31 - 49 MELALUKA ROAD, LEOPOLD

FLOOD IMPACT ASSESSMENT AND STORMWATER MANAGEMENT PLAN REPORT

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Reference: 18706-01
May 2019

Client: Rob Clifton

Project: 31-49 Melaluka Road, Leopold

Document: Flood Impact Assessment and Site Stormwater Management Plan

Document Control

Version	Document type	Prepared by	Reviewed by	Date Issued
V00	Flood Impact Assessment and Site Stormwater Management Plan	AP	YL / MC	March 2019
V01	Flood Impact Assessment and Site Stormwater Management Plan	MC	CM	May 2019

Project Details

Project Name	31-49 Melaluka Road, Leopold, Flood Impact Assessment and SSMP
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TGM Reference	18706-01

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Document Location

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EXECUTIVE SUMMARY

Study objectives

TGM Group has been engaged by Rob Clifton to prepare a Flood Impact Assessment (FIA) and Site Stormwater Management Plan (SSMP) for the proposed development of the property at 31-49 Melaluka Road, Leopold VIC to support its planning permit application.

The SSMP aims to demonstrate that the site can be developed using best practice stormwater management principles and techniques. The objectives will inform stormwater designs and ensure that stormwater quality and quantity targets are achieved and maintained. The FIA aims to demonstrate that the development with suggested SSMP will have no adverse impacts on flood characteristics external to the site.

Study Methodology

This study employed methods and data from the latest revision of Australian Rainfall and Runoff (ARR2016). Four models have been established for the FIA and SSMP:

- a regional distributed hydrological model of the proposed site and the catchments the site discharging to using XP-STORM
- a regional two-dimensional (2D) hydraulic model using TUFLOW, which employs the runoff hydrographs generated by the hydrological model as boundary conditions, to simulate the existing flood conditions and assess the flood impact of the proposed development
- a local hydrological model for the design of detention facilities to manage site stormwater discharges using XP-STORM
- a water quality model to predict the efficiency of the proposed treatment system in the reduction of contaminants and pollutants using MUSIC

Results

The flood impact assessment indicates that the proposed development does not generate adverse impacts external to the site during flood events.

Slight increases in flood level are observed within the swale along Melaluka Rd and the reserved area within the site, however, this does not create a change in flood hazards.

Results show that safe access and egress can be achieved and no adverse impacts will occur to downstream neighbouring properties.

In conclusion, the hydrological and hydraulic model analysis undertaken in this study has demonstrated that the proposed development can meet the **requirements** and **objectives** for site stormwater management, flood impact and **safety** access and egress during floods as seen in Figures E1 and E2.

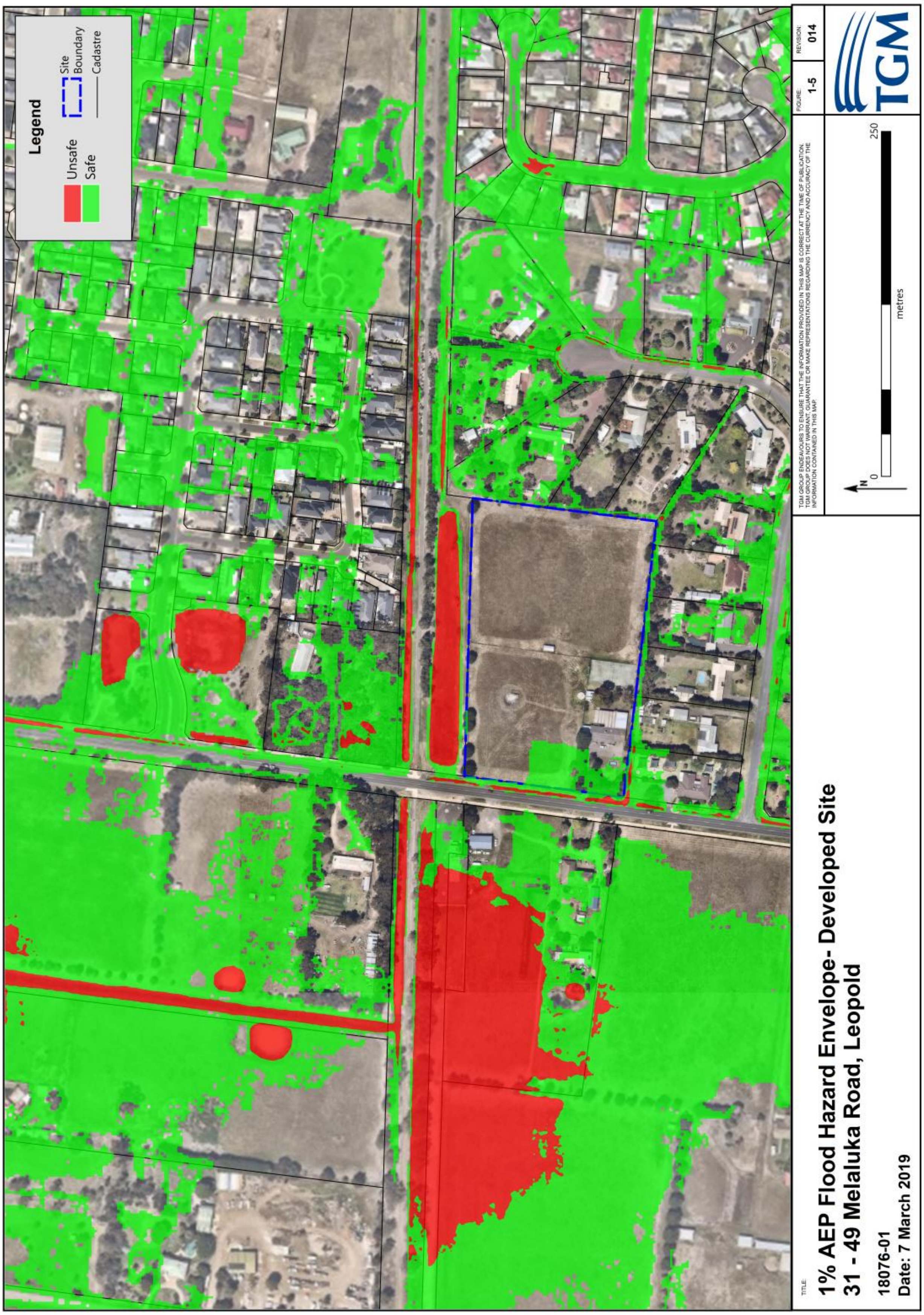

Figure E1: Developed Conditions - 1% AEP Hazard Envelope

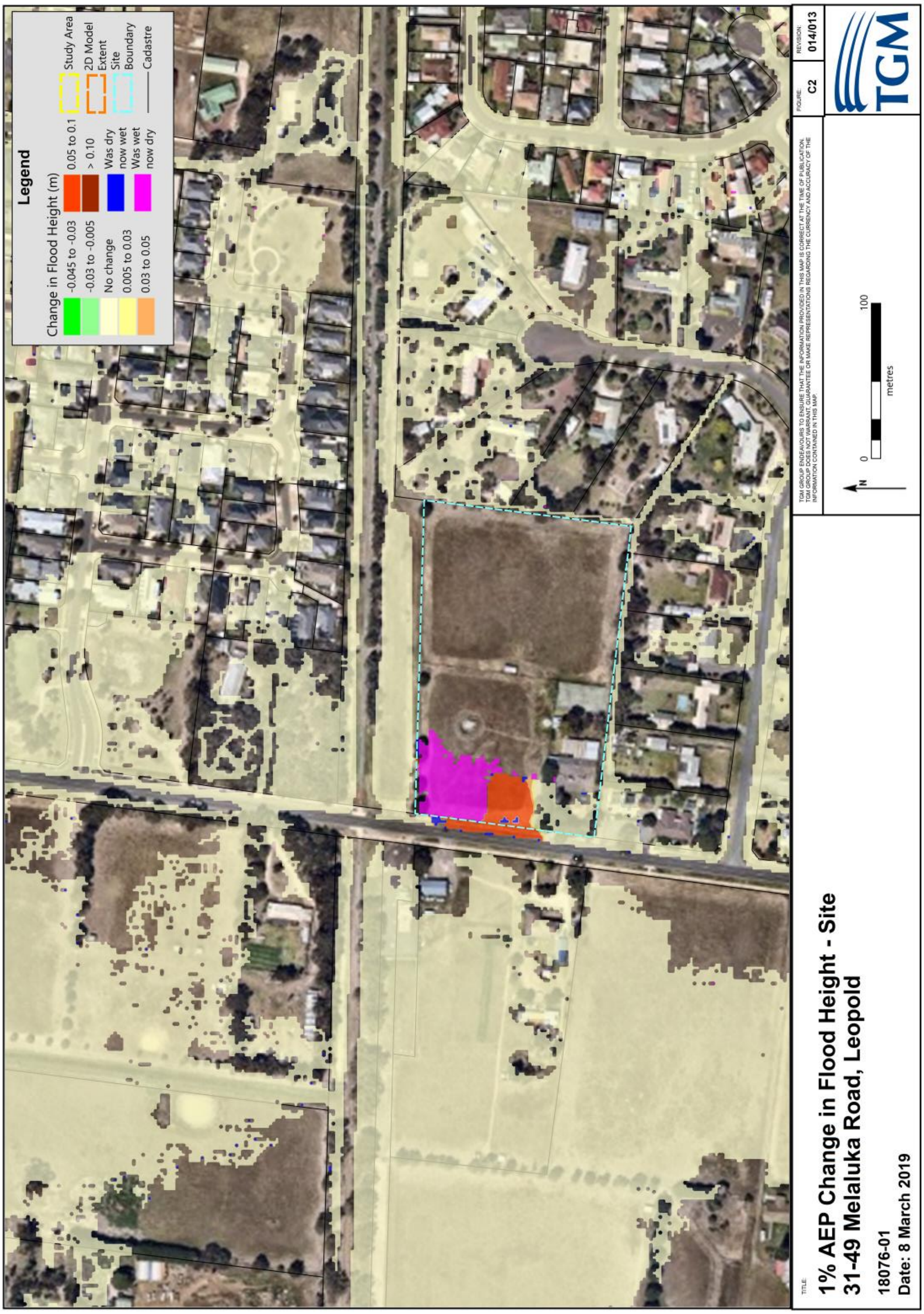

Figure E2: Change in Flood Levels - 1% AEP Impact Map

Contents

List of Tables

List of Figures

1. INTRODUCTION

TGM Group were engaged by Rob Clifton to prepare a Flood Impact Assessment (FIA) and Site Stormwater Management Plan (SSMP) for the proposed development of the property located at 31-49 Melaluka Road, Leopold VIC to support a planning permit application.

The following report details the analytical process adopted in this study and design outcomes. The analytical process includes the development of:

- a regional distributed hydrological model of the proposed site and the catchments the site is discharging to using XP-STORM;
- a two-dimensional (2D) hydraulic model using TUFLOW, which employs the runoff hydrographs generated by the hydrological model as boundary conditions, to simulate the existing flood conditions and assess the flood impact of the proposed development;
- a local hydrological model for the design of detention facilities to manage site stormwater discharges;
- and a water quality model to predict the efficiency of the proposed treatment system in the reduction of contaminants and pollutants

The flood extent for existing conditions has been defined in this study for 50%, 20%, 10%, 5%, 2%, and 1% AEP flood events. The existing flood characteristics were used to inform the proposed development layout and SSMP; and to provide a 'base-case' for the analysis of the FIA.

The following study employed methods and data from the latest revision of Australian Rainfall and Runoff (ARR2016), including improved Intensity Frequency Duration (IFD) curves and the Regional Flood Frequency Estimator (RFFE) that are based on 30 years of additional rainfall and streamflow data; and ensembles of storm burst rainfall temporal patterns.

2. OBJECTIVES

Establishing the existing flood conditions allows an understanding of the availability of developable land and identification of regional stormwater constraints associated with the development site. The defined predeveloped (existing) flood characteristics will form the ‘base-case’ for the FIA analysis of the proposed site development.

The objective of this SSMP is to demonstrate that the site can be developed using best practice stormwater management principles and techniques. This will enable the subdivision to meet the conditions and requirements set in the planning permit for stormwater management. The objectives will inform stormwater designs and ensure that stormwater quantity targets are achieved and maintained.

Specific objectives are detailed below.

2.1 Existing Flooding Objectives

The overriding objectives of the flood investigation are to –

- Prepare existing flood mapping for designated range of storm events;
- Establish the predeveloped ‘base case’ flood characteristics for the site;
- Identify the associated flood risk.

2.2 Site Stormwater Objectives

The site stormwater objectives are:

1. Best Practice reductions for Water Quality
    - 80% reduction in Suspended solids (SS)
    - 45% reduction in total nitrogen (TN)
    - 45% reduction in total phosphorus (TP)
    - 70% reduction in gross pollutants (GP)
2. No-worsening stormwater peak discharges
    - Ensure pre-development site discharges are maintained
3. Storage for regional flood
    - Provide enough reserve area (storage) to accommodate regional flood

2.3 Regional Stormwater Objectives

Development of the site must not have an adverse impact on the surrounding and downstream area during the regional flood events up to and including an event with a 1% Annual Exceedance Probability (AEP).

The impact of the development will be assessed in regards to the following:

- Flood extents – No worsening of flood extents;
- Velocities and flow characteristics – Velocity-Depth product must not exceed safety limits for people and vehicle access;
- Cumulative flooding impact – No worsening of overall flood impacts.

3. STUDY AREA

3.1 Existing Site Context

The proposed site is 31-49 Melaluka Road, Leopold where a Combined Rezoning and Subdivision Application is to be submitted to the City of Greater Geelong Council (COGG) for assessment.

At present, the site is within the Low Density Residential Zone (LDRZ1), surrounded by Farming Zone (to the west), Public Park & Recreation Zone and General Residential Zone (to the north, west and south), as it can be seen in Figure 3.1.

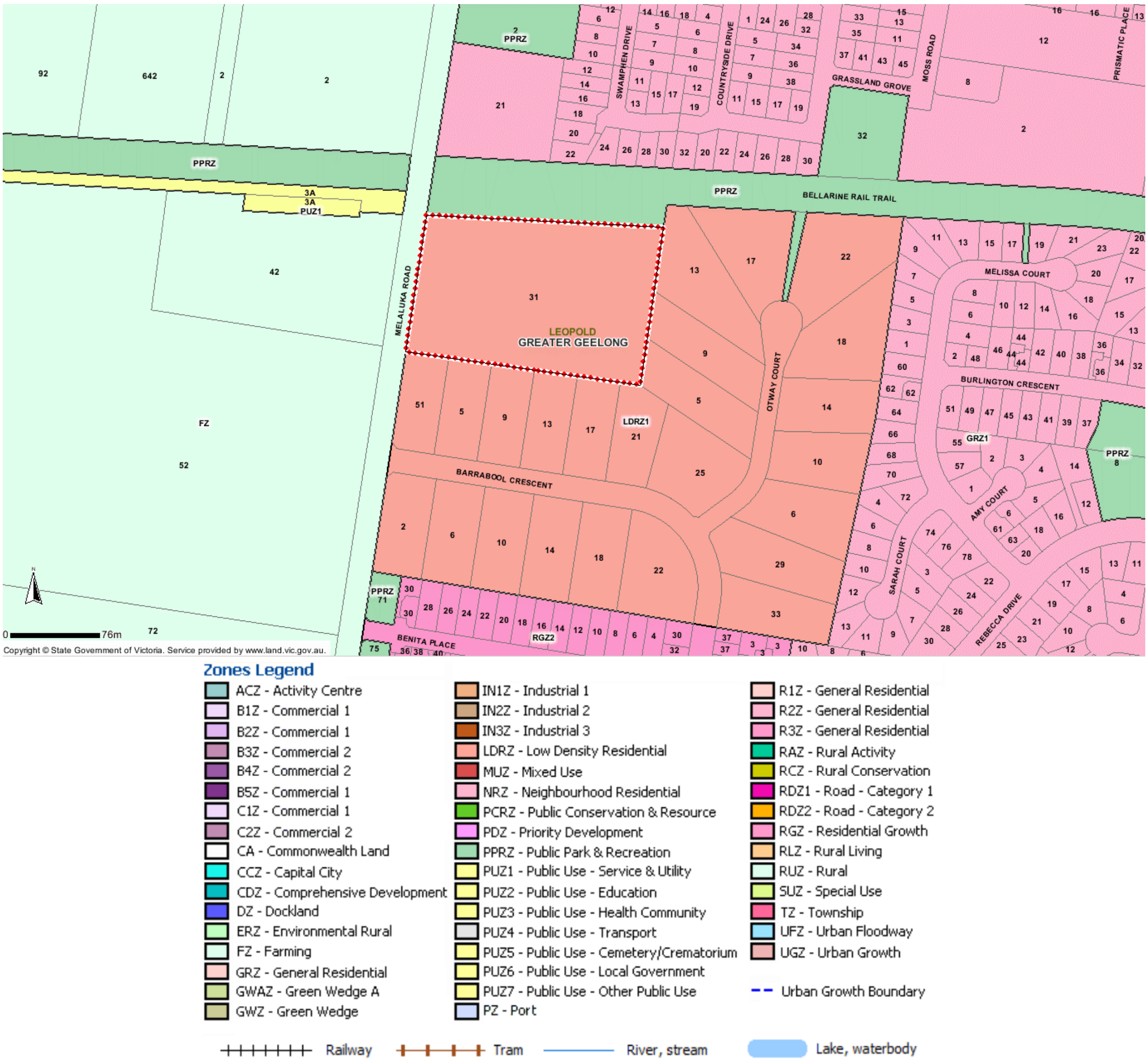

Figure 3.1: The site and Existing Planning Zone Context[1]

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[1] Planning Maps Online, http://services.land.vic.gov.au/landchannel/jsp/map/PlanningMapsIntro.jsp. Accessed on 16 Jan 2019.

3.1.1 Sensitive area

TGM is aware of the childcare - "Leopold World of Learning", located south of the subject site, as depicted in Figure 3.2.

Special attention will be paid to the change in flood surrounding the childcare and the post-development status will be addressed in Section 7.3.5.

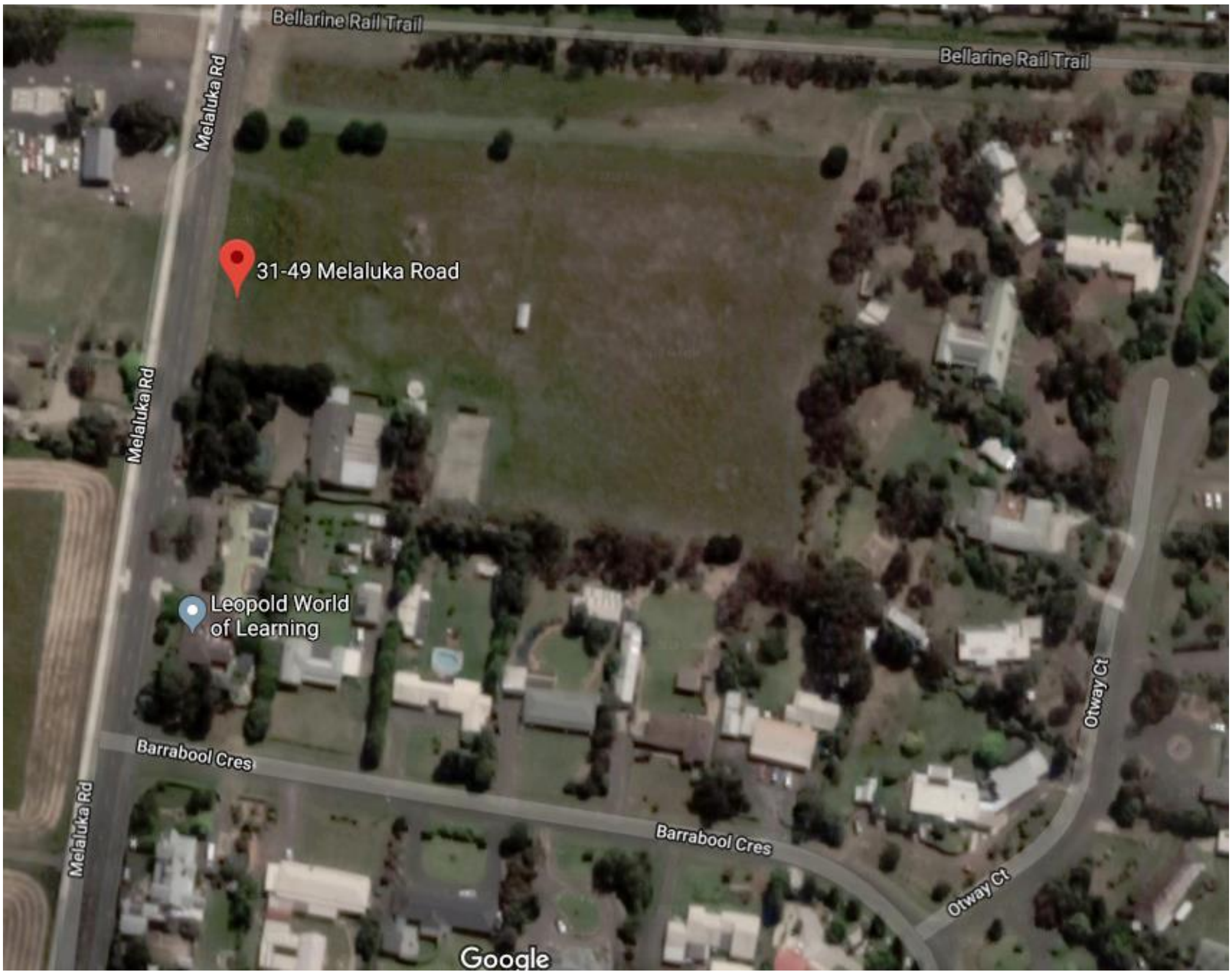

Figure 3.2: Location of the childcare - "Leopold World of Learning" (Source: Google Maps 2019)

3.2 Developed Site Context

The proposed development requires a planning application to the City of Greater Geelong Council (COGG) for assessment. It proposes to combine a conceptual subdivision (no plans are provided as yet) with the rezoning of the site from Low Density Residential Zone (LDRZ1) to General Residential Zone (GRZ1).

In an initial meeting between COGG and TGM on 20 Sep 2018, COGG indicated that the site could be rezoned to the general residential zone given that an appropriate design and stormwater management are provided.

Specifically, the site is known to be subject to flooding according to the flood map provided by COGG shown in Figure 3.3.

Figure 3.3: Existing conditions 1% AEP Flood Extent, City of Greater Geelong

However, the predicted 1% AEP flood extent defined in the COGG mapping was generated by a flood model applying the rainfall and flood estimation processes prescribed in Australian Rainfall and Runoff 1987 (ARR87), which has now been superseded.

