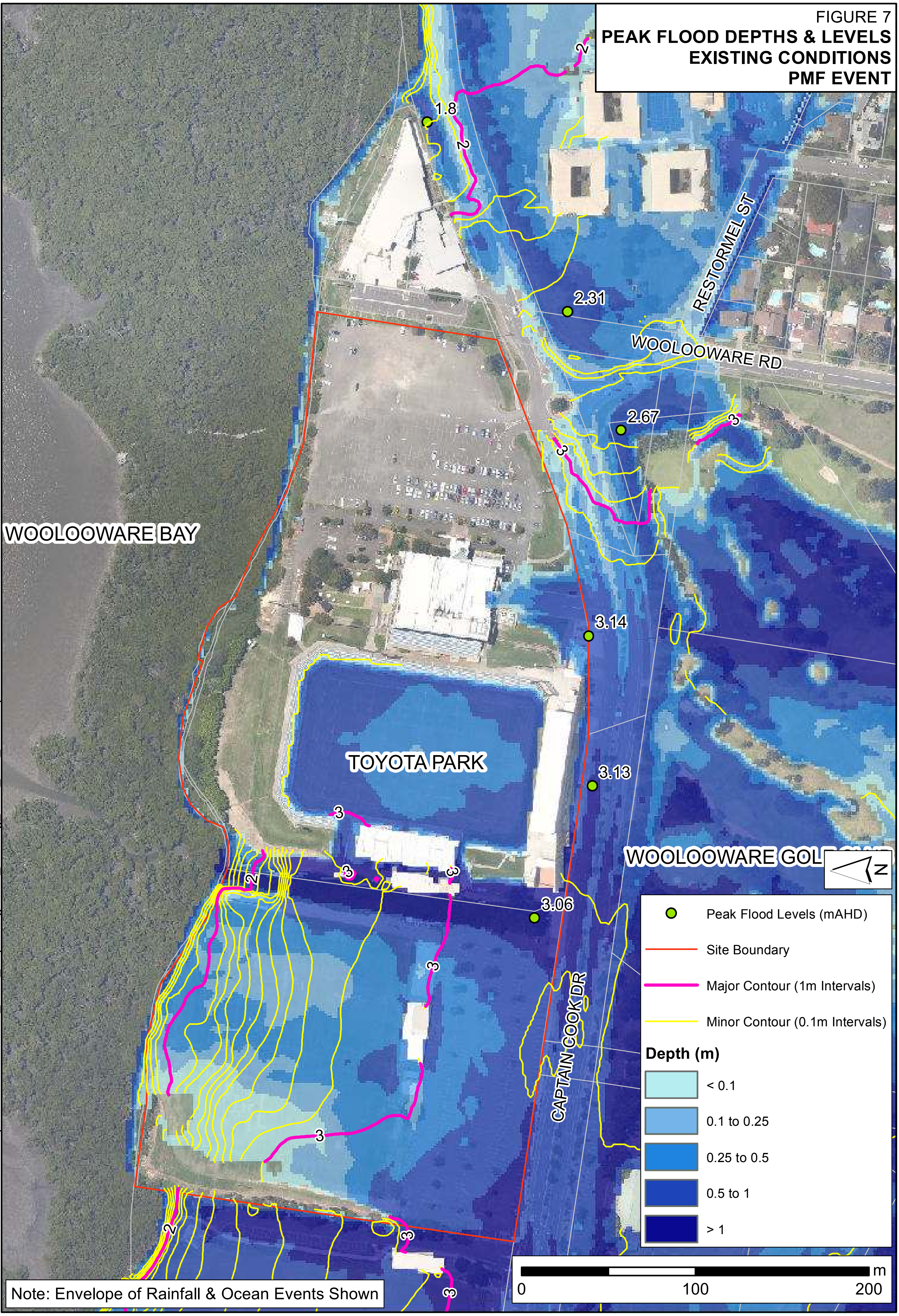
FIGURE 7
PEAK FLOOD DEPTHS & LEVELS EXISTING CONDITIONS PMF EVENT



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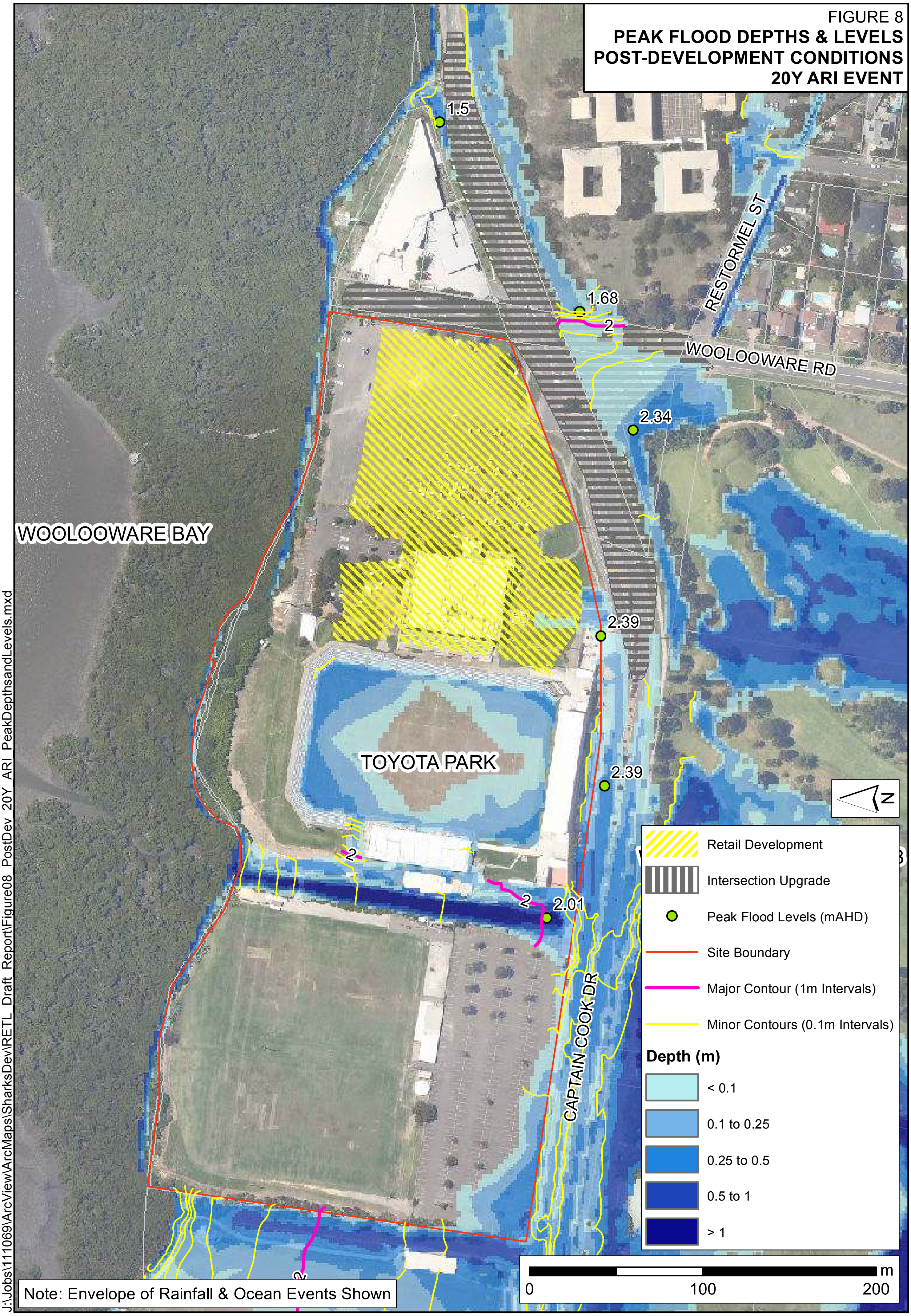
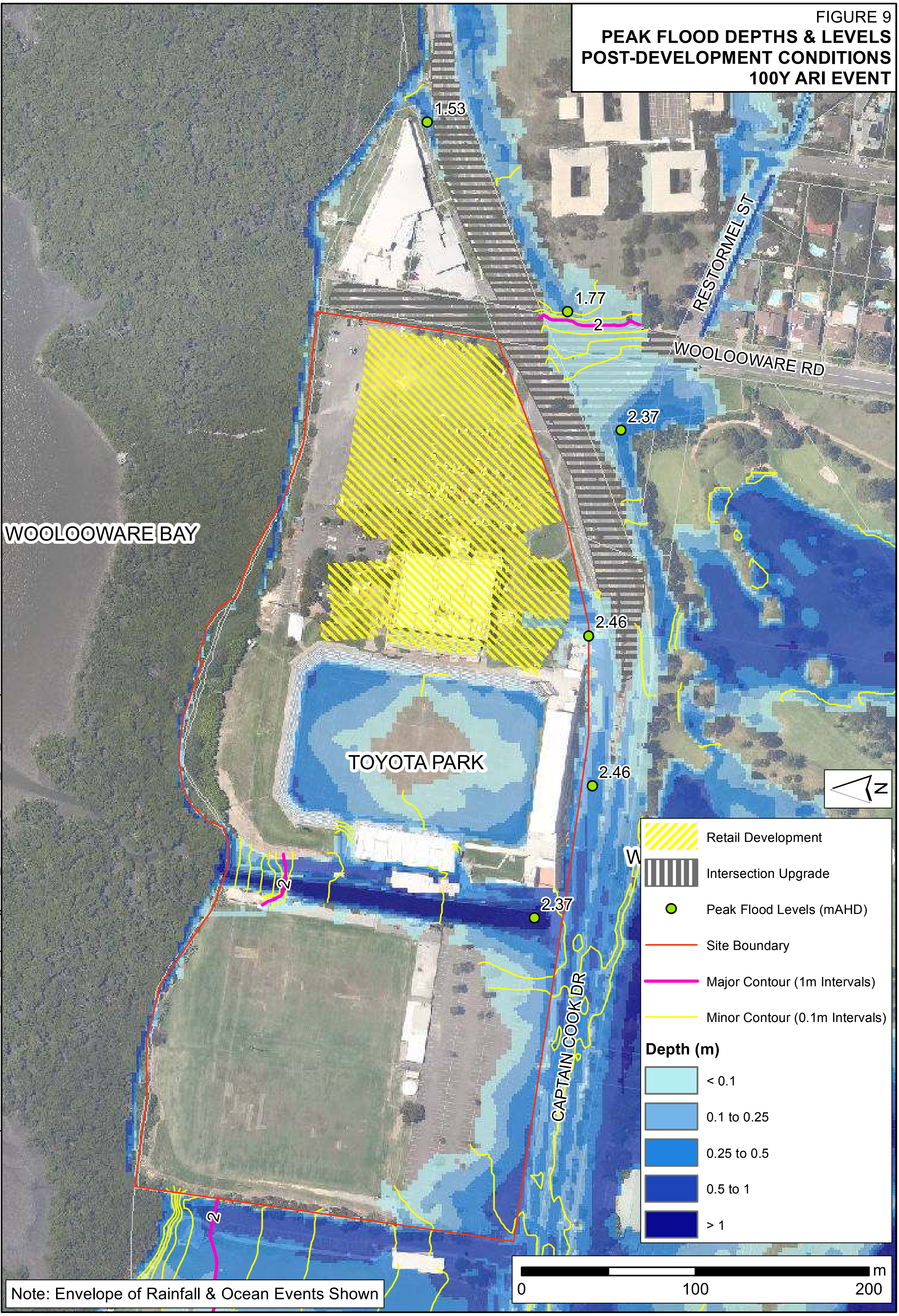


FIGURE 8
PEAK FLOOD DEPTHS & LEVELS
POST-DEVELOPMENT CONDITIONS
20Y ARI EVENT
1.5
1.68
2
2.34
2.39
2.39
2.01
RESTORMEL ST
WOOLOOWARE RD
WOOLOOWARE BAY
TOYOTA PARK
CAPTAIN COOK DR
N
Retail Development
Intersection Upgrade
Peak Flood Levels (mAHD)
Site Boundary
Major Contour (1m Intervals)
Minor Contours (0.1m Intervals)
Depth (m)
< 0.1
0.1 to 0.25
0.25 to 0.5
0.5 to 1
> 1
m
0
100
200
Note: Envelope of Rainfall & Ocean Events Shown
J:\Jobs\111069\ArcView\ArcMaps\SharksDev\RETL_Draft_Report\Figure08_PostDev_20Y_ARI_PeakDepthsandLevels.mxd

FIGURE 9
PEAK FLOOD DEPTHS & LEVELS
POST-DEVELOPMENT CONDITIONS
100Y ARI EVENT



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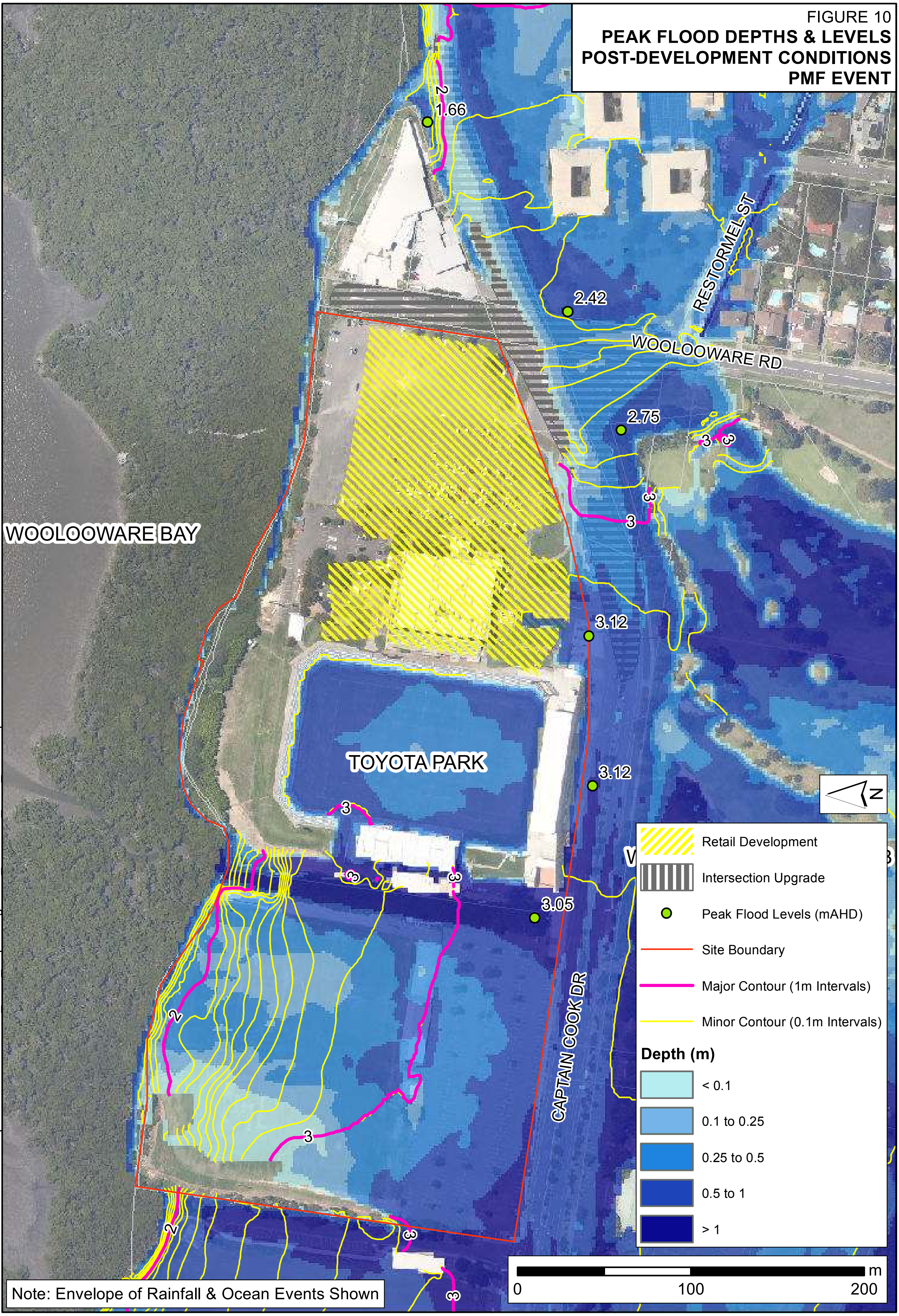

FIGURE 10
PEAK FLOOD DEPTHS & LEVELS
POST-DEVELOPMENT CONDITIONS
PMF EVENT

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FIGURE 11
PEAK FLOOD VELOCITIES
POST-DEVELOPMENT CONDITIONS
20Y ARI EVENT

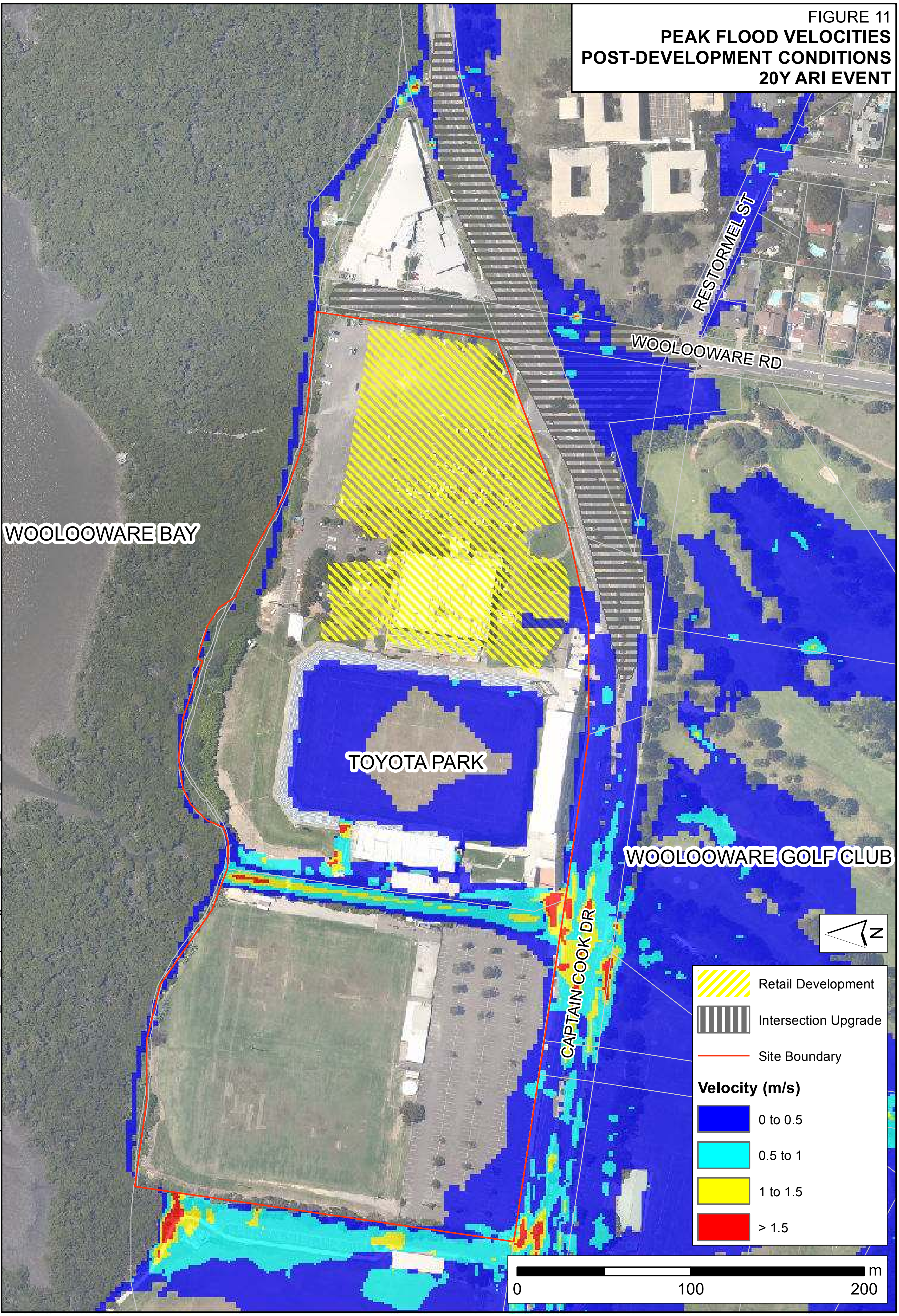

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FIGURE 12
PEAK FLOOD VELOCITIES POST-DEVELOPMENT CONDITIONS 100Y ARI EVENT