TGM Group has undertaken development and drainage works throughout the Greater Leopold area and has an intimate knowledge of flood and drainage characteristics throughout the region. The mapping presented by COGG does not appear to accurately account for the stormwater runoff contribution from the built up eastern and southern catchment areas of Leopold.

Therefore, the hydrologic and hydraulic analysis of the contributing catchment area affecting the site needs to be remodelled in accordance to the new methodologies outlined in Australian Rainfall and Runoff 2016 (ARR2016).

The ARR2016 incorporates an additional 30 years of data within resources to support the estimation of stormwater runoff including a Regional Flood Frequency Estimator (RFFE) for use in rural areas, ensembles of updated peak rainfall bursts and full volume storms, and continuous simulation.

An appropriate design is expected to mitigate the flooding on the property. The SSMP will ensure that the water quality meets the requirements and the proposed development will have no negative impact on the neighbour properties in terms of water quantity.

This study will focus on generating a suitably detailed flood model to simulate the hydrological and hydraulic processes within the catchments, incorporating current best available topographical and GIS data, and full adoption of ARR2016 computational processes and techniques for rainfall and flood estimation.

4. REGIONAL HYDROLOGICAL MODEL

The hydrological analysis was simulated with XP-STORM 2017.2 by applying the rainfall-runoff, Laurenson routing, and one-dimensional (1D) hydraulic channel techniques based on the ARR2016 methodology. XP-STORM provides features to efficiently interface with the ARR Data Hub and Bureau of Meteorology (BOM) to obtain IFD and rainfall data to generate temporal patterns for a range of event probabilities.

Two (2) regional hydrological models were set up for the greater study area using XP-STORM:

1. A lumped hydrological model of the study catchment - used for model parameter sensitivity analysis; and
2. A distributed hydrological model (71 nodes) of the study catchment - used to compute the stormwater hydrographs used as inflow boundary conditions in the 2D hydraulic model.

4.1 Sub-areas Delineation

Delineation of sub-areas was carried out by applying a mathematical algorithm called TauDEM[2,3] (Terrain Analysis Using Digital Elevation Models) to topographic data sets.

TauDEM is a suite of Digital Elevation Model (DEM) tools for the extraction and analysis of hydrologic information from topography as represented by a DEM. Please refer to Section 5.1 for a detailed description of the DEM used in this study.

TauDEM provides the distinctive 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 watersheds and sub-watersheds draining to each stream segment.

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

4.1.1 Rain on Grid Analysis

A Rain-on-Grid (RoG) analysis was undertaken at this stage of the project to assist in definition and confirmation of the contributing catchment area, preferred flow paths and storage within the catchment.

The RoG model applied rainfall depths generated for the 1% AEP 6 hour and 1 hour duration events, directly to the analytical grid surface referencing the LiDAR DEM of the greater Leopold area.

Hydrological losses were simulated by applying a soil infiltration (Hortons) characteristics to each cell based on losses associated with land use characteristics. The RoG 2d-hydraulic model did not include embedded 1d elements.

The RoG flood results can be seen in Figure 4.1 for the 1% AEP flood envelope for the 1 hour and 6 hour duration event.

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[2] http://hydrology.usu.edu/taudem/taudem5/index.html .
[3] Tarboton, D. G., (1997), "A New Method for the Determination of Flow Directions and Contributing Areas in Grid Digital Elevation Models," Water Resources Research, 33(2): 309-319.

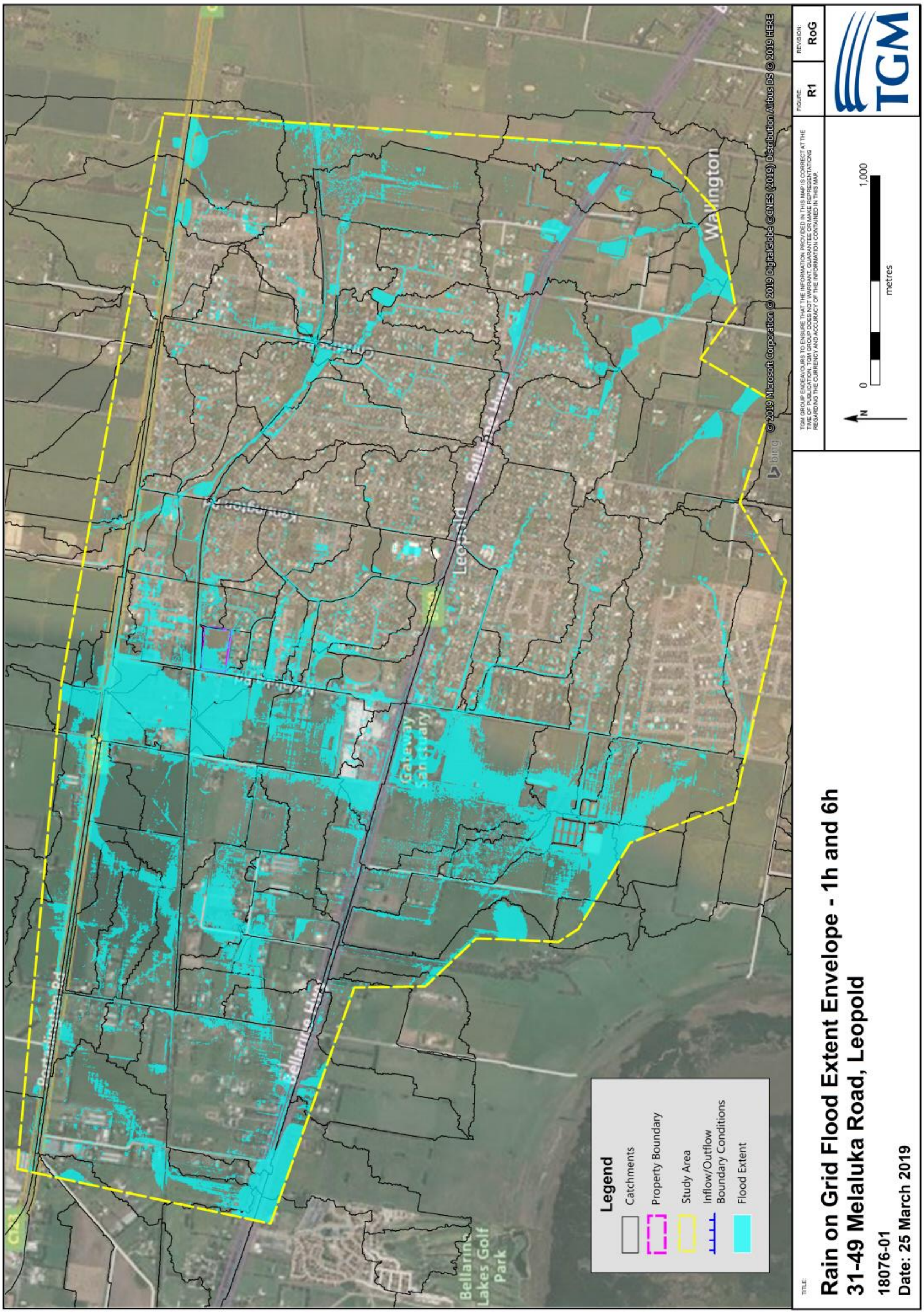

Figure 4.1: Rain on Grid 1% AEP Flood Extent

4.1.2 Analysed Sub-areas

Sub-area delineation was revised according to site boundary and existing urban drainage networks. The final 71 sub-areas are illustrated in Figure 4.2 below.

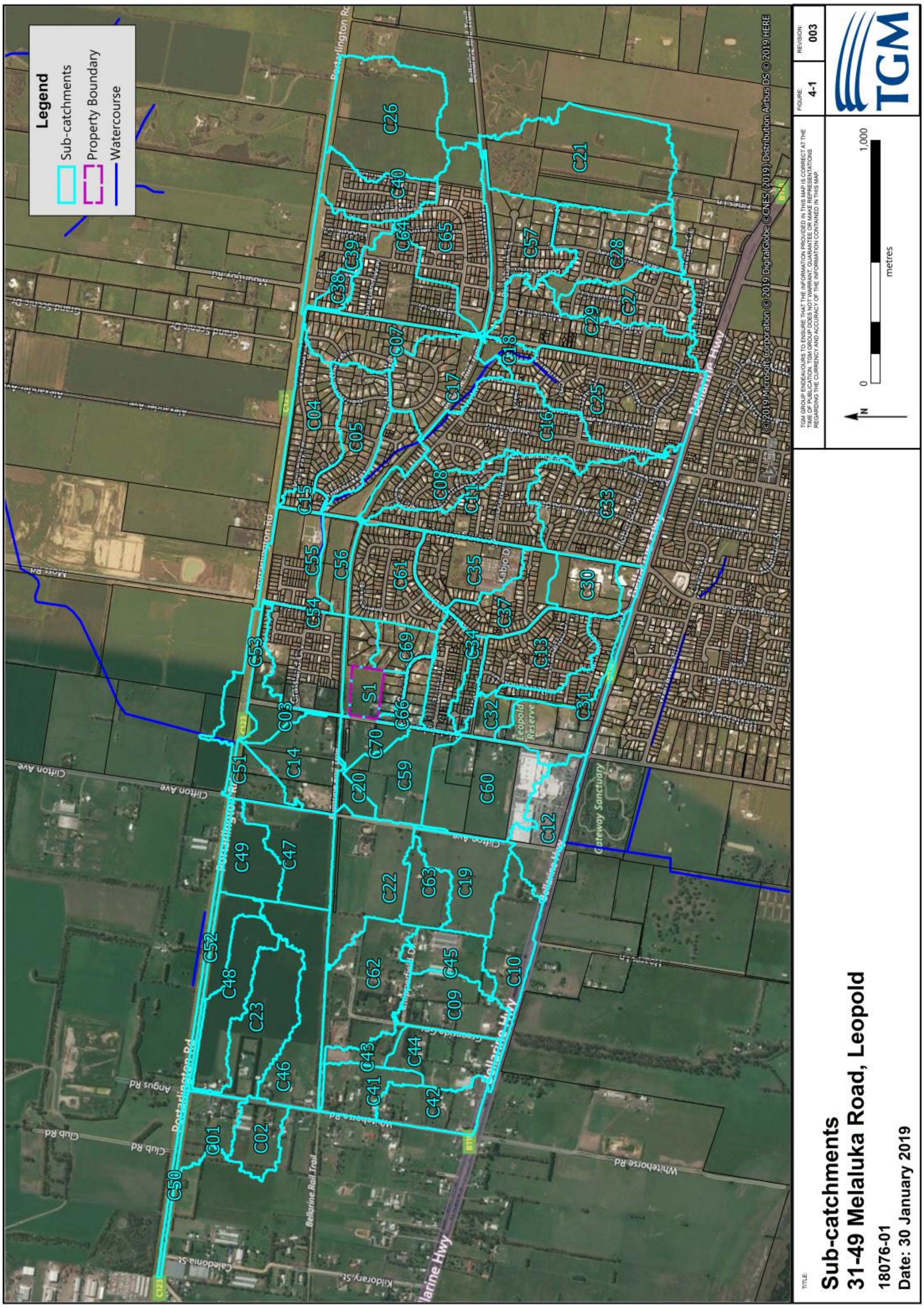

Figure 4.2: Sub-area Delineation for the Study Area

4.2 Model Parameters

4.2.1 Permeability and Fraction Impervious

A fraction impervious percentage was assigned to the sub-areas to reflect expected permeability based on planning context and actual land use characteristics.

Planning zone mapping and the most recent aerial imagery (nearmap[4]) was used to identify land use and assign a fraction impervious, reflecting the total impervious area (TIA), based on zone classifications detailed in guidelines by Melbourne Water[5].

However, Use of the TIA, which includes impervious areas with no direct connection to the drainage network, can result in the overestimation of urban runoff volumes and peak flows. Identifying the effective impervious area (EIA) provides a more realistic measure of the impervious area that generates runoff at the catchment outlet[6].

Urban sub-catchments include directly connected impervious areas (DCIA), indirectly connected impervious areas (IDCIA) and pervious areas as described in ARR2016, Section 3.4 (Estimation of Effective Impervious Area) [7]. Classification of the sub-catchment areas are as follows -

- Directly Connected Areas, which consist of:
    - Impervious areas (e.g. roofs and paved areas) which are directly connected to the drainage system – referred to as Direct Connected Impervious Areas (DCIA).
- Indirectly Connected Areas, which consist of:
    - Impervious areas which are not directly connected, runoff from which flows over pervious surfaces before reaching the drainage system (eg. a roof that discharges onto a lawn) – referred to as Indirectly Connected Impervious Areas (ICIA).
    - Pervious areas that interact with Indirectly Connected Impervious Areas, such as nature strips, garden areas next to paved patios, etc.
- Pervious areas consisting of parklands and bushland that do not interact with impervious areas.

For large urban catchments, isolating the separate hydrologic effects of these two types of impervious surfaces is challenging. Limited regional investigations suggest that a combination of these effects produced effective impervious areas (EIA) which are 55%- 65% of the TIA of urban sub-catchments.

Estimates of indirectly connected areas are further impacted by interactions between impervious and pervious areas, by storage in sub-catchments with Water Sensitive Urban Design (WSUD) measures and are influenced by antecedent soil moisture conditions (AMC).

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[4] Nearmap. https://www.nearmap.com.au/. Accessed on 9 January 2019.
[5] Melbourne Water (2018). MUSIC Guidelines – Input parameters and modelling approaches for MUSIC users in Melbourne Water's service area 2018.
[6] Ball J, Weinmann E, (Editors), 2016, Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia. Book 5 Flood Hydrograph Estimation, Chapter 3 Losses, Section 3.4.1.2 Challenges with Total Impervious Area
[7] Ball J, Weinmann E, (Editors), 2016, Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia. Book 5 Flood Hydrograph Estimation, Chapter 3 Losses, Section 3.4 Estimation of Effective Impervious Area

Recent commercial development has been undertaken in the upper catchment area. The TIA has been increased significantly, however, the EIA is much lower due to the addition of stormwater mitigation and WSUD measures.

The estimated EIA value for a catchment is calculated and then applied to hydrologic calculations using the adopted modelling software. Therefore, a conservative Effective Impervious Area (EIA) fraction was calculated, representing 70% of the Total Impervious Area (TIA) fraction. This information is summarised in Table 4.1.

Table 4.1: Total Impervious Area *vs* Effective Impervious Area for Planning Scheme Zones

Zone	Fraction Impervious	
	TIA	**EIA**
Commercial 1 Zone	0.9	0.63
Farming Zone	0	0
General Residential Zone	0.75	0.525
Low Density Residential Zone	0.2	0.14
Public Park & Recreation Zone	0.1	0.07
Rural Living	0.2	0.14
Major Road	0.7	0.7

4.2.2 Loss Parameters

XP-STORM was run as an Initial Loss and Continuing Loss (IL/CL) model using parameters provided from the ARR Data Hub[8].

The hydrologic losses adopted in this study are summarised in Table 4.2 below.

Table 4.2: Adopted Hydrological Loss Parameters

Surface	**Storm Initial Loss (mm)**	**Pre-burst Depth (mm)**	**Adopted Losses**	
			Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Pervious	19	1.55	17.45	3
Impervious	0		0	0

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[8] ARR Data Hub, http://data.arr-software.org/. Accessed on 9 January 2019.

4.2.3 Manning's Roughness Coefficients

In the hydrological model, all sub-areas are also characterised by Manning's 'n' coefficients, which describe the hydraulic roughness properties of the soil surfaces.

The Manning's coefficients adopted in this study are summarised in Table 4.3.

Table 4.3: Manning's Coefficients 'n' Adopted in the Hydrological Model

Surface	Manning's Coefficients 'n'
Pervious	0.03 – 0.05
Impervious	0.018

The Manning's coefficients for the pervious surface have been further analyzed through the sensitivity analysis process (Section 4.3).

4.3 Sensitivity Analysis

The parameters of hydrological models are usually determined through a calibration procedure to optimise the model performances in relation to a specific site. Indeed, the choice of the hydrological model parameters usually reflect the characteristics of the site and the soil properties.

The most used calibration procedure in rainfall-runoff hydrological models involves the comparison between observed and computed data. In the calibration phase, hydrological model parameters are adjusted to attain an output that matches the observed data. However, in absence of gauged data, both calibration and validation are not possible.

The Regional Flood Frequency Estimation (RFFE) tool provided by ARR2016 indicates peak flood estimates for **rural catchments** and cannot be applied to urban catchments (where more than 10% of the catchment is affected by residential or urban development).[9] The RFFE cannot be used to define the expected runoff discharge from the 'existing catchment', however, it can be used to define expected runoff discharges from a 'pre-urban development' catchment.

The RFFE tool was employed as a point of reference in the assessment of the suitability and sensitivity of the selected hydrological parameters.

The sensitivity analysis was **setup to reflect rural or pre-development catchments** to allow the comparison with flood estimation techniques. More in detail, all sub-areas have been **considered as rural**, and the **pervious fraction has been set at 100%**.

A key element of the sensitivity analysis process is the identification of the stormwater catchments that impact on the study area, the characteristics of those catchments and the configuration of waterways.

Factors such as availability of observed rainfall data, soil type, soil conditions, land use and local knowledge were considered in this investigation.

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[9] ARR - Limits of Applicability - https://rffe.arr-software.org/limits.html, viewed on 12/03/2019

The sensitivity analysis has been undertaken to the 10% AEP event identified using the ARR2016 RFFE tool for the lumped catchment.

The 10% AEP event was selected – as the recorded data used to generate the RFFE discharges have a larger sample and more robust records of 10% AEP events. This provides a more reliable flood frequency estimate.

The sensitivity analysis procedure has been performed by changing the Manning coefficient ‘n’ for the pervious surfaces to better represent the energy losses within the study area.

A further analysis of the model predictability has been carried out by comparing the XP-STORM model and the RFFE predicted peak discharges for the 1%, 2%, 5%, 20% and 50% AEP events.

4.3.1 ARR2016 Regional Flood Frequency (RFFE) Model

The ARR2016 RFFE model[10,11] available online at http://arr.ga.gov.au/; was used to provide peak flow estimates for the study catchments. The RFFE model interface, input parameters and statistical outputs can be seen in the Appendix A.

The RFFE model provides peak flood estimates for rural catchments, therefore, the lumped model was initially considered to be undeveloped (pre-development) to allow comparison.

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[10] Rahman. A, et al (2013). New Regional Flood Frequency Estimation (RFFE) Method for the whole of Australia: Overview of progress. Paper. Flood plain conference 2013.

[11] Rahman, A, Haddad, K, Kuczera. G and Weinmann, E, 2016, Peak Flow Estimation, Chapter 3 Book 3 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia.

4.3.2 Storm Burst Pattern Ensemble

The XP-STORM model applied ensemble rainfall patterns, storm burst loss factors and runoff estimation techniques from ARR2016[12] to the study catchment area to generate runoff hydrographs and predict the volume of stormwater generated.

As detailed in ARR2016[13] the majority of hydrograph estimation methods used for flood estimation require a temporal pattern that describes how rainfall falls over time as a design input.

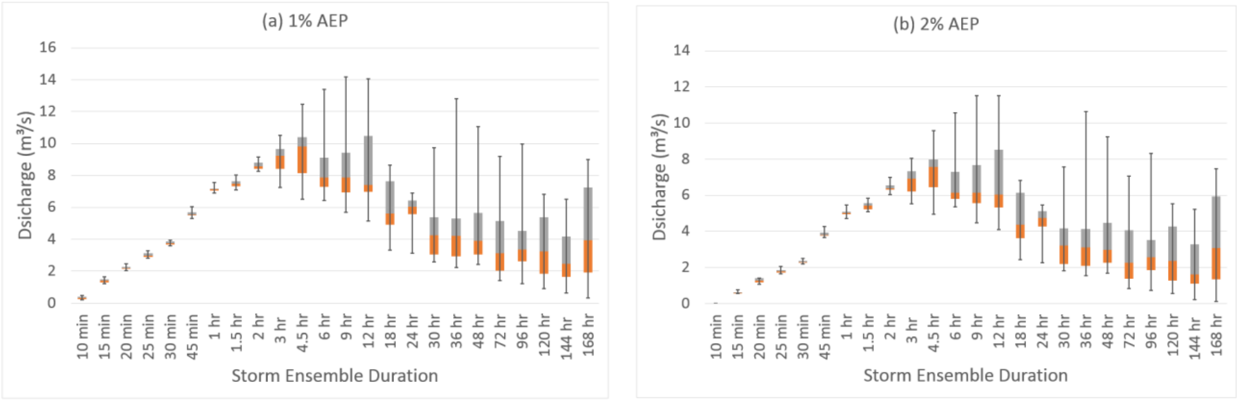
The importance of temporal patterns has increased as the practice of flood estimation has evolved from peak flow estimation to full hydrograph estimation.

An ensemble of 240 storms was analysed within the XP-STORM model for each storm event probability. For the sensitivity analysis process, the study catchment was set up as a rural catchment with no impervious surfaces.

Using the burst initial loss and continuing loss identified in Table 4.2, and known catchment characteristics, i.e. area, slope, overland flow path profiles, etc. the model was run using all 240 storm burst patterns for each AEP.

The Median, Average, Maximum and Minimum peak flow output hydrographs were identified for each storm duration using a statistical spreadsheet. The results are presented in a box and whiskers plot in Figure 4.3.

ARR2016 states that the temporal pattern that represents the worst (or best) case should not be used by itself for design. Testing has demonstrated that on most catchments large number of events in the ensemble patterns are clustered around the mean and median[14].



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[12] Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), 2016, Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia.