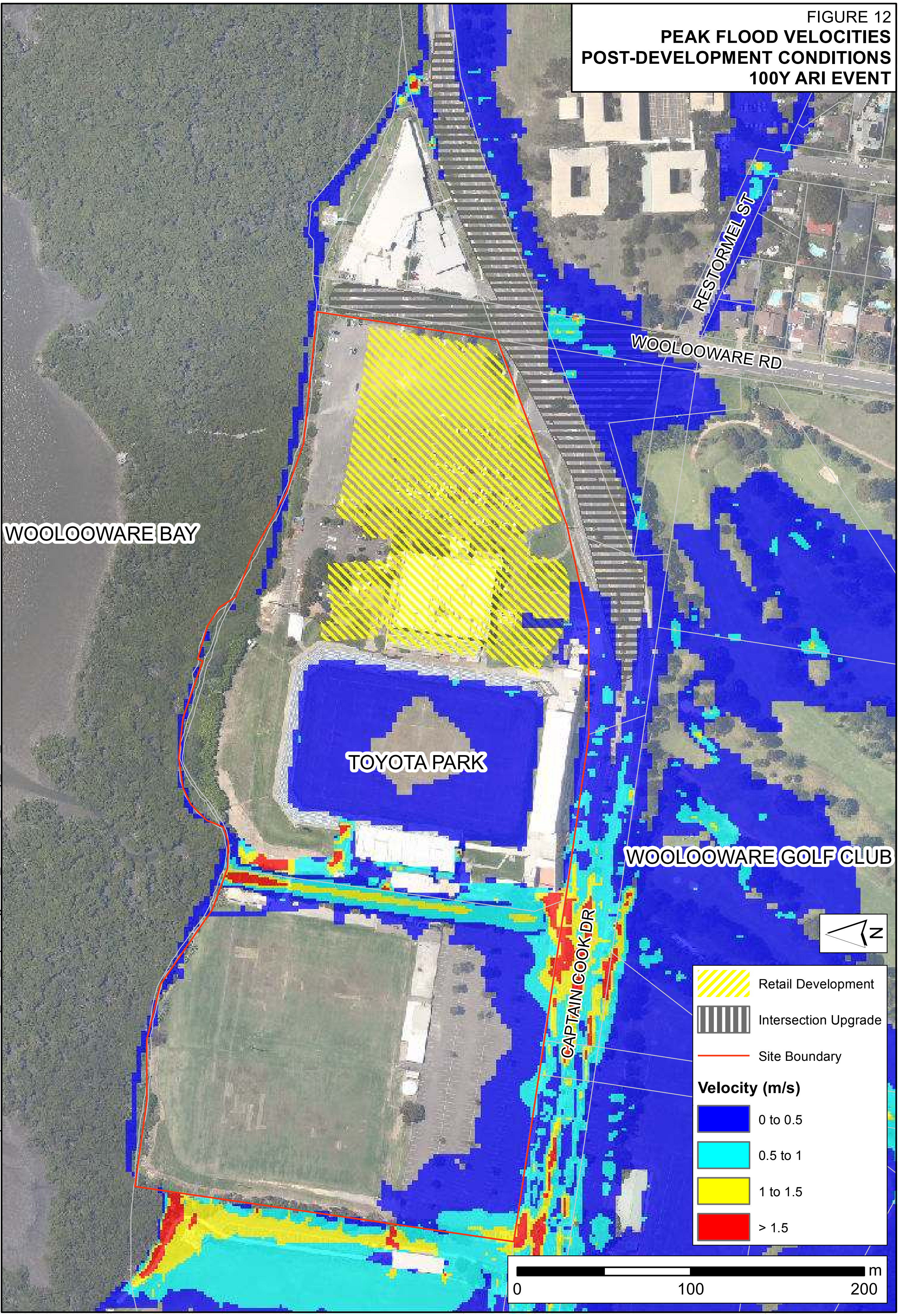

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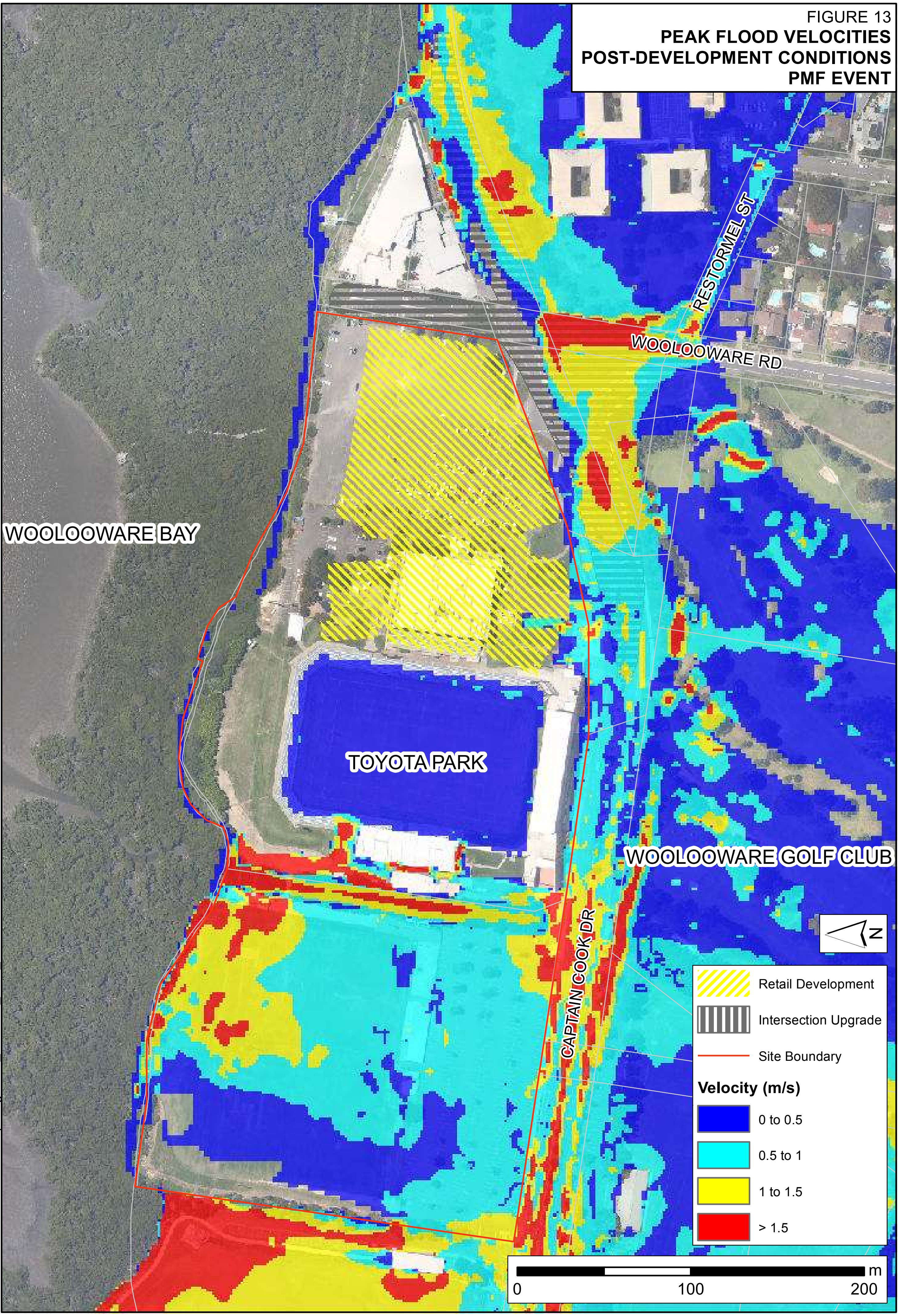
FIGURE 13
PEAK FLOOD VELOCITIES
POST-DEVELOPMENT CONDITIONS
PMF EVENT