[13] Babister, M, Retallick, M, Loveridge, M, Testoni, I, and Podger, S, 2016. Temporal Patterns, Chapter 5 Book 2 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

[14] Babister, M, Retallick, M, Loveridge, M, Testoni, I, and Podger, S, 2016. Temporal Patterns, Chapter 5 Book 2 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

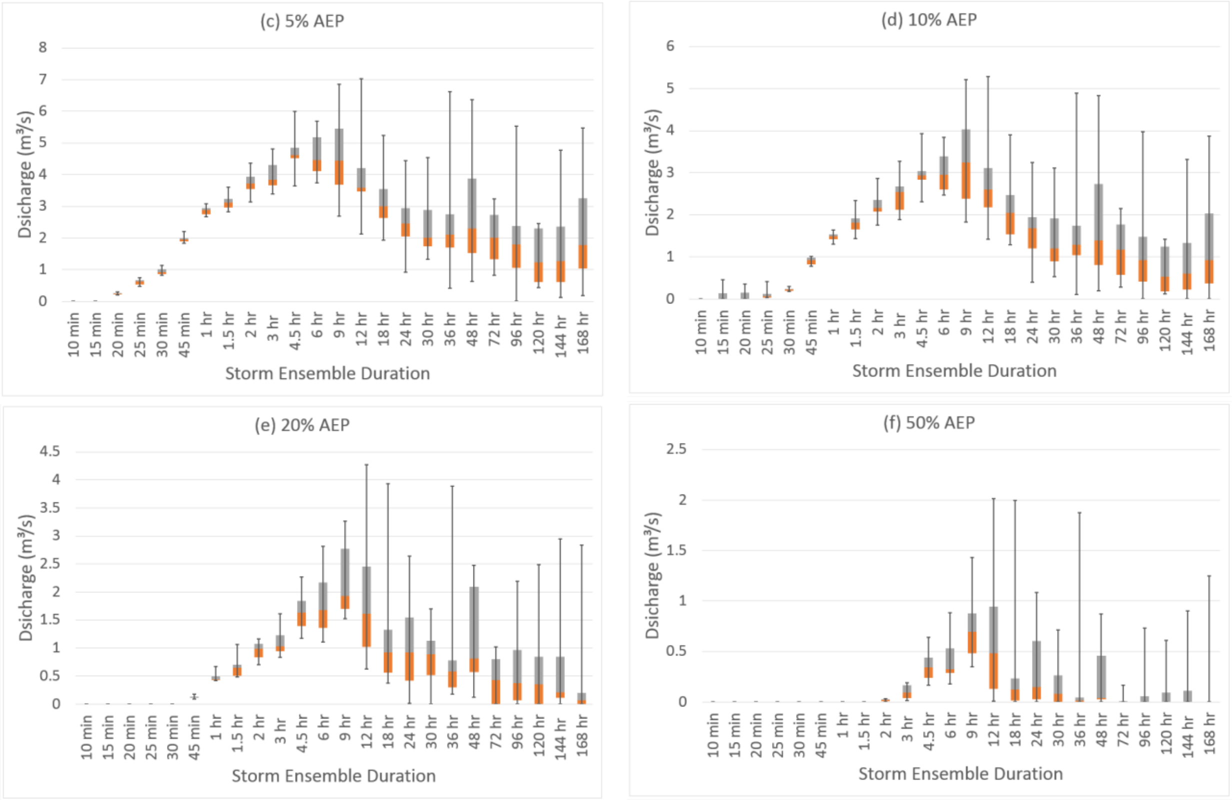

Figure 4.3: Sensitivity Analysis - Temporal Pattern Box and Whiskers Plots

4.3.3 Result Summary

Based on the sensitivity analysis, the Manning's '*n*' was refined. The comparison between the peak discharges generated with the XP-STORM model and the estimated RFFE model for the catchment is summarised in Table 4.4 and shown in Figure 4.4.

The hydrological parameters defined by the catchment characteristics, were capable of generating discharges within an acceptable range of the predicted RFFE discharge targets for all event probabilities.

Table 4.4: Manning's '*n*' and Peak Discharges

Event AEP (%)	Area (ha)	Manning's '*n*' adopted	RFFE Discharge (m³/s)	XP-STORM Discharge (m³/s)
50	389.459	0.035	1.25	0.70
20			2.29	1.93
10			3.18	3.25
5			4.19	4.62
2			5.74	7.57
1			7.10	9.80

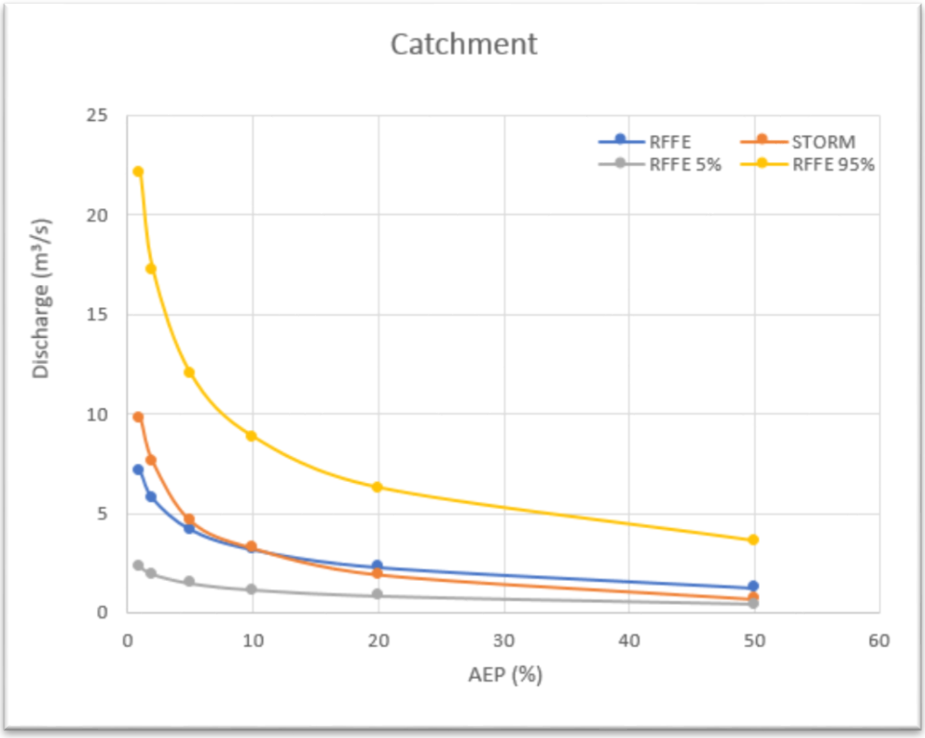

Figure 4.4: Estimated Peak Discharge - XP-STORM vs RFFE

4.3.4 RFFE Accuracy Considerations

A RFFE technique essentially represents a 'transfer function' that converts predictor variables to a flood quantile estimate. It is assumed that the use of a limited number of predictor variables (e.g. catchment area and design rainfall intensity) combined with an optimised transfer function captures the general nature of the rainfall-runoff relationship for flood events and hence provides flood quantile estimates of 'acceptable' accuracy.

ARR2016 identified ongoing concerns about estimation of parameter values (such as runoff coefficient and time of concentration) that are the basis of using the Probabilistic Rational Method[15].

The use or application of the Probabilistic Rational Method, including the VicRoads variant, is no longer supported or recognised in ARR2016 as being a suitable RFFE technique[16,17].

All RFFE techniques are subject to uncertainty, which, generally, is likely to be greater than for at-site Flood Frequency Analysis when a good quality and long record of streamflow data set is available at the location of interest.

The RFFE model estimates of regional flood frequency included substantial error bounds and are considered to be a best estimate of rarer events that cannot be described in the ungauged catchment. Recent studies[18] show how hydrological parameters from gauged catchments can be transferred to nearby ungauged catchments with similar natural characteristics.

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[15] Coombes P.J., Babister M., and McAlister A., (2015), *Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments.* 36th Hydrology and Water Resources Symposium, Engineers Australia, Hobart.

[16] Rahman, A, Haddad, K, Kuczera. G and Weinmann, E, 2016, Peak Flow Estimation, Chapter 3 Book 3 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

[17] Coombes, P, Babister, M, McAlister, T, 2015, Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments, Conference Paper. Hydrologic Water Resource Symposium.

[18] Coombes, P, Colegate, M, Barber, L, Babister, M, 2016, Modern perspective on hydrology processes of two catchments in Regional Victoria. 37th Hydrologic and Water Resource Symposium 2016.

4.4 Temporal Pattern Selection

4.4.1 Temporal Pattern Concept

In order to properly understand the concept of temporal pattern, it is necessary to understand the components of a storm event and how they relate to Intensity Frequency Duration Data (IFD) and catchment response.

Components of a typical storm pattern have been characterised in Figure 4.5. It is important to note the components can be characterised either by IFD relationships or by catchment response and are highly dependent on the definitions used. The components of a storm include:

1. Antecedent rainfall - is rainfall that has fallen before the storm event and is not considered part of the storm but can affect catchment response. This is important to understand when calibrating to or modelling historic events.
2. Pre-burst rainfall - is storm rainfall that occurs before the main burst. With the exception of relatively frequent events, it generally does not have a significant influence on catchment response but is very important for understanding catchment and storage conditions before the main rainfall burst. Pre-burst rainfall often accounts for a proportion of the initial losses within a catchment. Pre-burst depths need to be quantified when only modelling storm burst patterns.
3. The burst - represents the main part of the storm but is very dependent on the definition used. Bursts have typically been characterised by duration. The burst could be defined as the critical rainfall burst, the rainfall period within the storm that has the lowest probability, or the critical response burst that corresponds to the duration which produces the largest catchment response for a given rainfall Annual Exceedance Probability (AEP).
4. Post-burst rainfall - is rainfall that occurs after the main burst and is generally only considered when aspects of hydrograph recession are important. This could be for drawing down a dam after a flood event or understanding how inundation times affect flood recovery, road closures or agricultural land.

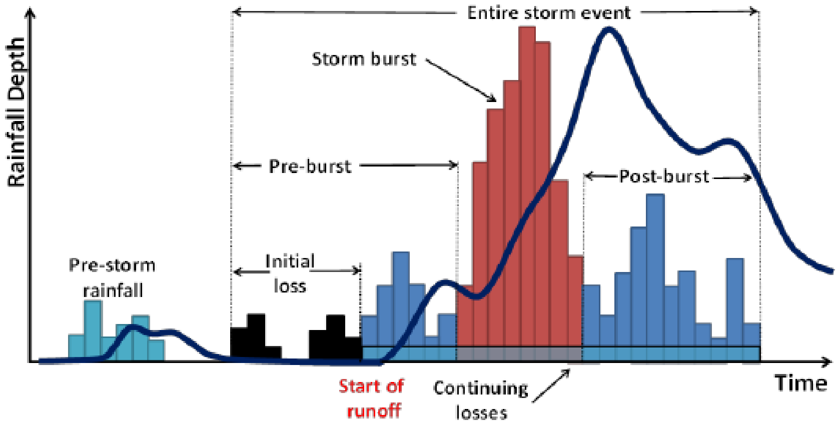

Figure 4.5: Elements of a Complete Storm Event and Hydrological Practice[19]

For this study, the Bureau of Meteorology's 2016 IFD data and ARR2016 temporal patterns were used to produce an ensemble of storm burst patterns which were analysed for a whole catchment response.

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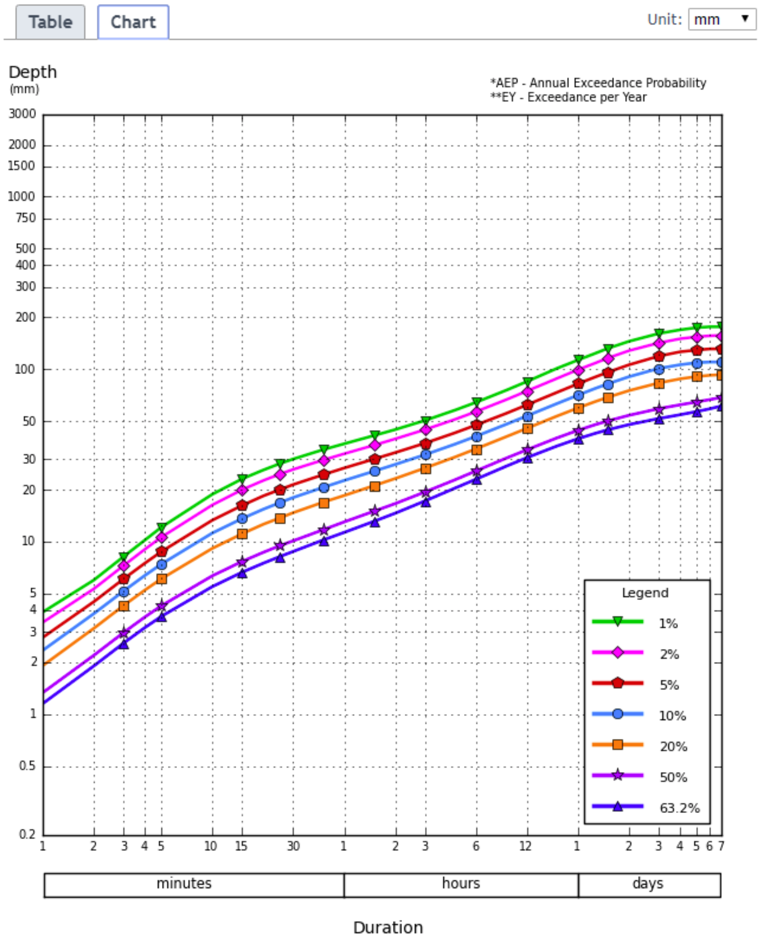
[19] Coombes, P, Colegate, M, Barber, L, Babister, M, 2016, A modern perspective on the hydrology processes of Armstrong Creek and Canadian Creek catchments in Regional Victoria. 4th National Conference on Urban Stormwater Management 2016.

4.4.2 IFD Data

The 2016 rainfall intensity frequency duration (IFD) climatic data used in the hydrological model was extracted from the Bureau of Meteorology (BOM) website[20]. The IFD curves are shown in Figure 4.6.



Note:

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

Figure 4.6: 2016 IFD Curves – Bureau of Meteorology 2019

4.4.3 Critical Storm Burst Pattern Selection

The historical process of using peak flows derived from a single critical storm burst does not account for the hydrological processes generated by the reality of complete (full volume) storms as demonstrated in Figure 4.5. It is important to understand the hydrological losses within the catchment and the relationship of the losses to both full storms and storm bursts.

For this analysis, 10 storm burst temporal patterns were extracted for 24 duration periods, for each AEP event. By analysing the hydrological response to the ensemble temporal patterns, one critical pattern was selected for each of the 24 durations. The fixed temporal patterns over the entire study area for design flood estimation were implemented and the spatial variation was not considered. The analysed events and durations are shown in Table 4.5.

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[20] Bureau of Meteorology, http://www.bom.gov.au/water/designRainfalls/. Accessed on 14 November 2018.

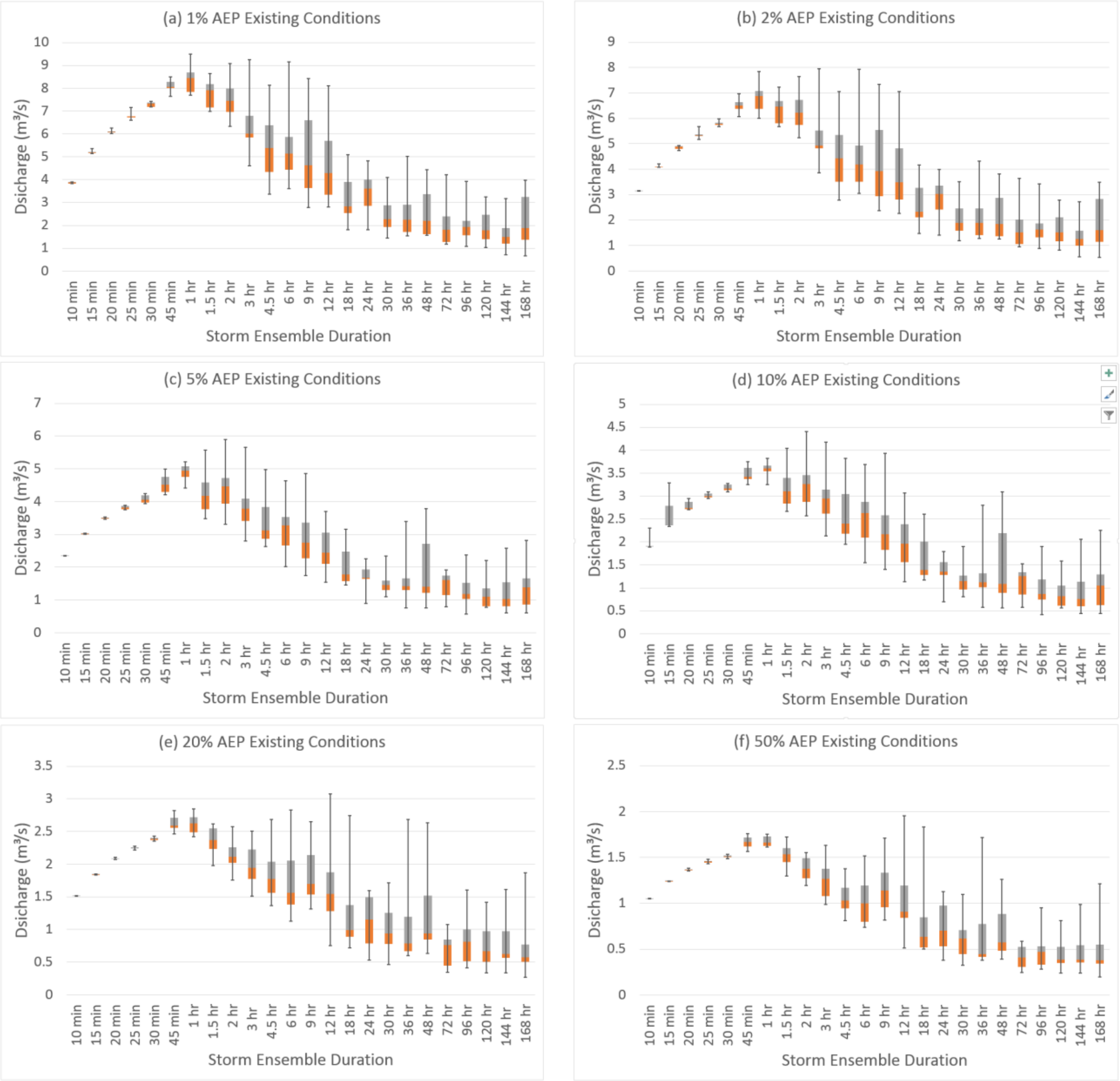
Table 4.5: Analyzed Rainfall Patterns, Durations and Events

Number of Storm Burst Patterns in Ensemble (per event duration)	Storm Durations Analysed (minutes)			Event Probability Range Analysed (AEP)	
				(%)	(1 in *x*)
10	10	120	1,800	1	100
	15	180	2,160	2	50
	20	270	2,880	5	20
	25	360	4,320	10	10
	30	540	5,760	20	5
	45	720	7,200	50	2
	60	1,080	8,640		
	90	1,440	10,080		

The median value of the peak discharges generated for 10 temporal patterns under pre-developed conditions has been calculated. The critical temporal pattern was selected by identifying the temporal pattern characterised by the peak discharge closest to the median for each of the 24 durations. The procedure has been then repeated for each event probability.

The box-plots are shown in Figure 4.7 to illustrate the variation of the discharge predicted by the ensemble patterns.

The adopted storm burst patterns are shown in Table 4.6.


(a) 1% AEP Existing Conditions

(b) 2% AEP Existing Conditions

(c) 5% AEP Existing Conditions

(d) 10% AEP Existing Conditions

(e) 20% AEP Existing Conditions

(f) 50% AEP Existing Conditions

Figure 4.7: Temporal Pattern Box and Whiskers Plots

4.5 Hydrological Model Simulations

Sensitivity analysis models applied 100% pervious surfaces within the catchments. Impervious surfaces and urban characteristics were integrated into the temporal pattern selection and existing conditions hydrological models.

The modelling work was conducted through the study area for all AEPs. The hydrographs of 71 sub-areas simulated by the rainfall-runoff model were then used as inputs (boundary conditions) to the 2D hydraulic model.

Table 4.6: Adopted Storm Burst Patterns

Duration	Temporal Pattern No.					
	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP	50% AEP
10 min	9	1	4	2	2	2
15 min	5	4	1	5	4	3
20 min	2	9	6	8	6	1
25 min	3	6	5	9	2	3
30 min	5	3	8	6	6	7
45 min	3	7	5	5	1	1
1 hour	9	3	6	3	5	5
1.5 hours	7	7	4	8	1	10
2 hours	7	7	4	7	1	1
3 hours	7	7	5	3	1	9
4.5 hours	6	6	1	1	5	7
6 hours	8	2	5	1	2	4
9 hours	7	8	7	7	7	3
12 hours	7	7	10	10	5	5
18 hours	3	9	3	2	4	4
24 hours	6	6	2	1	1	1
30 hours	6	1	4	4	10	9
36 hours	5	2	3	6	6	1
48 hours	6	2	9	1	7	2
72 hours	4	4	7	5	4	4
96 hours	4	7	5	5	2	2
120 hours	6	10	2	3	2	1
144 hours	1	1	2	2	1	3
168 hours	5	5	6	6	3	2

5. HYDRAULIC MODEL

The extent of flooding within the study area was evaluated by employing outputs from the hydrological model in the two-dimensional hydraulic model TUFLOW HPC (build 2017-09-AC-iSP-w64) using Graphics Processing Unit (GPU).

The 1D elements have been adopted to represent underground drainage network.

In addition, 1D elements are employed to represent the hydraulic devices that control the flood dynamics within the study area, such as culverts, weirs, sluice gates, pumps, etc.

A 2-metre model of the area of interest was set up for this study. This model was used to:

- Identify the critical duration storm events impacting the site. This allowed for simulation of the full 24 duration event ensembles within reasonable runtimes;
- Generate the predicted flood extents to be employed as the existing conditions ‘base case’ for this stage of the project. This model only applied the critical storm durations impacting the study area;
- Assess the flood impact of the developed conditions by integration of the design.

The mathematical derivation of the shallow water equations solved by TUFLOW requires that the adopted 2D cell size is greater than the local water depth. For this reason, a cell size of 2 m has been adopted to adequately represent the 2D domain from a numerical modelling perspective in the fine grid model.

Moreover, the 2-metre grid enabled detailed definition of the variable terrain topography, road network, overland flow paths and various hydraulic obstructions. This grid size was also determined to be of suitable size to allow accurate definition of the terrain without adversely affecting the simulation run times.

5.1 Digital Elevation Model

A bottom elevation value was assigned to each cell of the hydraulic model using the most updated digital elevation model (DEM) available for the area of interest.

The DEM was generated using LIDAR data from the *Vicmap Digital LiDAR Elevation DEM* dataset. The LIDAR DEM, shown in Figure 5.1 has a resolution of 1 m with a horizontal accuracy of ± 20 cm and a vertical accuracy of ± 10 cm.

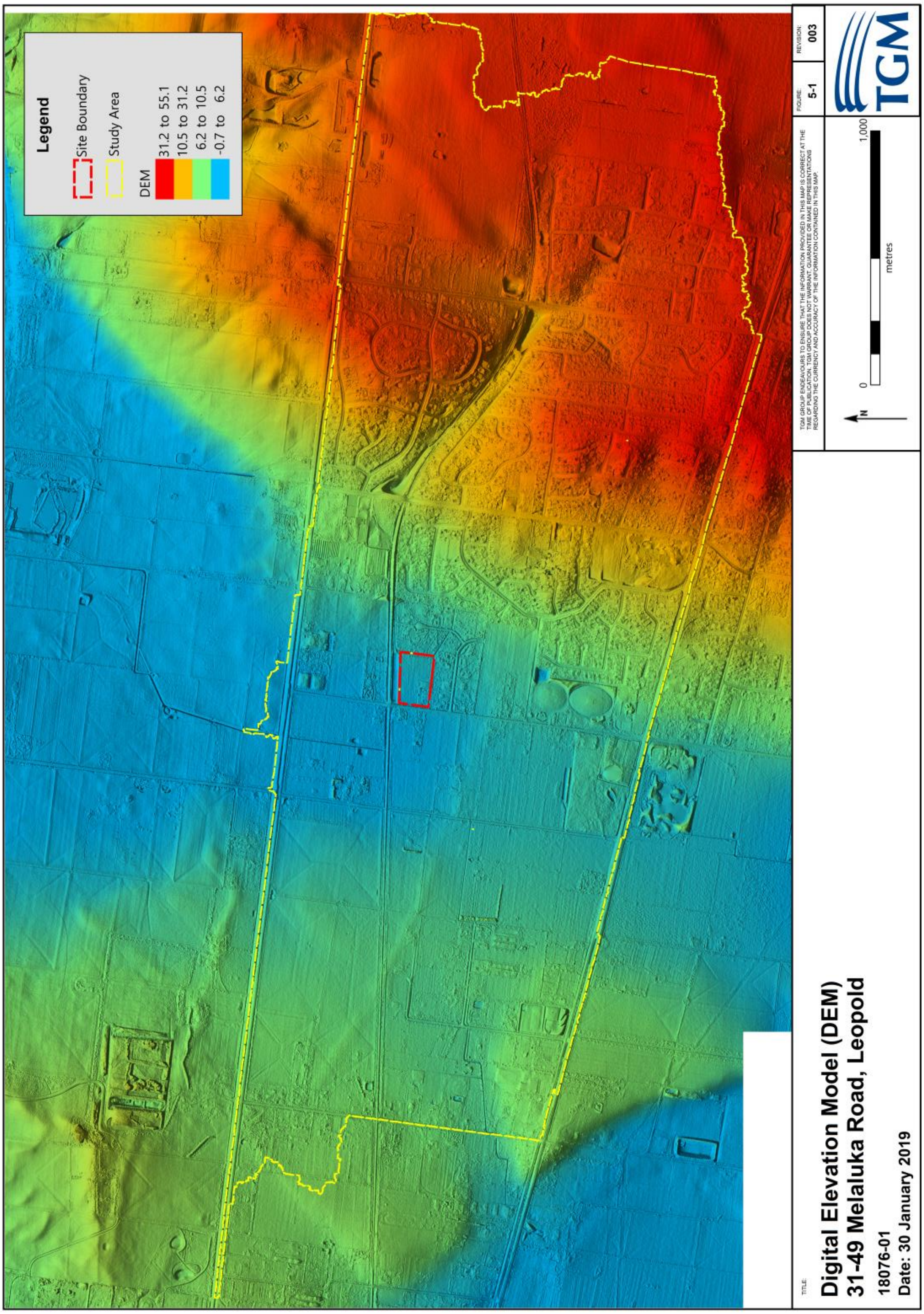


Figure 5.1: Digital Elevation Model (DEM)

5.2 Underground Drainage Network

Underground drainage systems are an integral factor in urban flooding. The capacity of the underground system to accommodate stormwater runoff will directly impact the overland flows.

City of Greater Geelong Geographic Information Systems (GIS) drainage tables were downloaded from the Australian Governments National Map website and used to identify the location and characteristics of the underground drainage systems.

It is noted that the GIS drainage database does not provide invert levels of underground drainage systems. The inverts levels of the underground pipe network were determined at pit locations on the basis of the ground elevations extracted from the DEM. Minimum grade and minimum cover requirements were used to estimate the pipe inverts throughout the system.

Key features of the drainage system in the area of interest, such as bridge culverts, trunk systems, pipes with a diameter ≥ 300 mm and associated drainage pits, were selected and represented as 1D elements in the TUFLOW hydraulic model.

It may be necessary to undertake additional survey and inspection of the underground drainage system at key locations during functional design.

The extent of the GIS underground pipe network integrated into the 2D hydraulic model can be seen in Figure 5.2, where different pipe sizes are highlighted with relevant widths and colours.

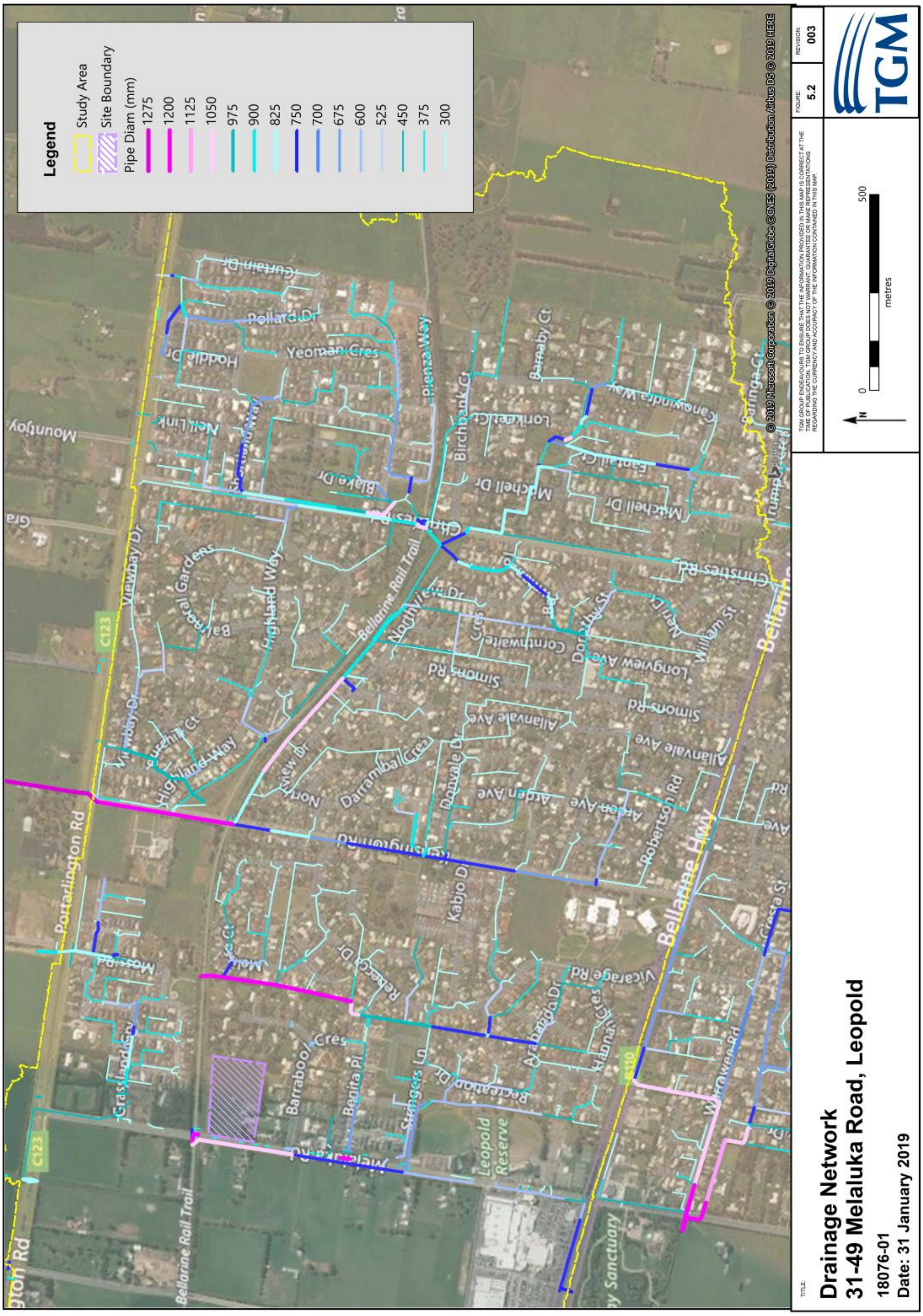


Figure 5.2: Underground Drainage Network

5.3 Manning's Roughness Coefficients

Manning's roughness coefficients were adopted for the TUFLOW hydraulic model with variation in roughness to reflect the urban development and gravel, sealed and asphalt road networks. The Manning's roughness coefficients applied in this study are shown in Table 5.1.

Table 5.1: Manning's Roughness Coefficients *'n'*

Land Use	Manning's n
Buildings	0.15
Gravel roads	0.029
Asphalt roads / road reserve	0.016
Pasture / Rural Living	0.035

5.4 Boundary Conditions

The output hydrographs from the XP-STORM hydrological model were applied as inflow boundary conditions into the TUFLOW hydraulic model. The locations of the inflow and outflow boundary conditions are shown in Figure 5.3.

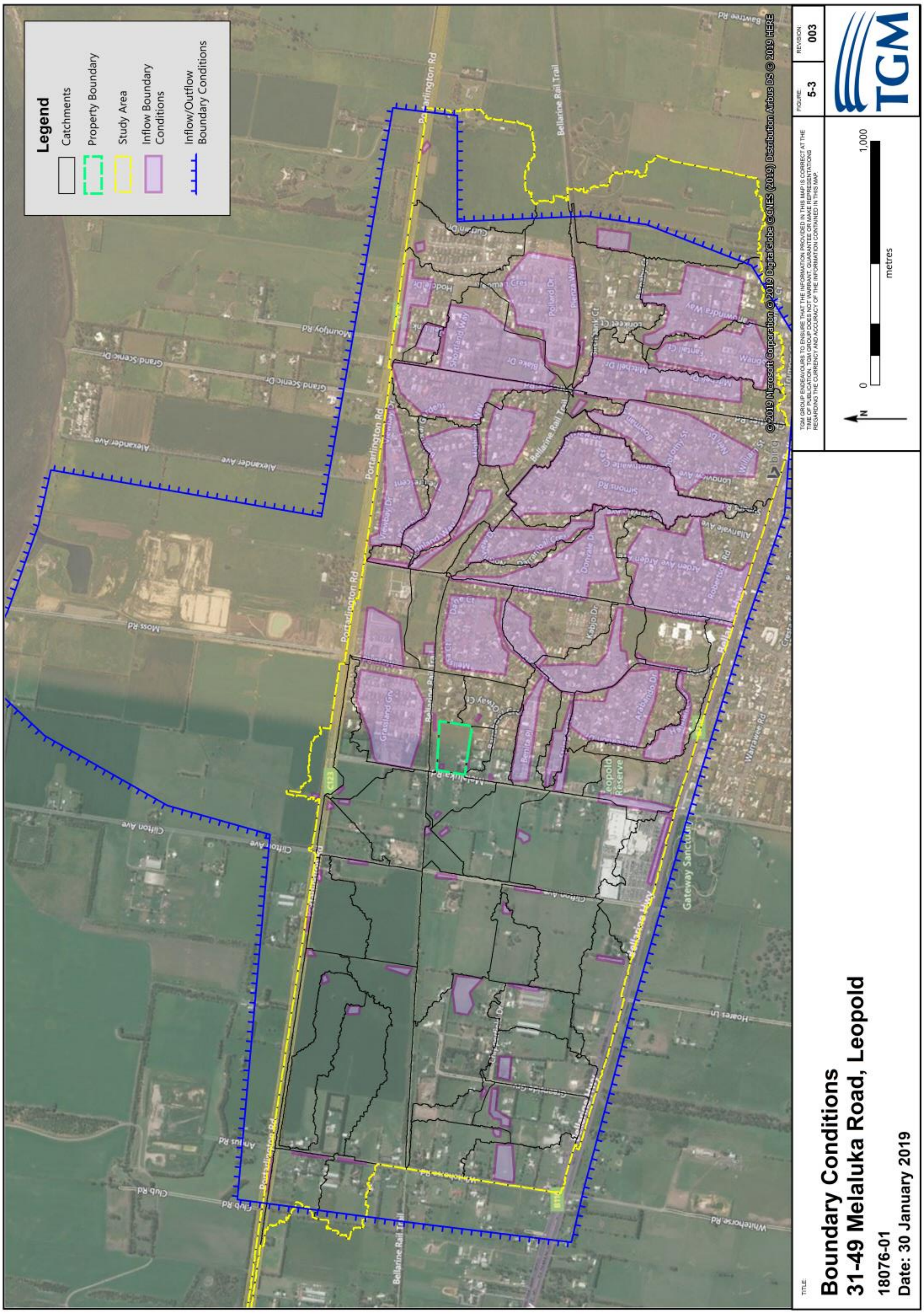


Figure 5.3: 2D Hydraulic Model – Boundary Conditions

6. SITE STORMWATER MANAGEMENT

The objective of the site stormwater management plan (SSMP) is to mitigate adverse impacts on stormwater discharges resulting from the development of the site. Site stormwater discharge will meet the conditions and requirements for stormwater management. These requirements ensure that appropriate design and stormwater mitigation is applied to ensure that stormwater quality and quantity targets are achieved and maintained.

The site stormwater objectives for new developments is summarised in Table 6.1.

Table 6.1: Stormwater Objectives

Developed Site - Stormwater Objectives	
Best Practice reductions for Water Quality	**80%** reduction in Suspended solids (SS)
	45% reduction in total nitrogen (TN)
	45% reduction in total phosphorus (TP)
	70% reduction in gross pollutants (GP)
No-worsening stormwater peak discharges	Maintain predeveloped discharges rates for storm events up to and including the 1% AEP

For this analysis, a MUSIC model was developed to analyse stormwater quality and efficiency of treatment facilities. A local XP-STORM model was used to define the hydrology of the study area and assess stormwater discharges. The following analysis applied detailed inputs including topographical, climatic and geological data to define local hydrological and stormwater quality models.

6.1 SSMP Site Area

For the SSMP, only the study site which is subject to development was considered in the design of the treatment train. The site area was identified as 2.506 ha, as detailed in Table 6.2.

The assumption for effective fraction impervious of the developed site is the same as defined in Section 4.2.1. The change in fraction impervious for the developed site is shown in Table 6.2.

Table 6.2: Change in Impervious Surfaces

Sub Catchment/Area	Area (ha)	Effective Fraction Impervious (%)	Impervious Area (ha)	Pervious Area (ha)
Existing	2.506	0.03	0.069	2.437
Developed	2.506	0.525	1.316	1.190

6.2 Stormwater Quality

Assessment of the quality of stormwater discharge from the developed site was undertaken using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) by eWater[21]. It allows the analysis of stormwater quality and the assessment of the efficiency of the treatment facilities. The operation of MUSIC requires climatic forcing, i.e., rainfall and potential evapotranspiration (PET), and geological parameters.

6.2.1 Climatic inputs

The closest rainfall gauge to the site with pluviograph data is Geelong North Station (087133) with a mean annual rainfall of 534.2 mm, located 10.55 km north-west from the site. The pluviograph data from this site was used as input for the MUSIC model.

The evapotranspiration (ET) is the process for the transfer of water from the land surface to the atmosphere as water vapour through surface evaporation and vegetation transpiration. Actual ET is determined by regional climate (e.g., radiation and temperature) and the availability of water and vegetation. The potential evapotranspiration (PET) is defined as the amount of ET which would occur with sufficient source of water. It varies throughout the Australian continent due to the different climate zones.

MUSIC adopts the monthly areal average PET data provided by the BoM[22] as a forcing variable with the assumption that the PET is constant within each calendar month. In this study the PET extracted for North Geelong station was used so as to be consistent with the pluviography data. A map of the average annual areal PET is shown below in Figure 6.1.

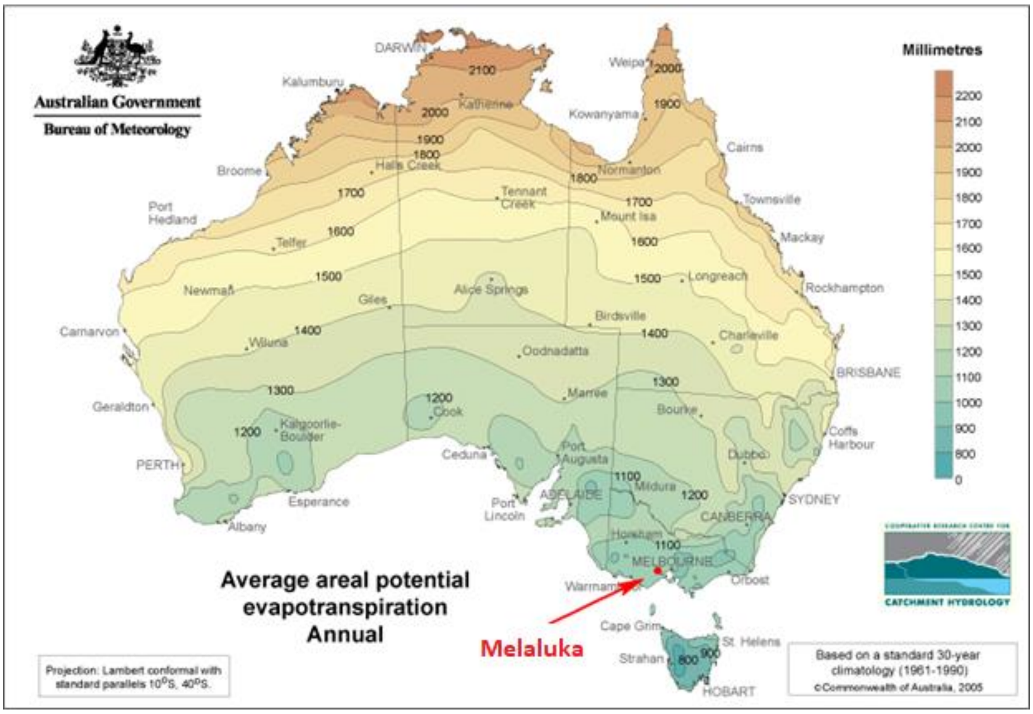


Figure 6.1: Average Annual Areal Potential Evapotranspiration (data sourced from BoM[22])

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[21] https://ewater.org.au/products/music/
[22] http://www.bom.gov.au/climate/

6.2.2 Geological parameters

Geological parameters were adopted from Melbourne Water MUSIC Guideline[5], as shown in Table 6.3.

Table 6.3: Soil Characteristics for the Study Site

Parameter	Urban Residential
Rainfall Threshold (mm/day)	1
Soil Capacity (mm)	120
Initial Storage (%)	25
Field Capacity	50
Infiltration Capacity coefficient a	200
Infiltration Capacity coefficient b	1
Initial Depth (mm)	10
Daily Recharge Rate (%)	25
Daily Base flow Rate (%)	5
Daily Deep Seepage Rate (%)	0

6.3 Stormwater Quantity

To ensure that the maximum site discharge in developed conditions does not exceed the existing conditions (i.e., PSD), an end-of-line detention basin was proposed.

The model was firstly used to estimate the permissible site discharges (PSDs) for the sub-catchment based on the existing undeveloped conditions.

An end-of-line detention basin was integrated into the reserve area located in the northwest corner of the site, forming the Legal Point of Discharge (LPOD) for the site (Figure 6.2). The basin size and outlet configurations were adjusted to ensure a ‘no worsening’ of discharges.

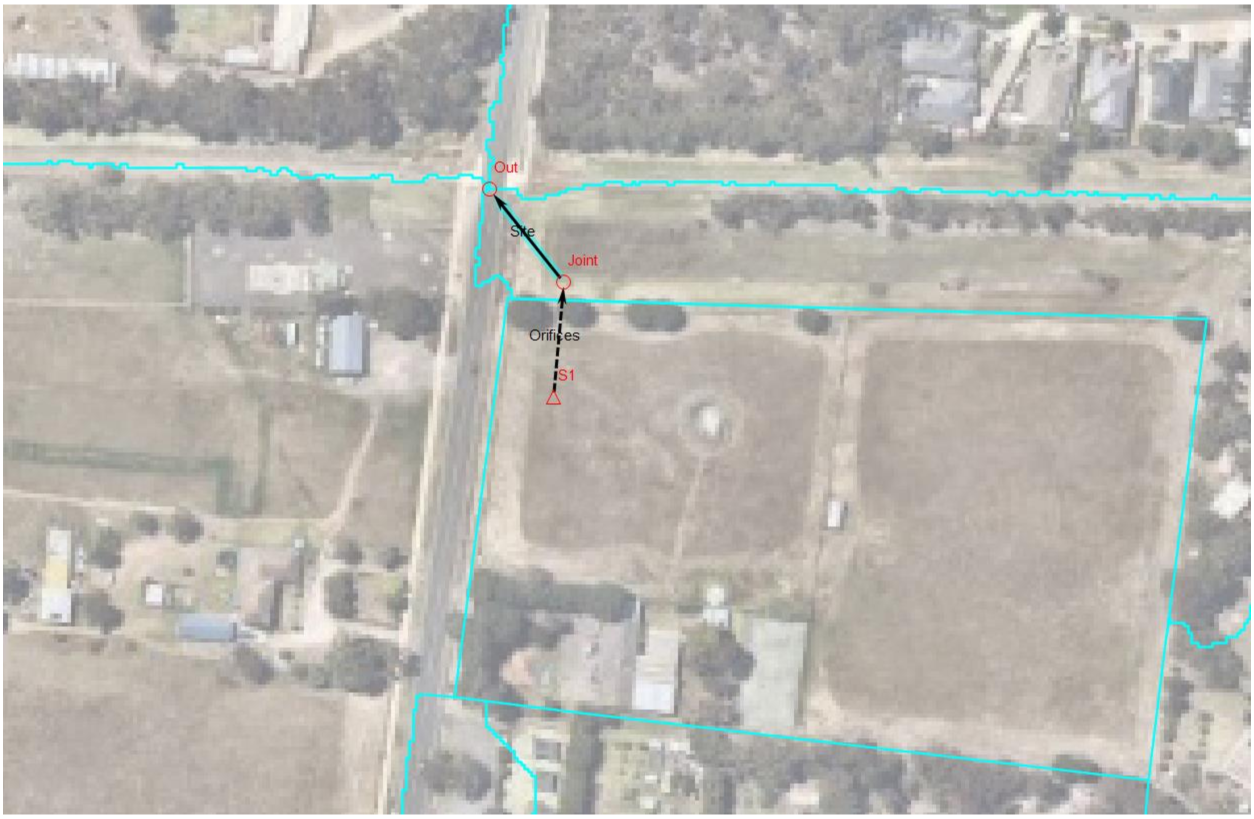

Figure 6.2: Local XP-STORM Model Schematic

6.3.1 Existing Site Conditions 'base-case'

An ensemble of storm events were used to simulate all the AEP events and evaluate the stormwater peak discharges generated by the contributing catchment areas.