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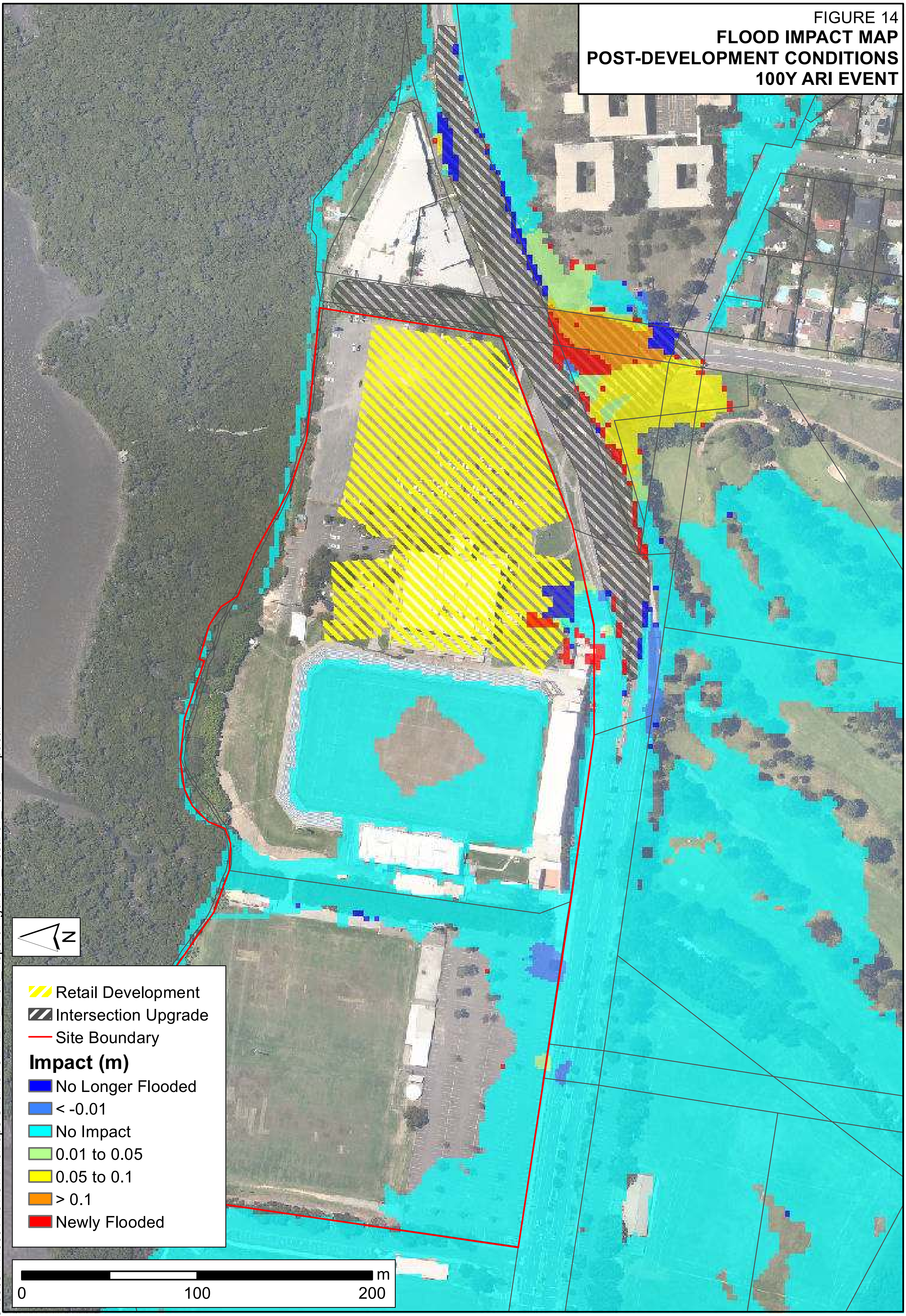

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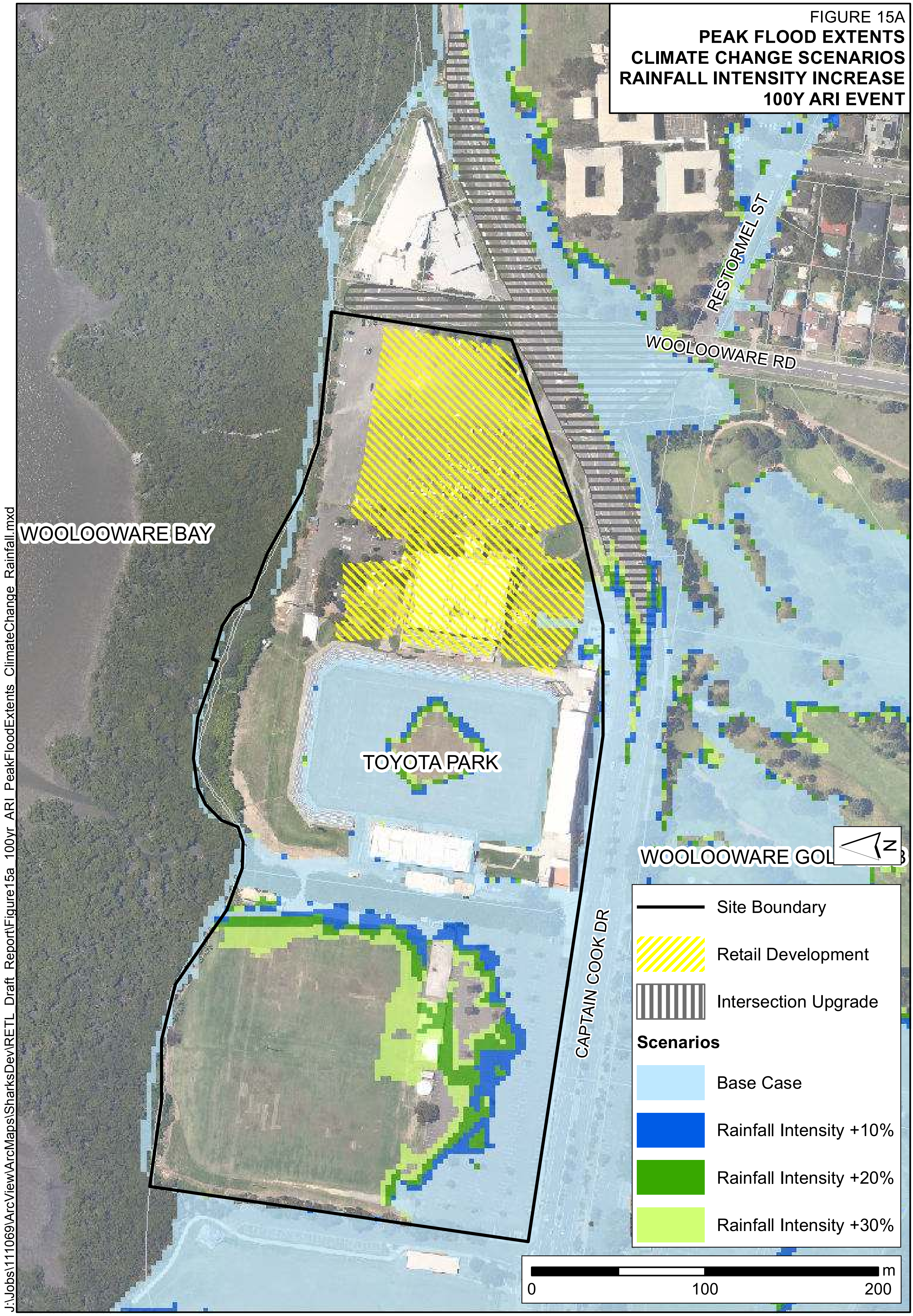
FIGURE 15A
PEAK FLOOD EXTENTS
CLIMATE CHANGE SCENARIOS
RAINFALL INTENSITY INCREASE
100Y ARI EVENT
RESTORMEL ST
WOOLOOWARE RD
WOOLOOWARE BAY
TOYOTA PARK
WOOLOOWARE GOL B
CAPTAIN COOK DR
Site Boundary
Retail Development
Intersection Upgrade
Scenarios
Base Case
Rainfall Intensity +10%
Rainfall Intensity +20%
Rainfall Intensity +30%
0
100
200
m
J:\Jobs\111069\ArcView\ArcMaps\SharksDev\RETL_Draft_Report\Figure15a_100yr_ARI_PeakFloodExtents_ClimateChange_Rainfall.mxd