The critical duration for each design event probability and each sub-catchment may vary depending on a number of conditions. Therefore, the consideration of a number of storm durations is important to ascertain the critical storm duration impacting the site.

For this study, the temporal patterns up to the 168 hr duration for each event probability were analysed. The critical peak discharge rates for all AEPs were simulated and used as PSDs for detention basins design.

6.3.2 Developed Conditions

Stormwater runoff from the developed site will be mitigated for peak discharge flow rates and discharge quality using an end-of-line wetland-detention basin facility.

It is proposed to integrate the wetland into the footprint of the detention basin. The basin size and outlet configuration was designed to allow an effective stage-storage-discharge relationship capable of ensuring a 'no worsening' of discharge from the developed site is achieved.

Basin design parameters were input into the XP-STORM program using the 'Storage' feature with the Stepwise Linear Storage method.

7. MODELLING RESULTS

The results of the hydrological, hydraulic and water quality analysis are shown in this section. The design has been undertaken to meet stormwater quality 'best practice' standards and to ensure that peak discharges for events up to and including the 1% AEP do not exceed predeveloped conditions and/or create a negative impact to neighbouring properties and receiving ecosystems.

7.1 Site Stormwater Quality Modelling Results

The ability of development to meet stormwater quality 'best practice' standards and the performance of the treatment system was continuously simulated using MUSIC. The MUSIC network is shown in Figure 7.1. Wetland facility is proposed to be constructed at the LPOD located in the northwest corner of the site. The inlet pond and storage sizes, and the outlet properties were designed to meet the stormwater quality objectives. The proposed wetland configuration and the stormwater quality treatment efficiency are summarised in Table 7.1

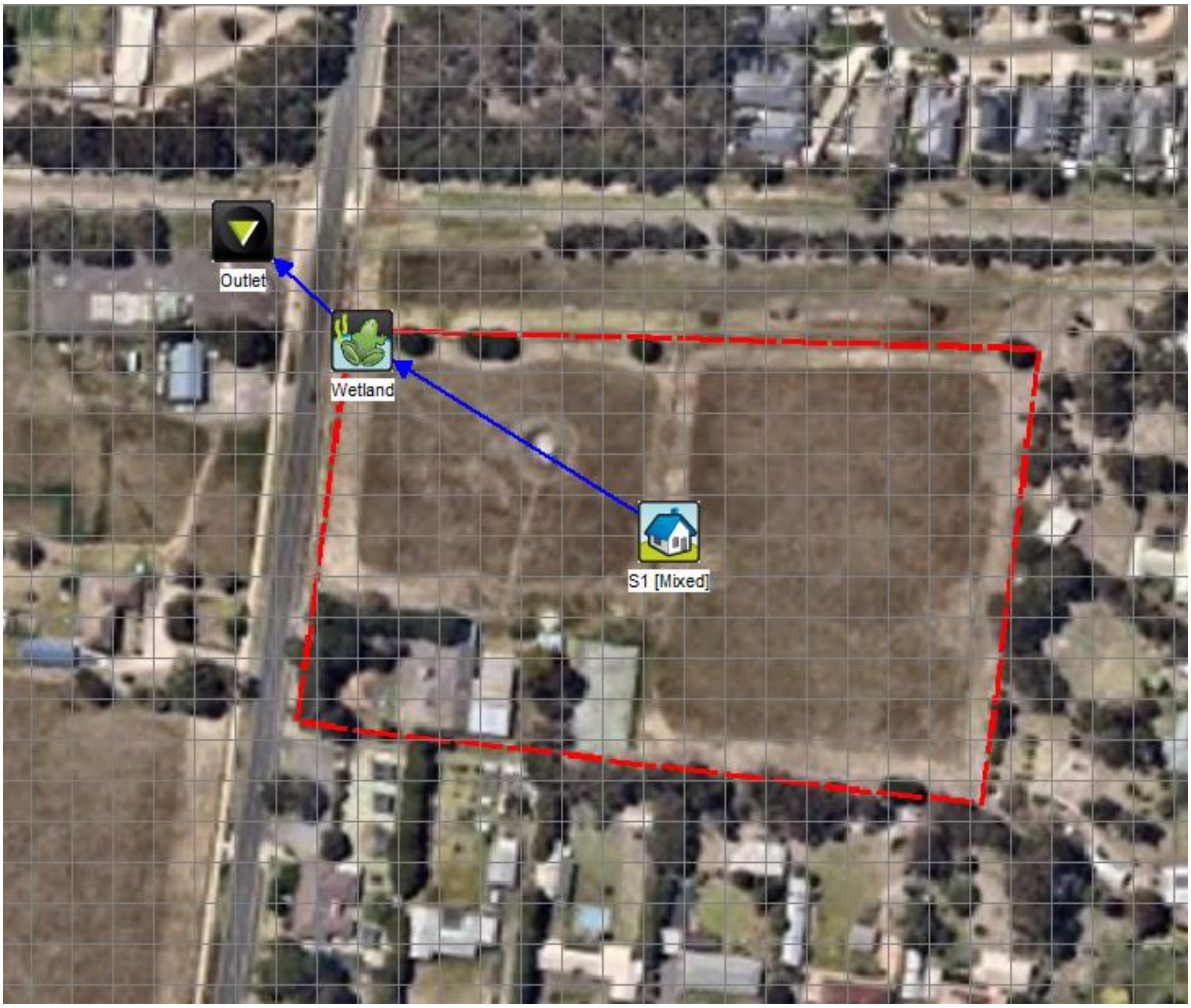

Figure 7.1: MUSIC Network Schematic

Table 7.1: Wetland Properties

Wetland Properties	Values
Low Flow By-pass (m^3/s)	0
High Flow By-pass (m^3/s)	100
Inlet Pond Volume (m^3)	100
Surface Area (m^2)	320
Extended Detention Depth (m)	0.3
Permanent Pool Volume (m^3)	32
Initial Volume (m^3)	32
Exfiltration Rate (mm/hr)	0
Evaporative Loss as % of PET	125
Equivalent Diameter (mm)	17
Overflow Weir Width (m)	5
Notional Detention Time (hrs)	72.3

Table 7.2: Stormwater Quality Treatment Efficiency

Criteria	Reduction (%)	Objectives (%)
Total Suspended Solids (kg/yr)	80.9	**80**
Total Phosphorus (kg/yr)	67.5	**45**
Total Nitrogen (kg/yr)	45.3	**45**
Gross Pollutants (kg/yr)	100	**70**

7.2 Site Stormwater Quantity Modelling Results

7.2.1 Existing Site Discharge

The permissible site discharge (PSD) from the LPOD for each event probability was determined using the local hydrological model under existing conditions. The runoff hydrograph for 1% AEP is shown in Figure 7.2 as an example. The critical peak discharges for all AEPs have been tabulated in Figure 7.3.

Table 7.3: Validated Peak Discharges in the Study Site

AEP	Critical Event Duration	Critical Peak Discharge (m³/s)
1%	2 hr	0.125
2%	2 hr	0.096
5%	3 hr	0.059
10%	1.5 hr	0.039
20%	4.5 hr	0.025
50%	9 hr	0.011

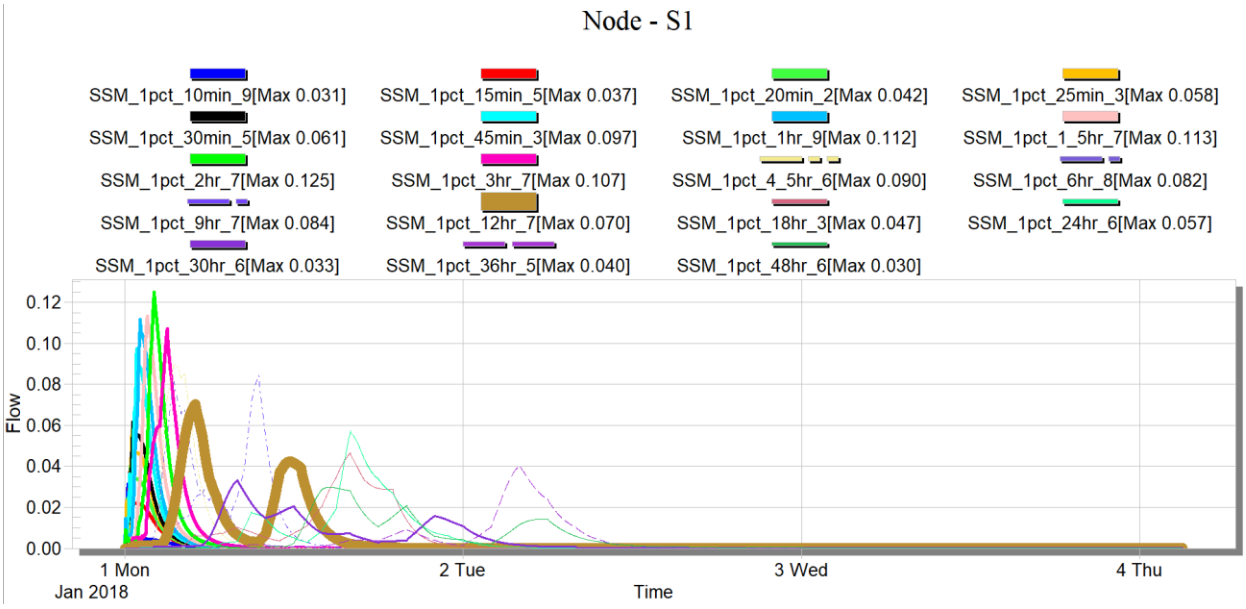

Figure 7.2: 1% AEP Hydrograph (Existing Site)

7.2.2 Developed Site Discharge

Basin size and outlet configurations were adjusted to achieve the ‘no worsen’ discharge objective. The runoff hydrograph after mitigation for 1% AEP is shown in Figure 7.3 as an example.

The peak discharge of the existing and developed (mitigated) site conditions and the final modelled configurations of the detention basin are summarized in Table 7.4.

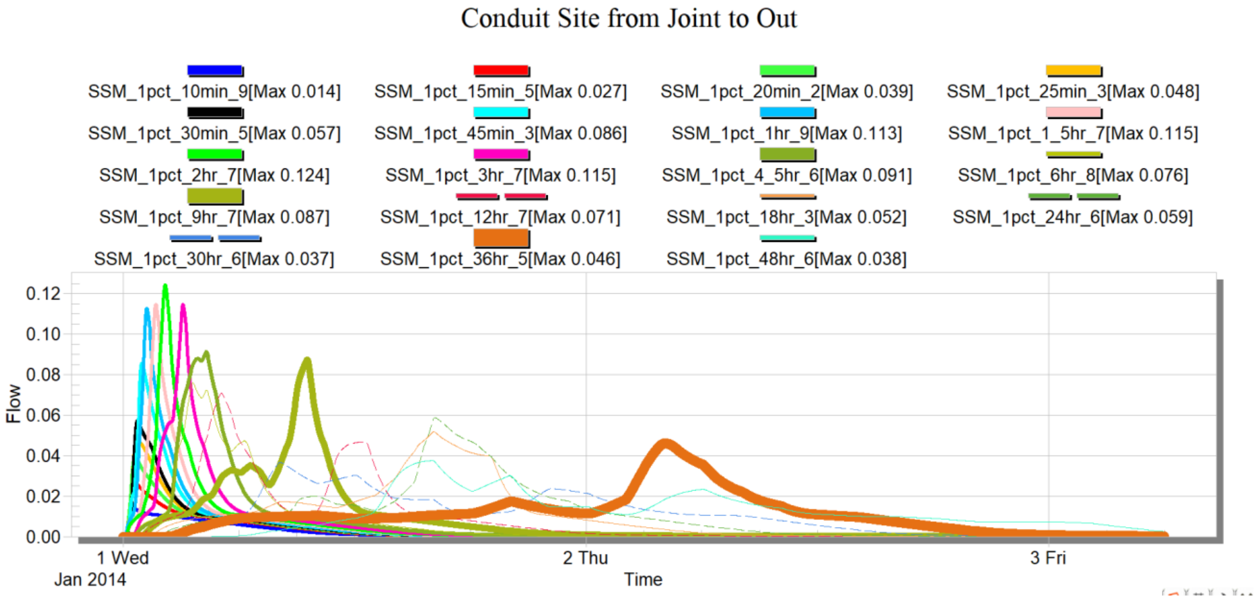

Figure 7.3: 1% AEP Hydrograph (Developed Site)

Table 7.4: Basin Requirements and Peak Discharges from Site - Existing vs Developed

Detention Basin Characteristics							
Event	Basin Stage (m)	Basin Area (m2)	Basin Volume (m³)	Orifice IL (m)	Modelled Outlet Configuration (Width x Height)	Basin Discharge (m³/s)	PSD (m³/s)
Freeboard	0.807	1560.6					
1%	0.507	1221.6	492	0.400	0.7 m x 0.2 m	0.124	0.125
2%	0.472	1184.6	450			0.089	0.096
5%	0.420	1130.7	390	0.250	0.3 m x 0.15 m	0.058	0.059
10%	0.370	1080.0	335			0.039	0.039
20%	0.332	1042.2	294			0.025	0.025
50%	0.254	966.6	216	0.000	0.1 m x 0.06 m	0.011	0.011
Bottom	0.000	739.1					

7.3 Hydraulic Model Results

The existing flood characteristics and impact of development on the regional flood extents are detailed in this section.

7.3.1 Critical Duration Selection

Runoff hydrographs, for each of the 24 event durations, were extracted from the regional hydrological model, and modelled as inflow boundary conditions in the 2D hydraulic TUFLOW model to identify the critical duration(s) impacting the site for each event probability.

The critical 1% AEP event durations for the entire catchment and the critical urban area where the flood impact assessment will be focused on are shown in Figure 7.4 as an example.

Based on the critical duration maps, the most relevant event durations were selected for further analysis to optimise model runtimes.

Critical storm durations selected for further flood impact assessment are summarised in Table 7.5.

Table 7.5: Critical Storm Durations Analysed in the Fine Grid (2 m) Model

Durations	1%	2%	5%	10%	20%	50%
10min	x	x	x			
15min	x		x	x	x	
20min		x		x	x	
30min	x		x			x
45min	x					
1hr	x	x	x	x	x	x
1_5hr	x	x	x	x		x
2hr	x	x	x	x		
4_5hr	x	x		x		
6hr			x			
9hr		x	x	x	x	x
12hr				x	x	
24hr	x	x		x	x	x

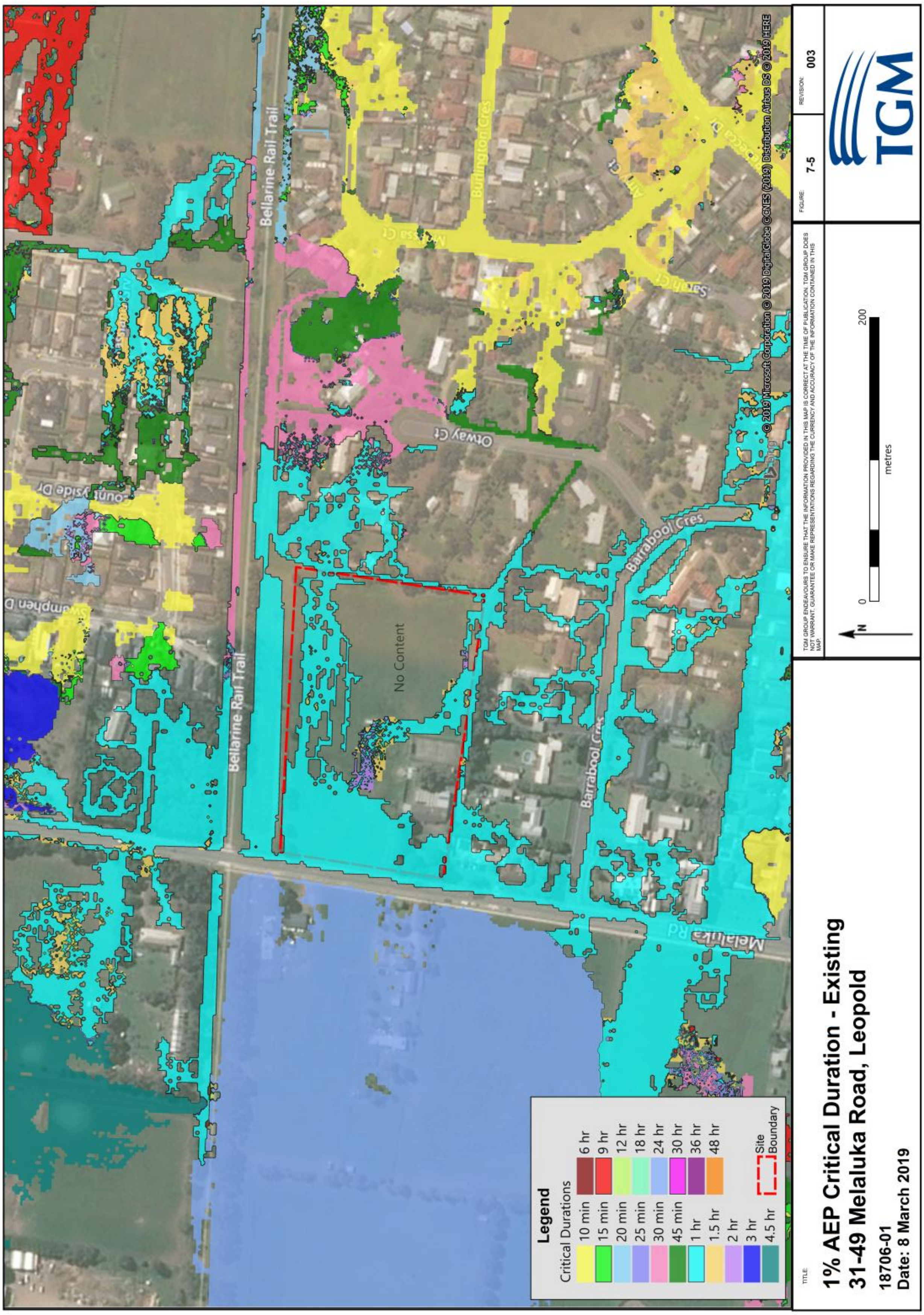

Figure 7.4: 1% AEP Critical Storm Durations

7.3.2 Flood Extent Mapping

The envelope of the computed flood depths of the entire modelled area for the existing conditions in the 1% AEP event are shown in Figure 7.6 (full flood extent). These maps are shown for the critical area in Figure 7.7 (full flood extent) and Figure 7.8 (depths ≥ 50 mm).

7.3.2.1 Developed Site Conditions

The end-of-line detention basin facility has been designed to mitigate stormwater discharged from the developed site. The basin is located at the site legal point of discharge (LPOD) in the north-west corner. The modelled bas as depicted in Figure 7.5.

Figure 7.5: Filled area (red) and basin location

For the purpose of this study, the developable area of the site (red hatching) was simulated as being filled above the predicted 1% AEP flood top water level (6.91 m AHD). This results in the removal of possible storage of regional flooding.

The site stormwater mitigation facilities, i.e., the wetland and detention basin; have been designed to manage stormwater runoff from the developed site only, therefore, the basin/wetland area will no longer accommodate regional flood flows.

For the purpose of this study a reserve was allocated for the temporary storage and conveyance of regional flood flows. The reserve is situated in the south west corner of the site and extends along Melaluka Road. It is expected that reserve size can be refined during detailed design.

Developed condition mapping is shown in Figure 7.9 to Figure 7.11.

For full set of flood depth envelopes for the 2%, 5%, 10%, 20%, and 50% AEP events, refer Appendix B (Section 9.2).

7.3.3 Flood Impact Mapping

The change in the flood level of the entire study area between the existing and developed conditions is shown for the 1% AEP critical duration event in Figure 7.12.

The reserve allows consolidation of flood water within an area of <1,900 m². Under existing conditions the area of inundation is 3,843 m^2. The proposed development results in a 51% reduction in area of inundation.

However, flood storage volume has only been reduced by 23%, 217 m^3 to 167 m^3. It is expected that this can be further reduced with functional civil design of the proposed reserve.

The aim of the the flood impact assessement is to ensure that the proposed development will not create a negative impact to neighbouring properties. Therefore, the main focus will be within the critical area.

Changes in the flood levels for all the AEP events within the critical area are inllustrated in Figure 7.13 to Figure 7.18. It can be seen that all negative impacts have been confined within the site and the swale along the Melaluka Rd.

No impact has been found on neighbour and downstream area.

7.3.4 Flood Hazard Mapping

The hazard maps have been created in accordance with the Safety and Hazard Criteria defined by ARR2016[23], which state that flow velocity, depths and the product of velocity and depth must not exceed safety limits for people and vehicle access (egress) to (from) the site. The criteria are as follows:

1. Site Safety (People)
   - Depth must be no greater than or equal to 0.5 metres;
   - Velocity must be no greater than or equal to 3.0 m/s; and
   - The product of depth multiplied by velocity must be no greater than or equal to 0.4 m²/s.
2. Access Safety (Vehicles)
   - Depth must be no greater than or equal to 0.3 metres;
   - Velocity must be no greater than or equal to 3.0 m/s; and
   - The product of depth multiplied by velocity must be no greater than or equal to 0.3 m²/s.

As safety for vehicles sets the lower threshold on acceptable hazard limits, it will be adopted in this study as the safety criteria.

The overall hazards, with the consideration of all the criteria listed above, related to stormwater runoff generated during a predicted 1% AEP critical duration storm event for existing and developed conditions are presented in Figure 7.19 to Figure 7.22.

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[23] Smith, G, Cox, R, 2016. Safety Design Criteria. Chapter 7 Book 6 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

It can be seen that the developed site and road access meet the safety criteria. Therefore, the increase in flood levels within the swale and reserved areas observed in the flood impact maps does not create an increase of extension in flood hazards.

7.3.5 Sensitive area

The outlined site stormwater management strategy enables the proposed development works to be undertaken without generating adverse impacts on flood characteristics around the childcare - “Leopold World of Learning”. Site development does not create additional flood related risks for the childcare.

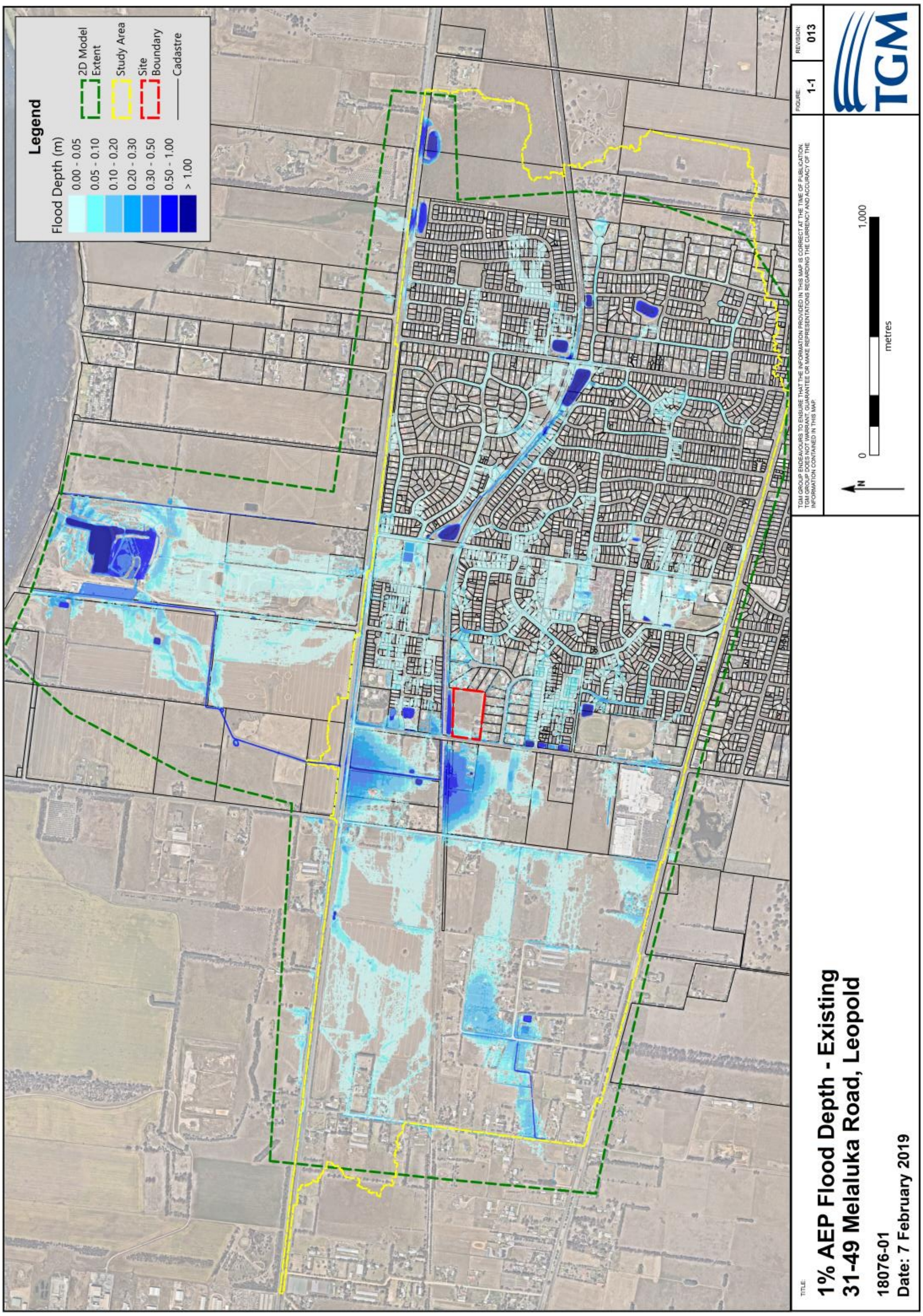


Figure 7.6: Existing Conditions - 1% AEP Flood Extent and Depths

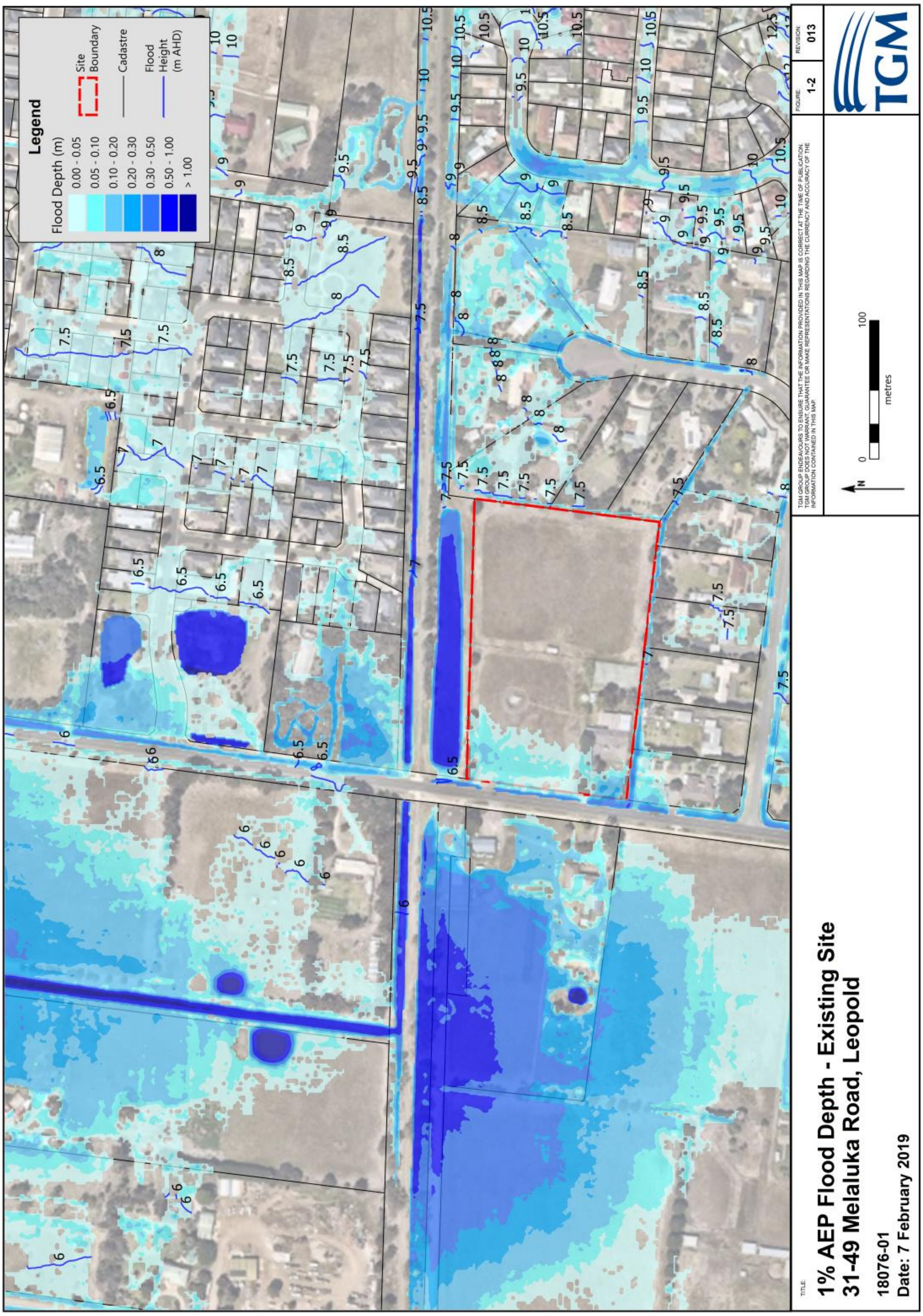


Figure 7.7: Existing Conditions - 1% AEP Flood Extent, Height and Depths (site area)

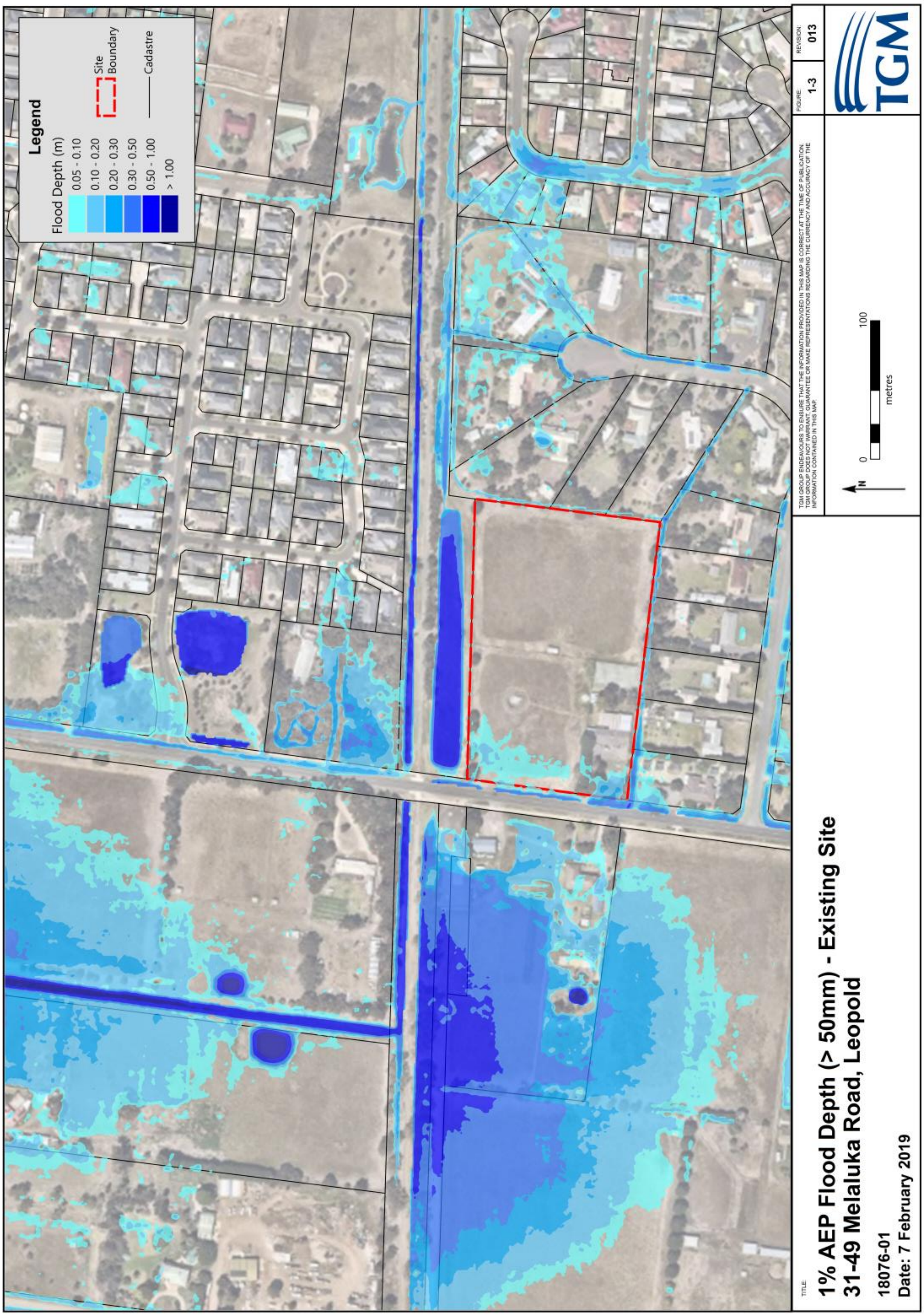


Figure 7.8: Existing Conditions - 1% AEP Flood Extent and Depths (depths ≥ 50 mm)

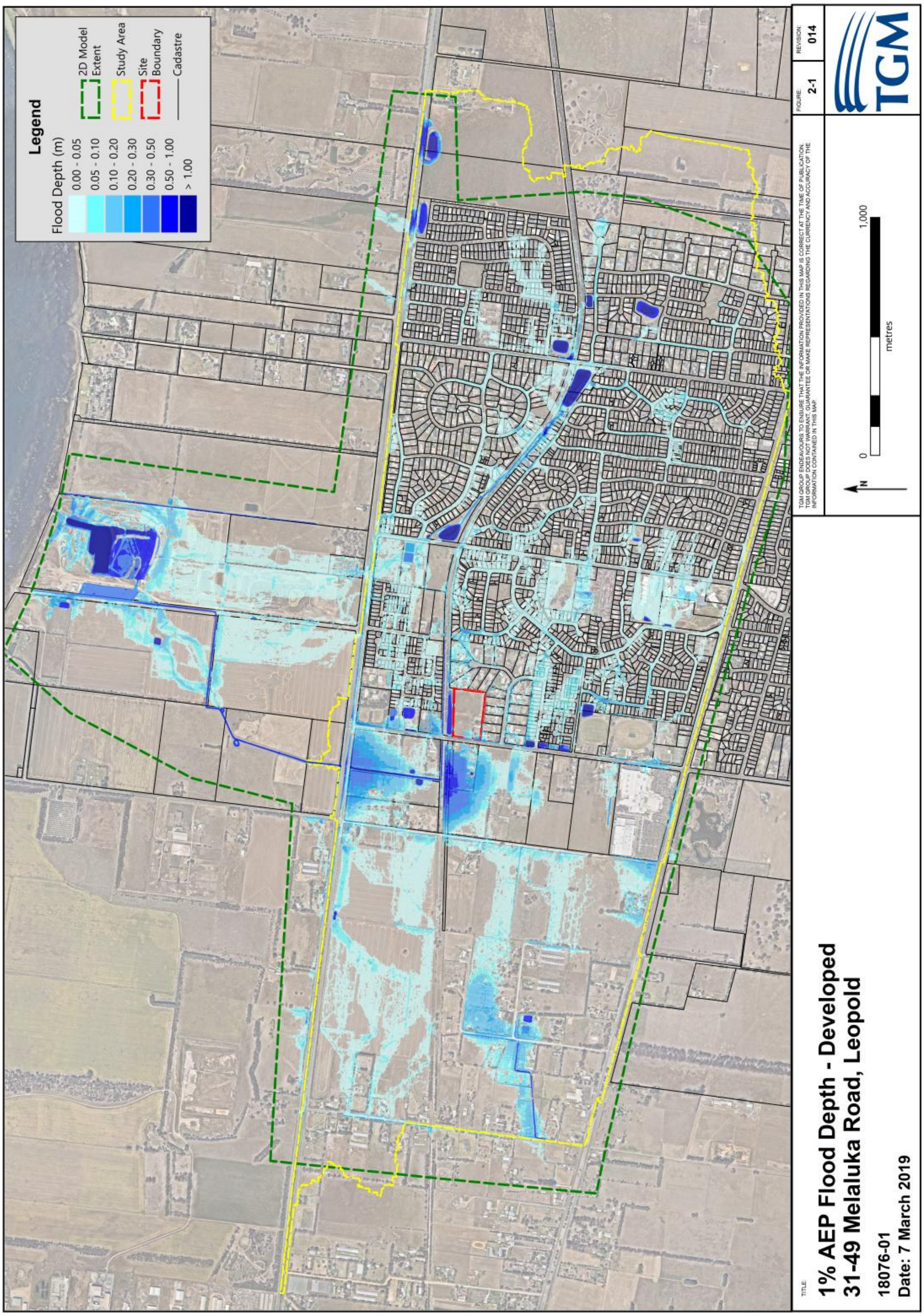


Figure 7.9: Developed Conditions - 1% AEP Flood Extent and Depths

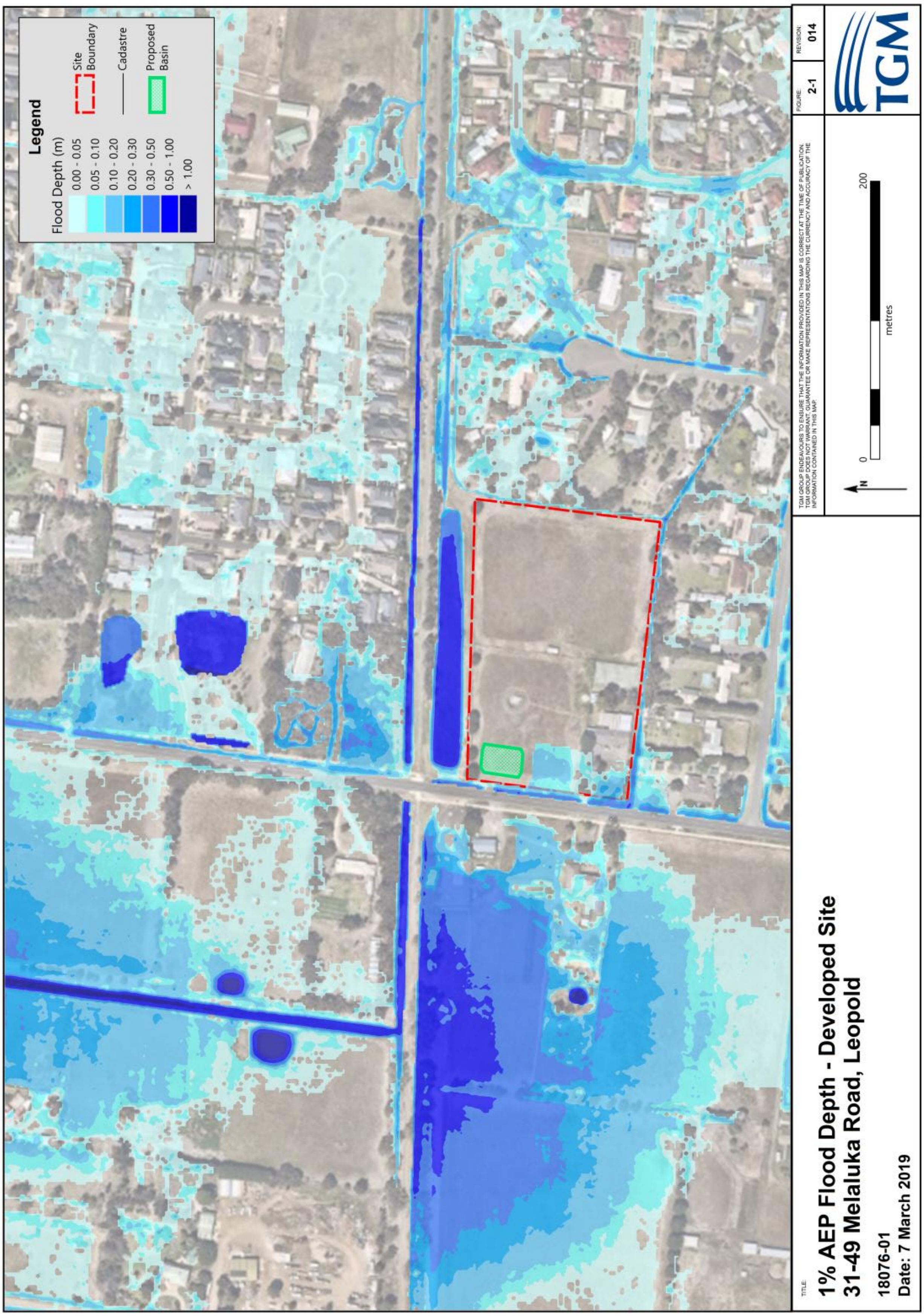


Figure 7.10: Developed Conditions - 1% AEP Flood Extent and Depths (site area)

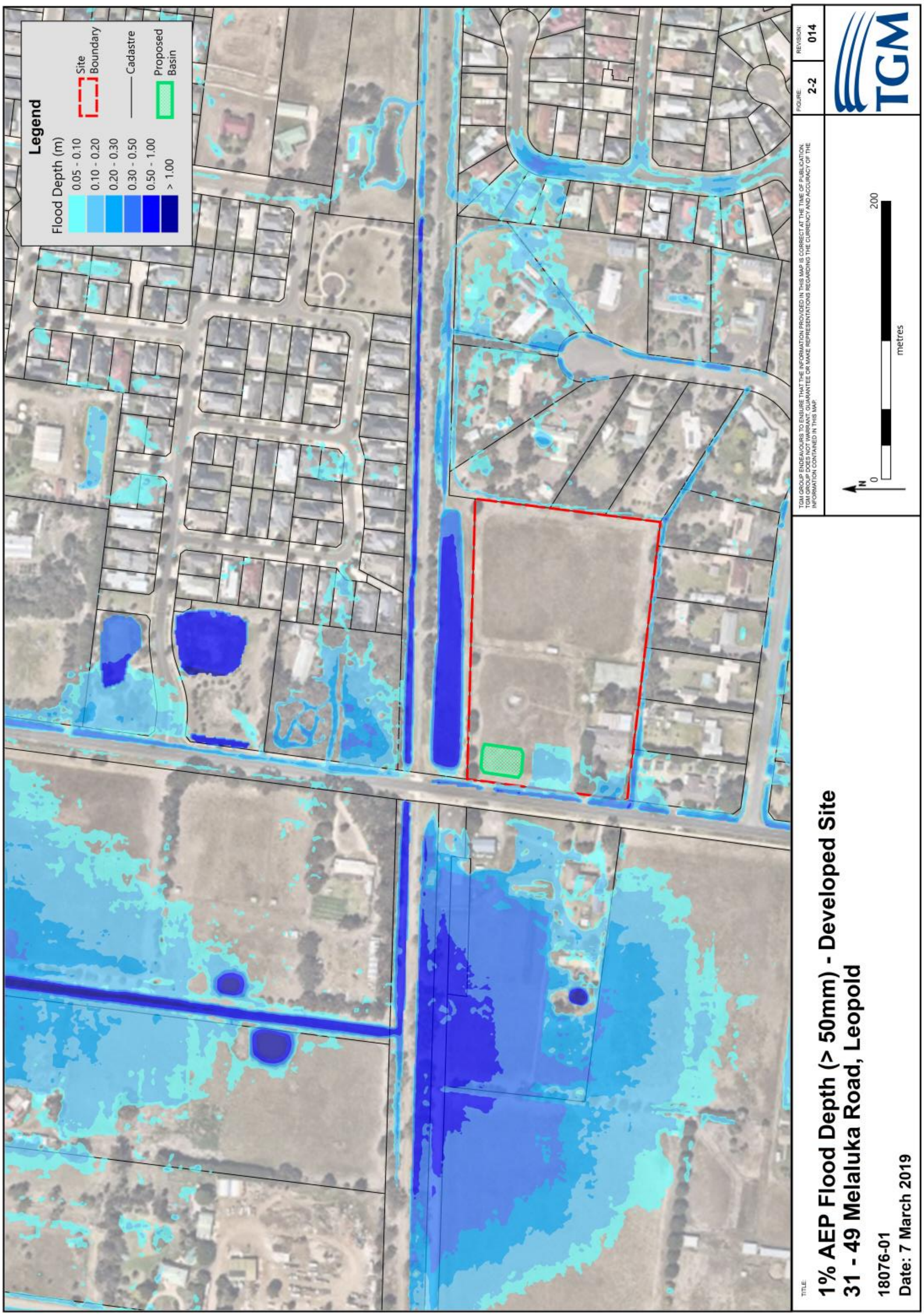


Figure 7.11: Developed Conditions - 1% AEP Flood Extent and Depths (depths ≥ 50 mm, site area)