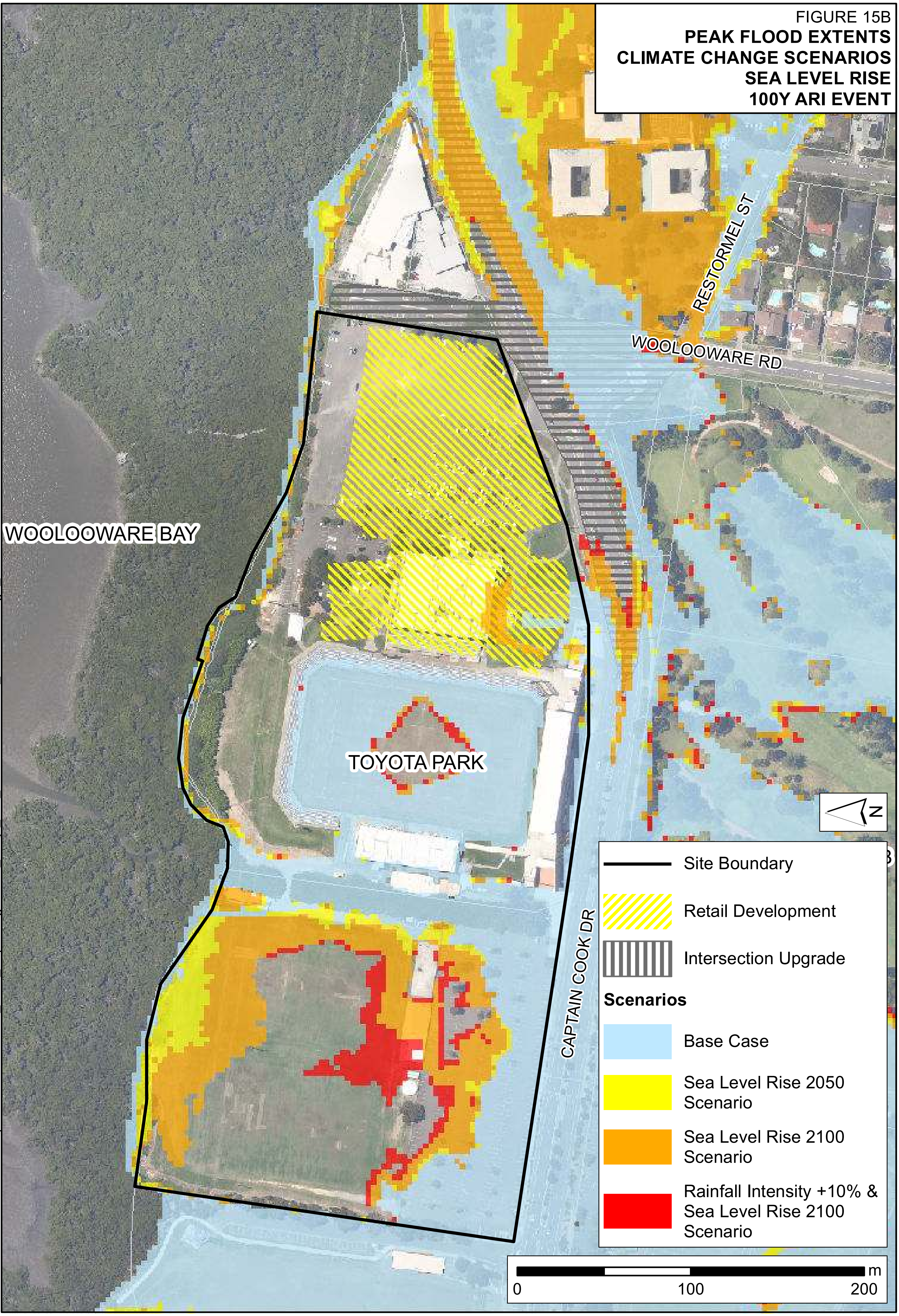

FIGURE 15B
PEAK FLOOD EXTENTS
CLIMATE CHANGE SCENARIOS
SEA LEVEL RISE
100Y ARI EVENT

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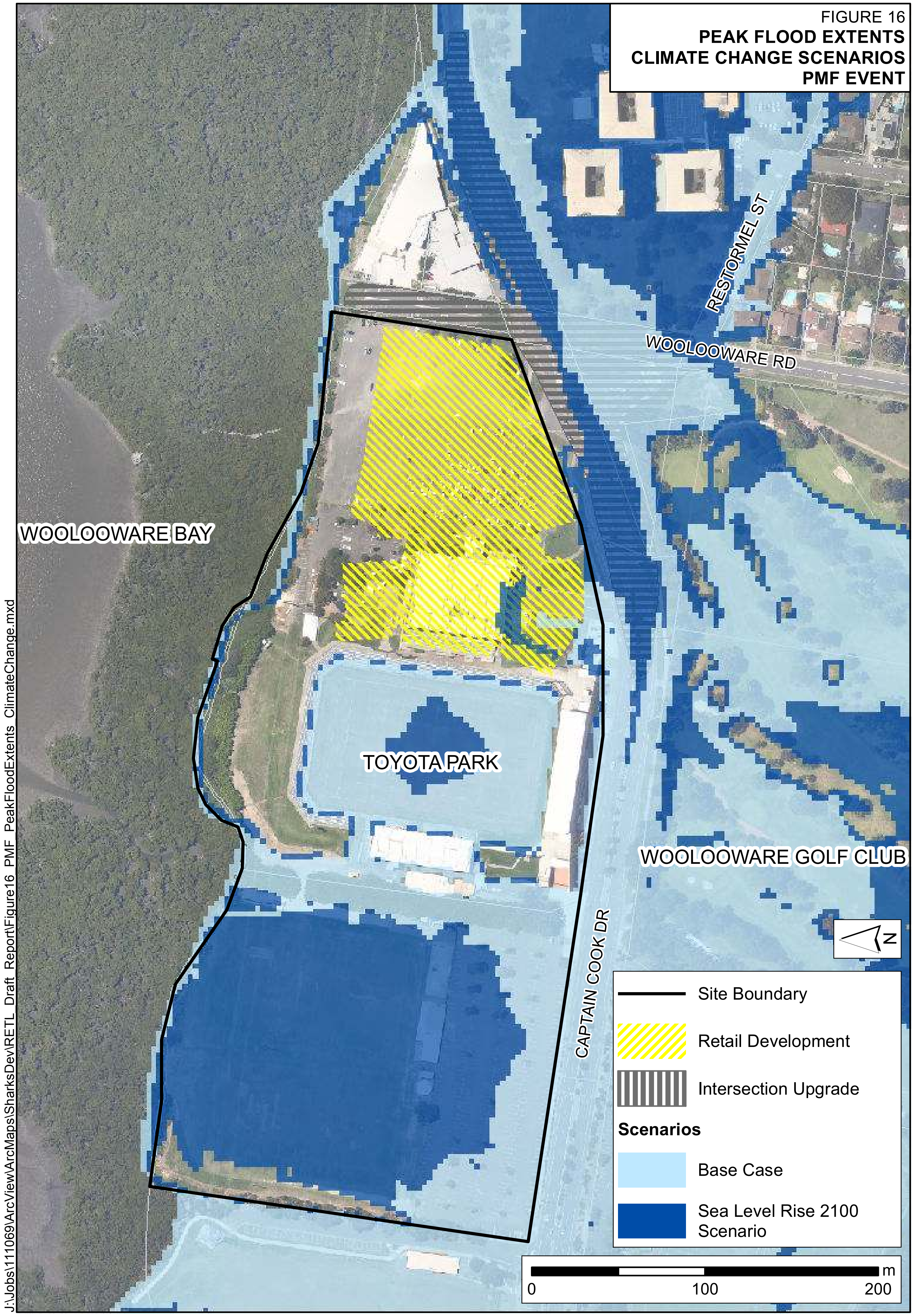
FIGURE 16
PEAK FLOOD EXTENTS
CLIMATE CHANGE SCENARIOS
PMF EVENT
RESTORMEL ST
WOOLOOWARE RD
WOOLOOWARE BAY
TOYOTA PARK
WOOLOOWARE GOLF CLUB
CAPTAIN COOK DR
N
Site Boundary
Retail Development
Intersection Upgrade
Scenarios
Base Case
Sea Level Rise 2100 Scenario