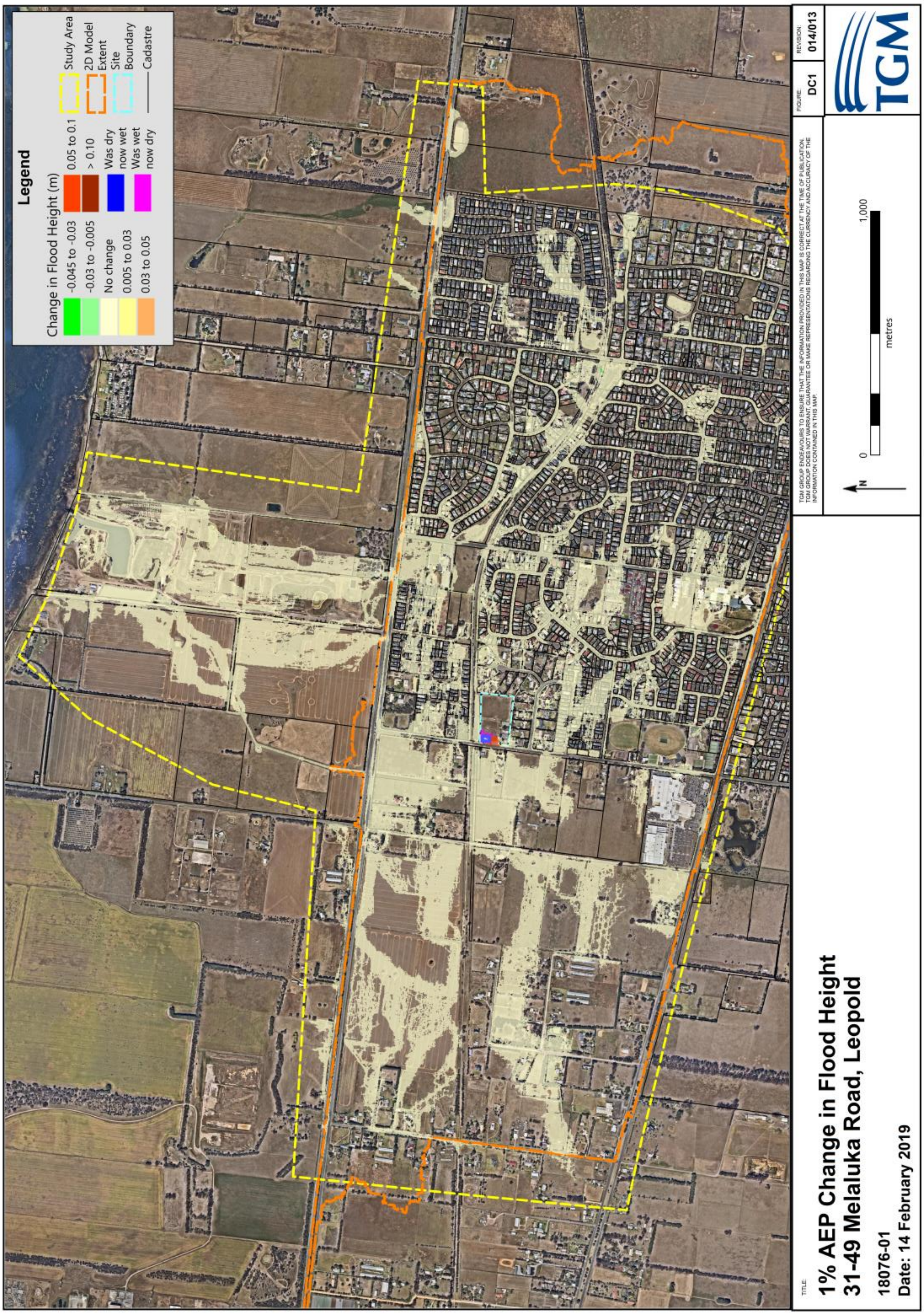


Figure 7.12: Change in Flood Levels - 1% AEP Impact Map

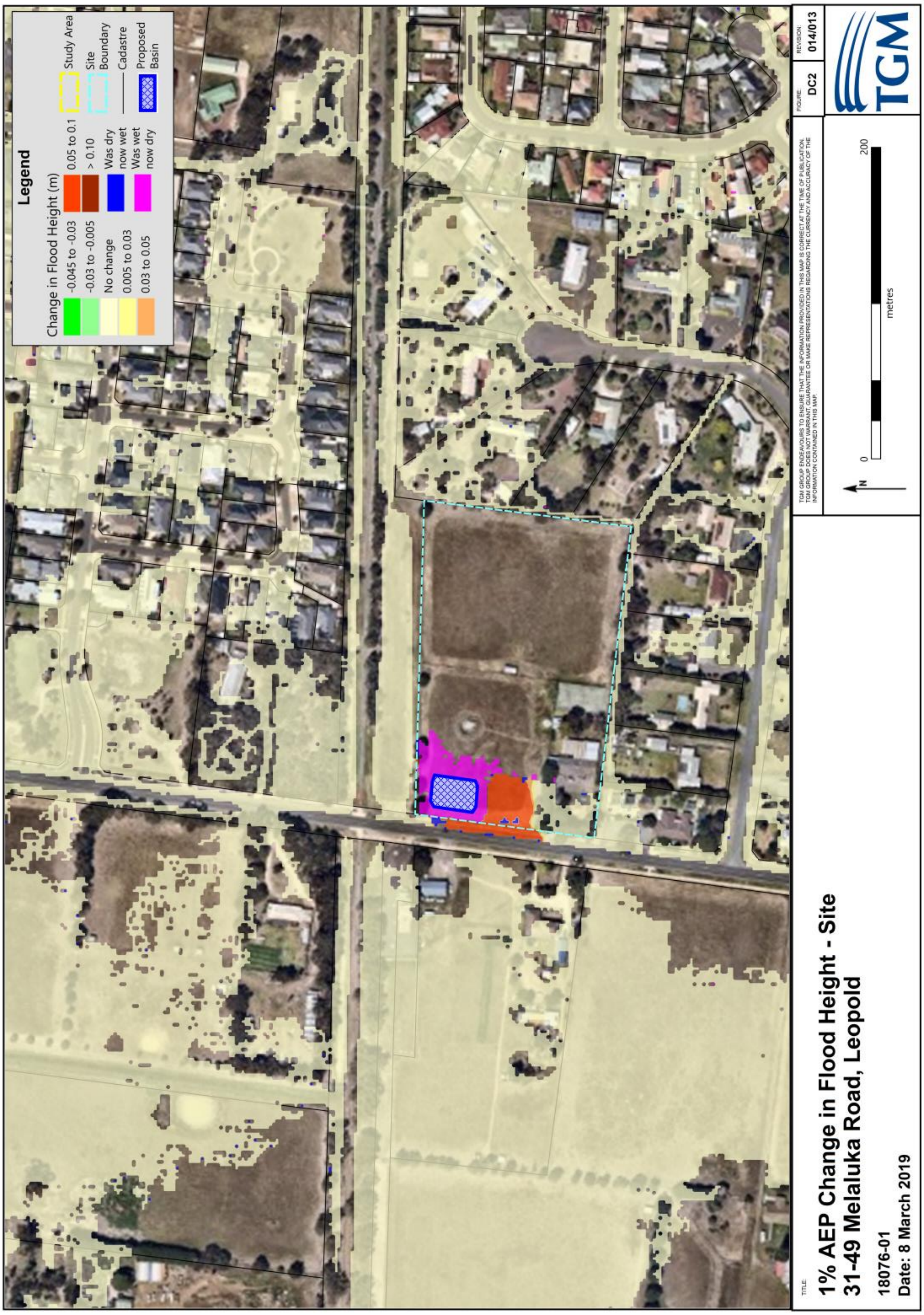


Figure 7.13: Change in Flood Levels - 1% AEP Impact Map (site area)

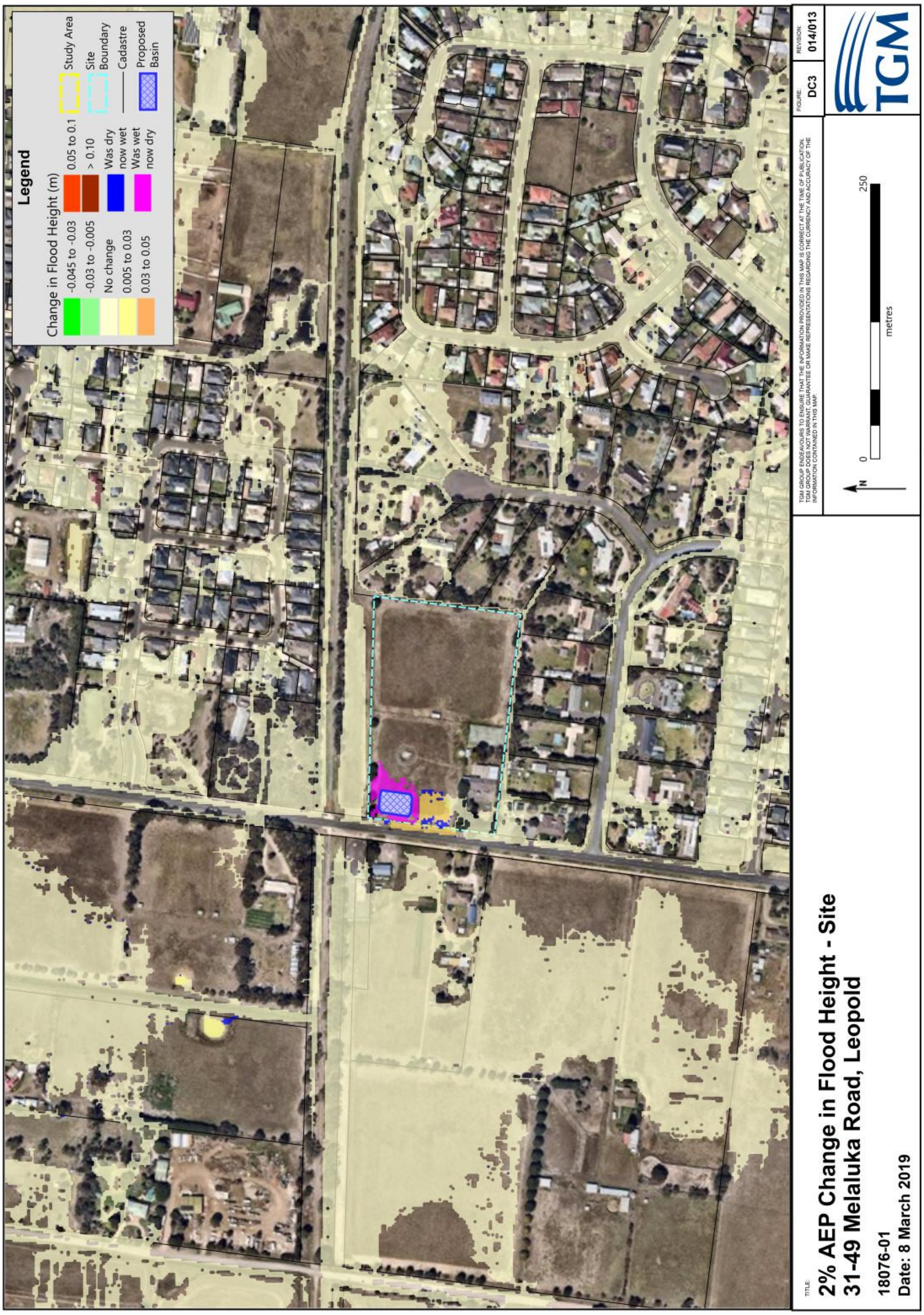


Figure 7.14: Change in Flood Levels - 2% AEP Impact Map (site area)

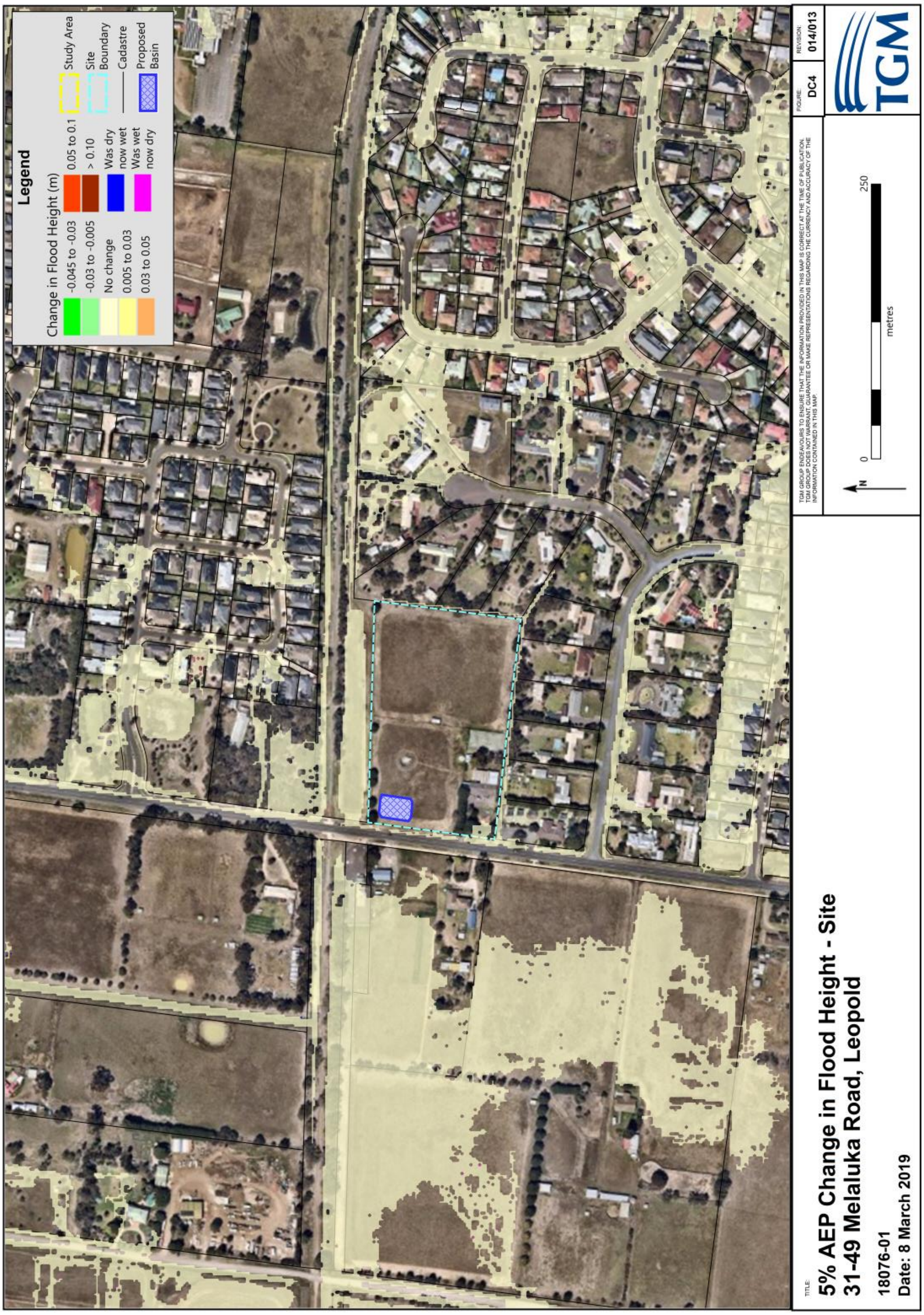


Figure 7.15: Change in Flood Levels - 5% AEP Impact Map (site area)

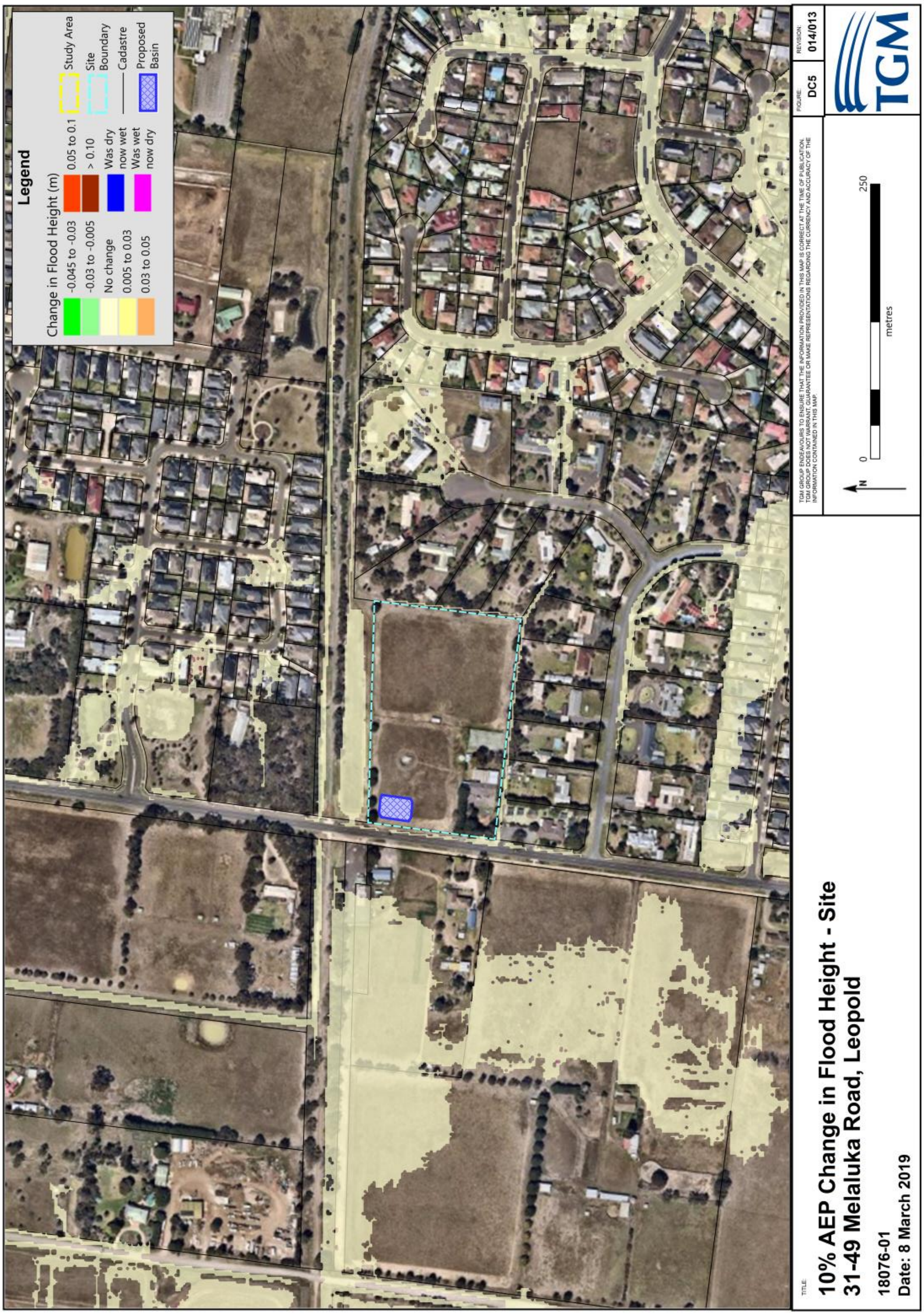


Figure 7.16: Change in Flood Levels - 10% AEP Impact Map (site area)

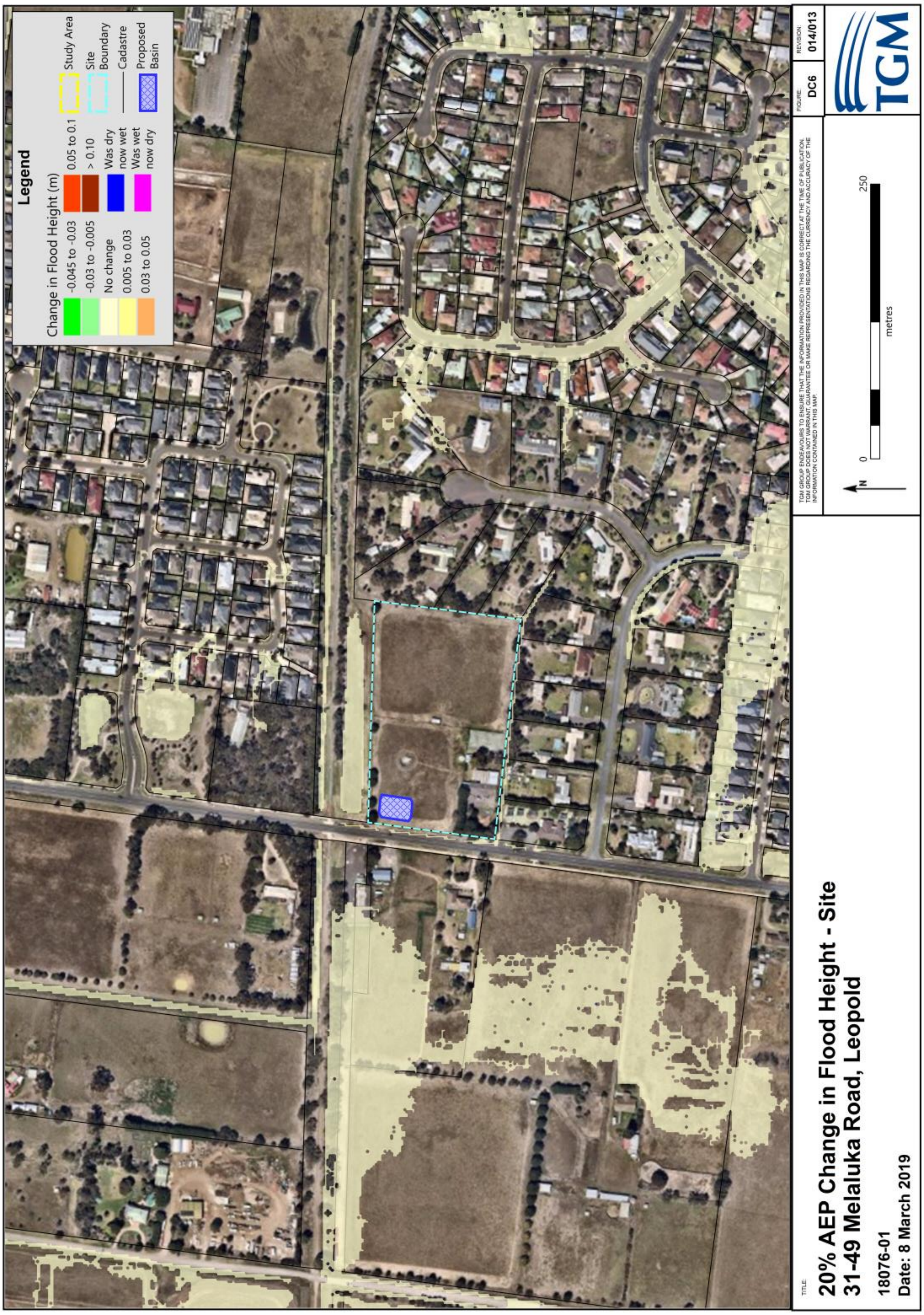


Figure 7.17: Change in Flood Levels - 20% AEP Impact Map (site area)