m
0
100
200
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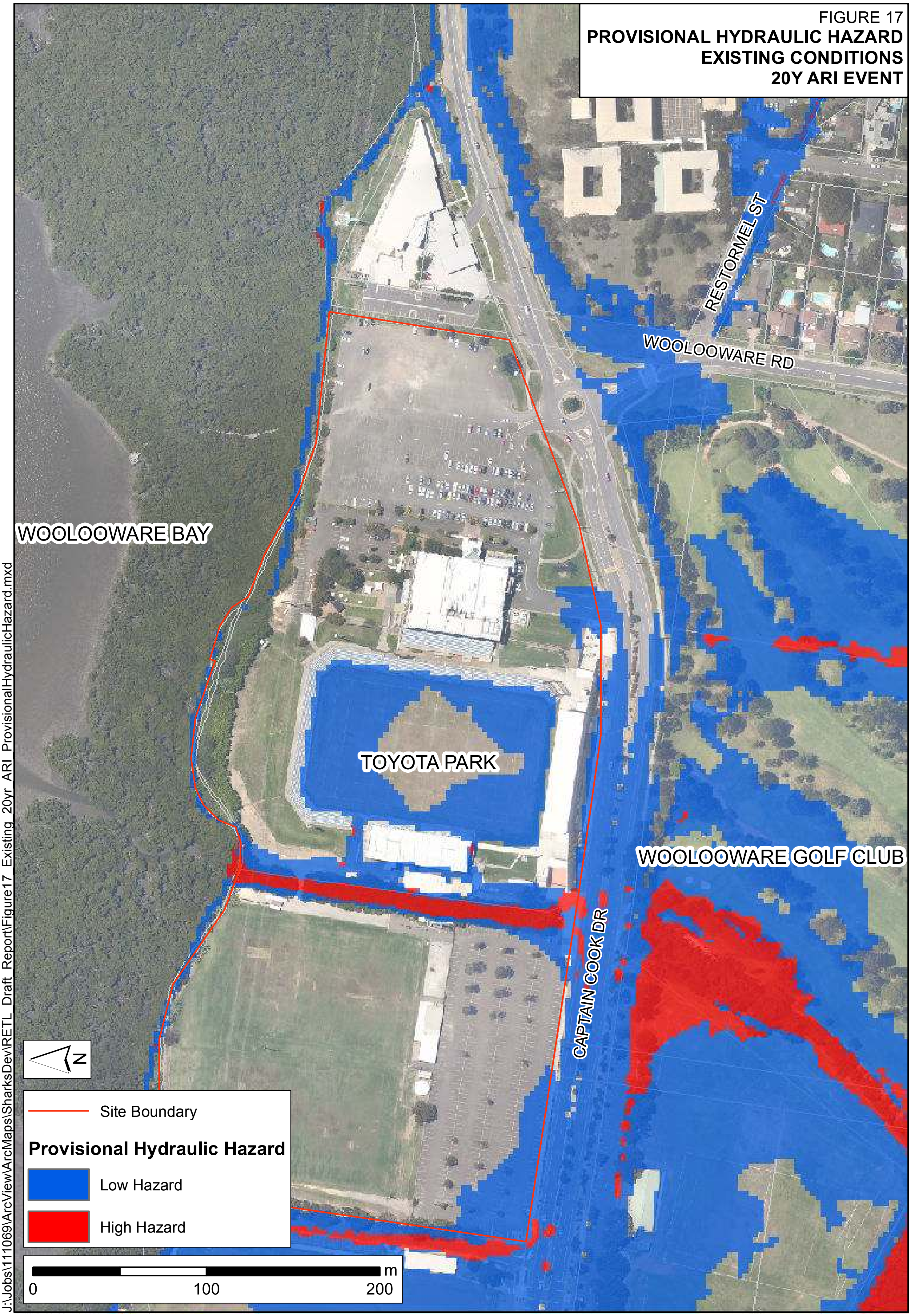
FIGURE 17
PROVISIONAL HYDRAULIC HAZARD
EXISTING CONDITIONS
20Y ARI EVENT
WOOLOOWARE BAY
RESTORMEL ST
WOOLOOWARE RD
TOYOTA PARK
WOOLOOWARE GOLF CLUB
CAPTAIN COOK DR
N
Site Boundary
Provisional Hydraulic Hazard
Low Hazard
High Hazard
m
0
100
200
J:\Jobs\111069\ArcView\ArcMaps\SharksDev\RETL Draft Report\Figure17 Existing 20yr ARI ProvisionalHydraulicHazard.mxd

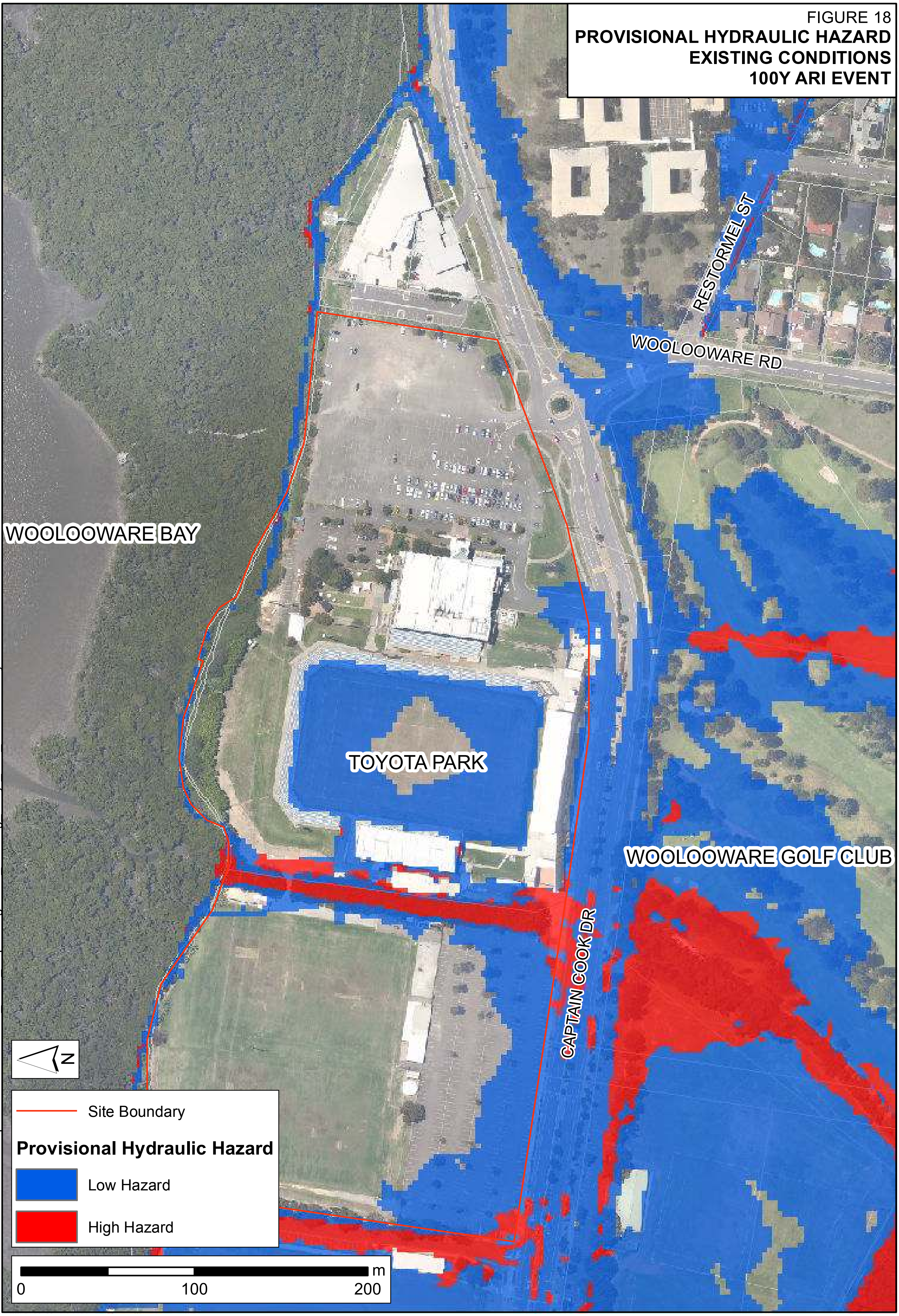

FIGURE 18
PROVISIONAL HYDRAULIC HAZARD EXISTING CONDITIONS 100Y ARI EVENT

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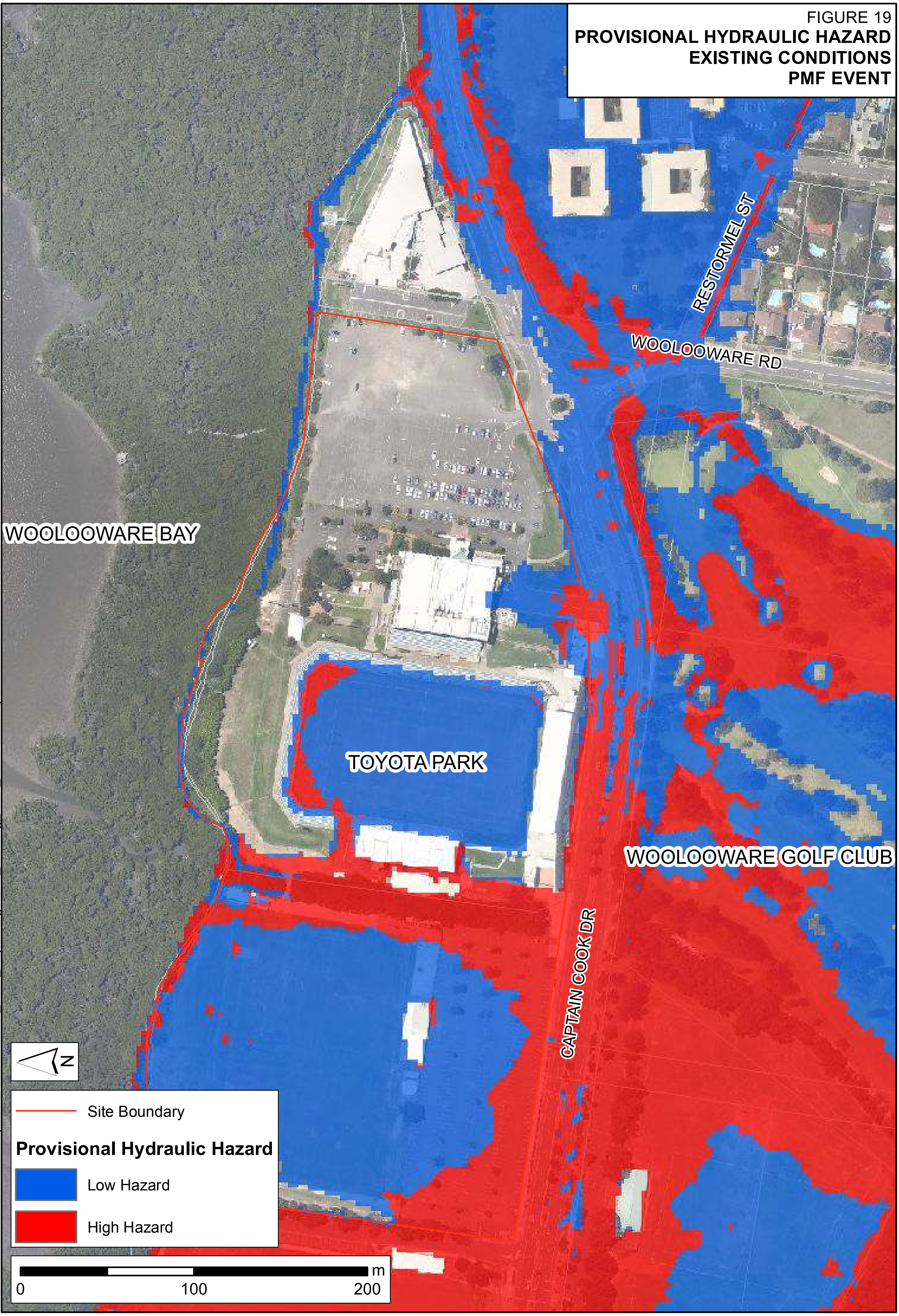
FIGURE 19
PROVISIONAL HYDRAULIC HAZARD
EXISTING CONDITIONS
PMF EVENT



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