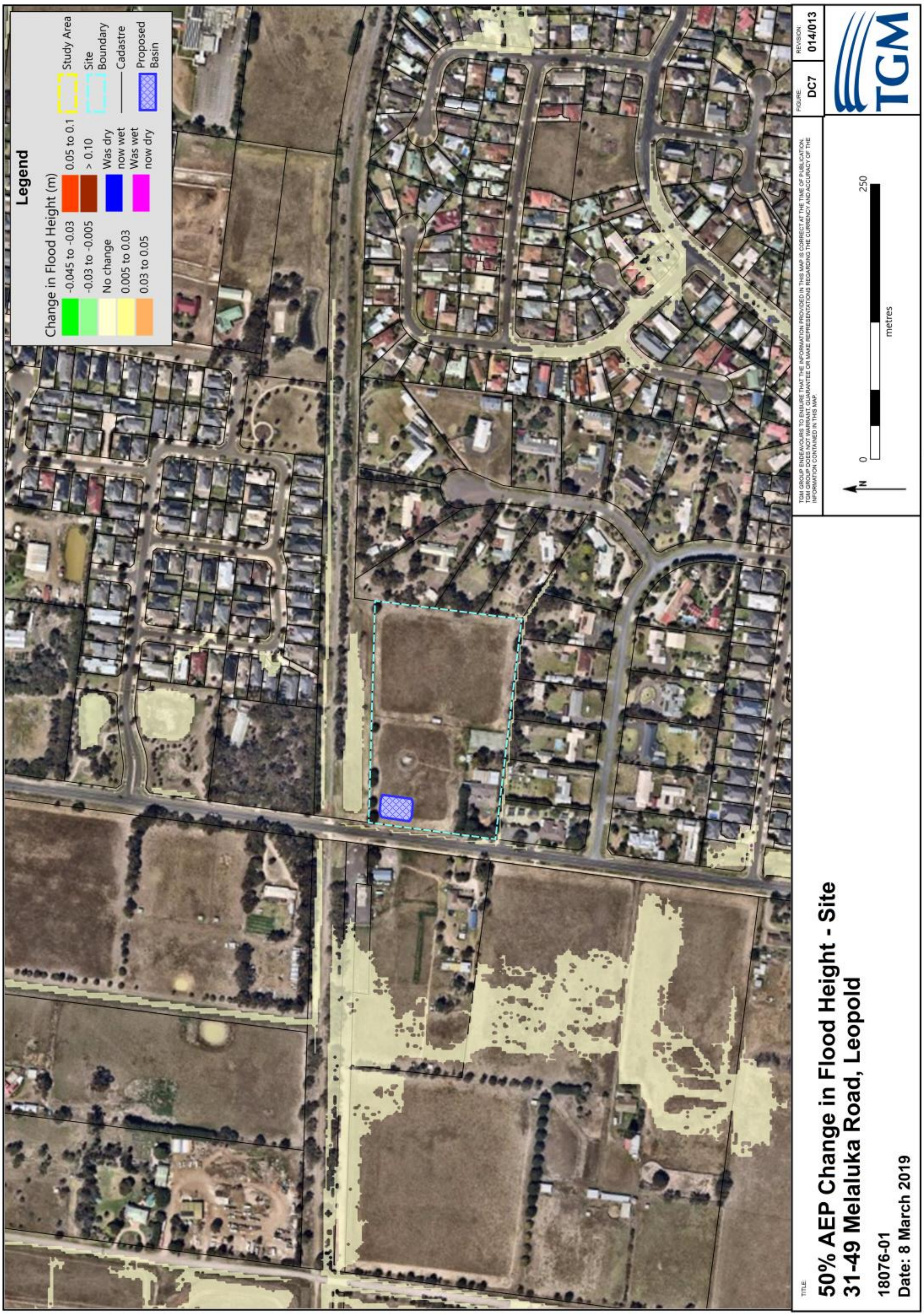


Figure 7.18: Change in Flood Levels - 50% AEP Impact Map (site area)

Figure 7.19: Existing Conditions - 1% AEP Hazard Envelope (site area)

Figure 7.20: Existing Conditions - 1% AEP Hazard Envelope (access and egress)

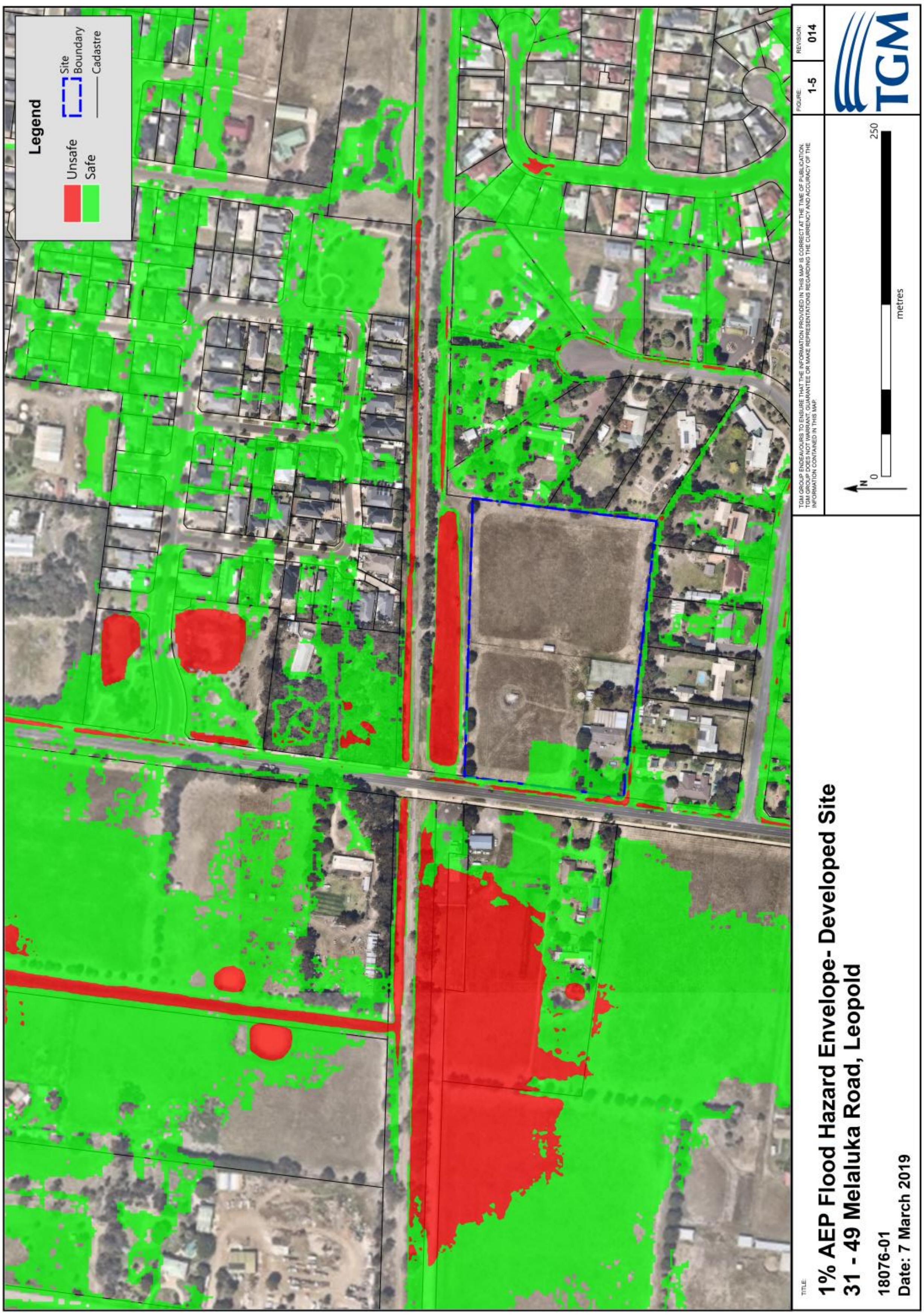


Figure 7.21: Developed Conditions - 1% AEP Hazard Envelope

Figure 7.22: Developed Conditions - 1% AEP Hazard Envelope (critical area)

8. CONCLUSIONS

A flood impact assessment and site stormwater management plan has been developed for the proposed subdivision of in 31-49 Melaluka Road, Leopold.

A regional distributed hydrological model has been set up with XP-STORM using rainfall and flood estimation techniques consistent with ARR2016 current industry best practice, to define the runoff hydrographs for the 1%, 2%, 5%, 10%, 20%, and 50% AEP events with a range of critical storm durations.

A regional two-dimensional (2D) hydraulic model (with 1D underground drainage and channel network elements) has been set up with TUFLOW to predict an accurate representation of the regional flood extents impacting the site during all the AEP storm events for both the existing and developed conditions.

A local hydrological model and water quality model have been set up with XP-STORM and MUSIC, respectively, for wetlands and detention basins design to ensure the site stormwater quantity and quality meeting the site stormwater objectives.

To prevent the developed site from flood inundation, it is proposed that part of the site (marked with red in Figure 7.5) is filled to 6.91 m AHD, leaving a drainage reserve along the frontage of Melaluka Road.

The reserve allows consolidation of flood water within an area of <1,900 m². The proposed development mitigation results in a 51% reduction in area of inundation within the site.

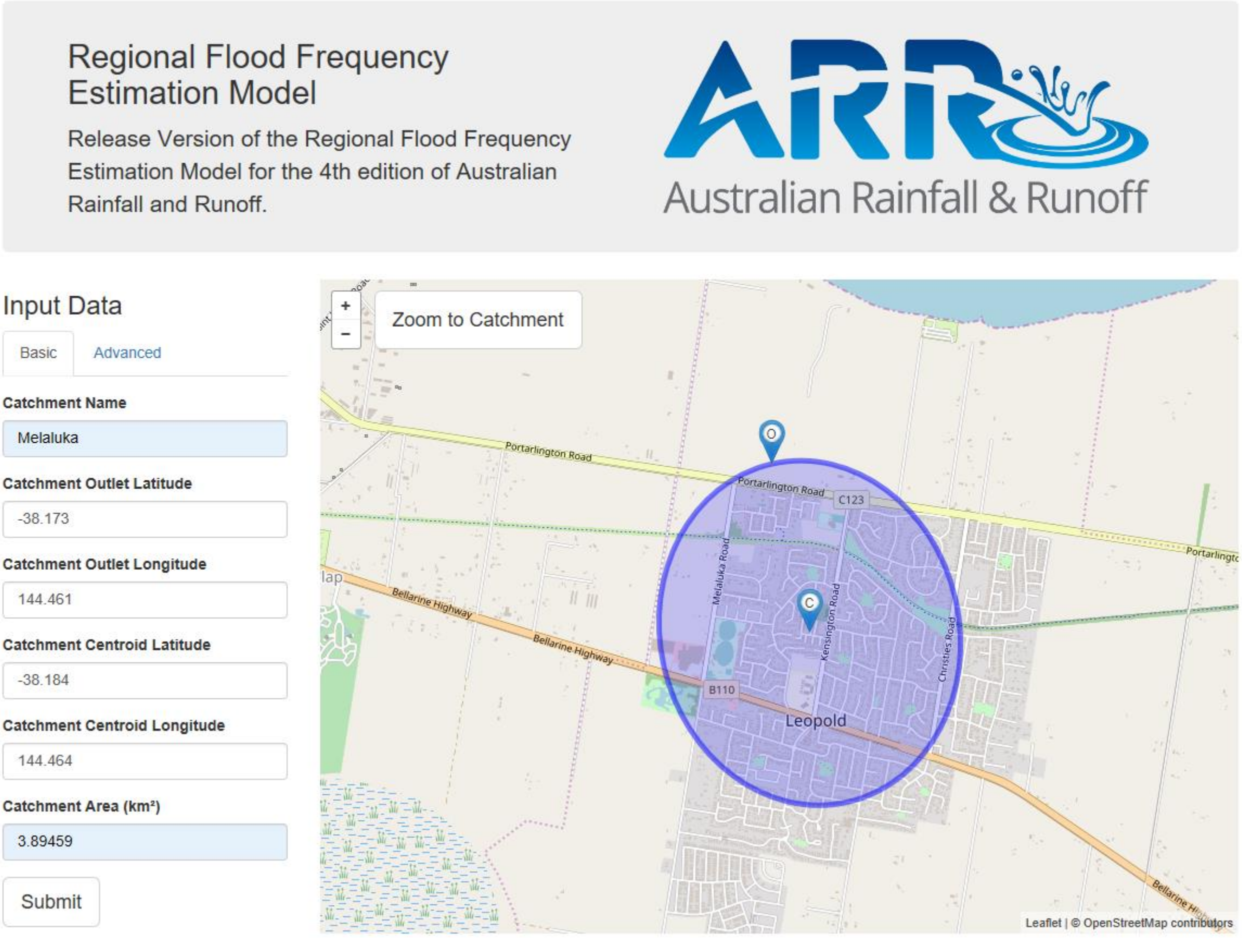
The site stormwater quality and quantity objectives for the proposed development can be achieved using end-of-line wetland and detention basin at the northwest corner of the site with a footprint of 1,560.6 m^2.

Based on the flood impact assessment results, it can be concluded that the proposed development, with appropriate SSMP as suggested in this report, will have no adverse impacts on flood characteristics external to the site.

In conclusion, the analysis undertaken in this study has demonstrated that the proposed development can be constructed and meet the requirements and objectives for site stormwater management, flood impact and safety and egress during floods.

9. APPENDIX

9.1 Appendix A: Regional Flood Frequency Estimation Model

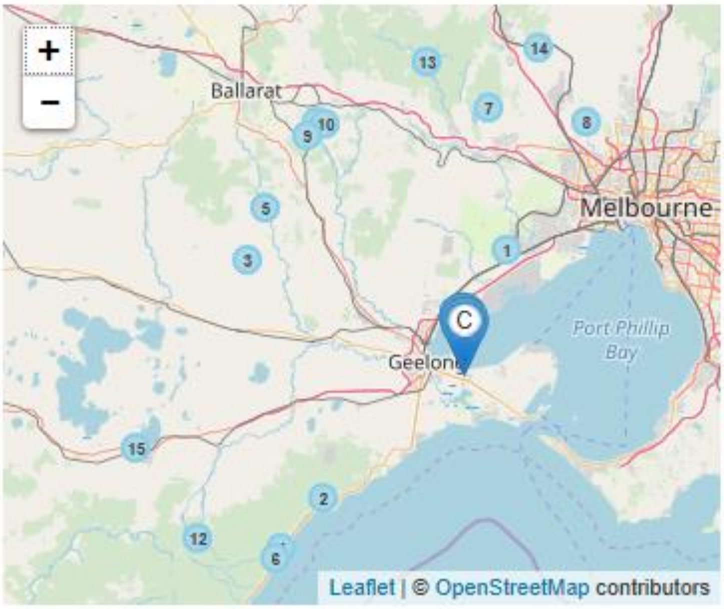


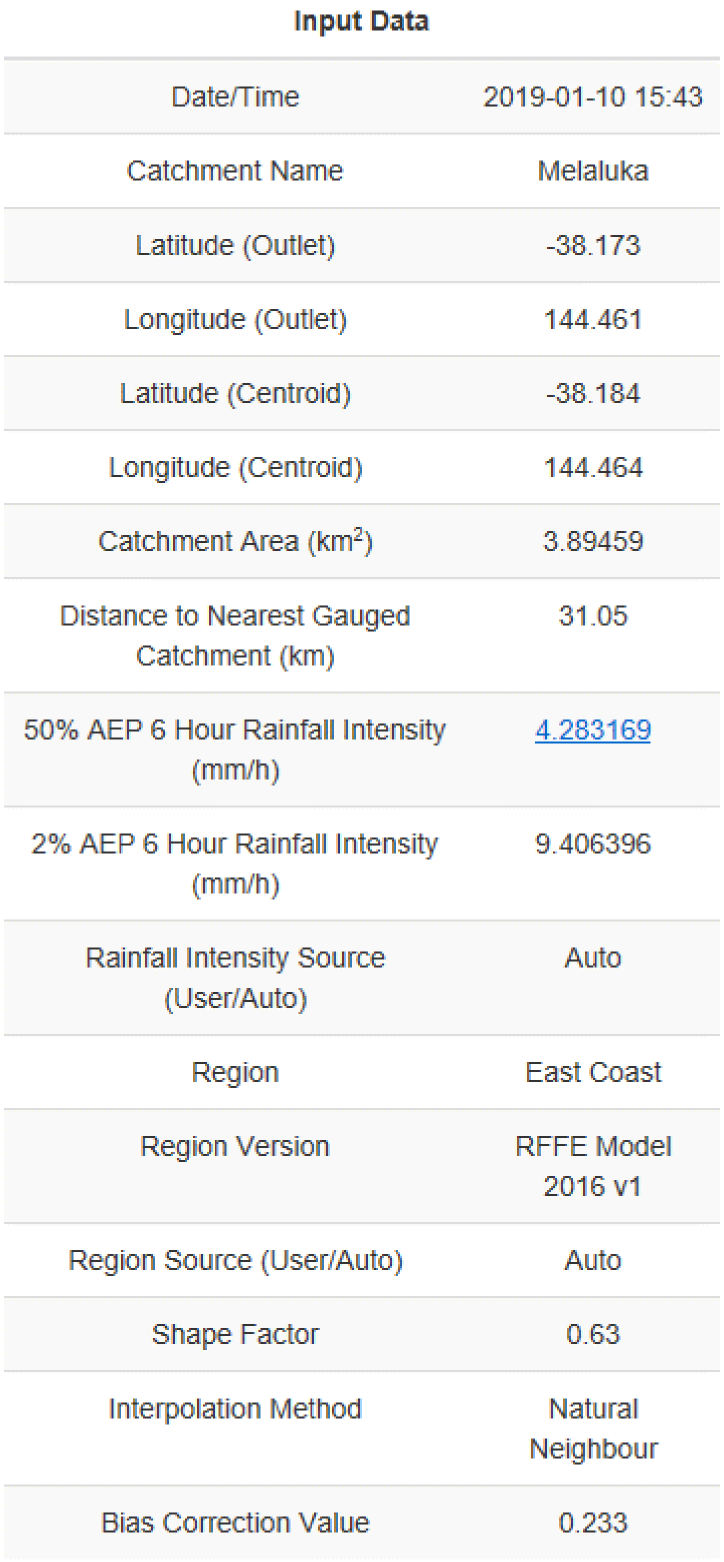
Regional Flood Frequency Estimation Model

Release Version of the Regional Flood Frequency Estimation Model for the 4th edition of Australian Rainfall and Runoff.

Input Data

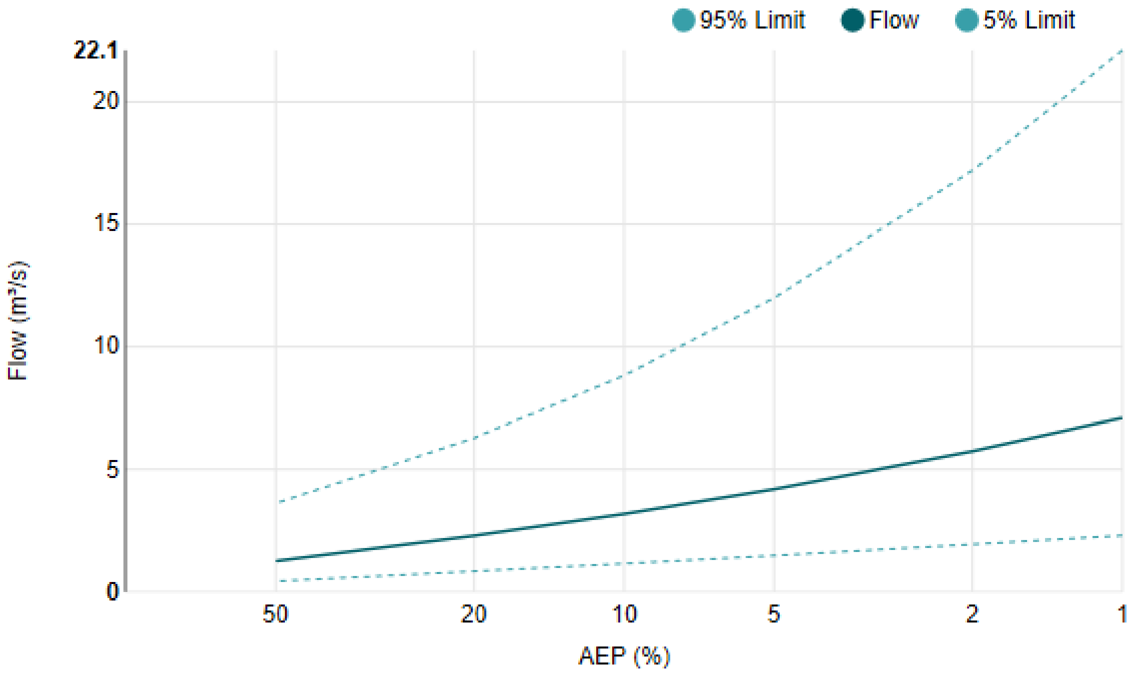
Catchment Name	Melaluka
Catchment Outlet Latitude	-38.173
Catchment Outlet Longitude	144.461
Catchment Centroid Latitude	-38.184
Catchment Centroid Longitude	144.464
Catchment Area (km²)	3.89459





Input Data

Date/Time	2019-01-10 15:43
Catchment Name	Melaluka
Latitude (Outlet)	-38.173
Longitude (Outlet)	144.461
Latitude (Centroid)	-38.184
Longitude (Centroid)	144.464
Catchment Area (km^2)	3.89459
Distance to Nearest Gauged Catchment (km)	31.05
50% AEP 6 Hour Rainfall Intensity (mm/h)	4.283169
2% AEP 6 Hour Rainfall Intensity (mm/h)	9.406396
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 v1
Region Source (User/Auto)	Auto
Shape Factor	0.63
Interpolation Method	Natural Neighbour
Bias Correction Value	0.233



AEP (%)	Discharge (m^3/s)	Lower Confidence Limit (5%) (m^3/s)	Upper Confidence Limit (95%) (m^3/s)
50	1.25	0.430	3.61
20	2.29	0.840	6.28
10	3.18	1.15	8.83
5	4.19	1.48	12.0
2	5.74	1.94	17.2
1	7.10	2.30	22.1

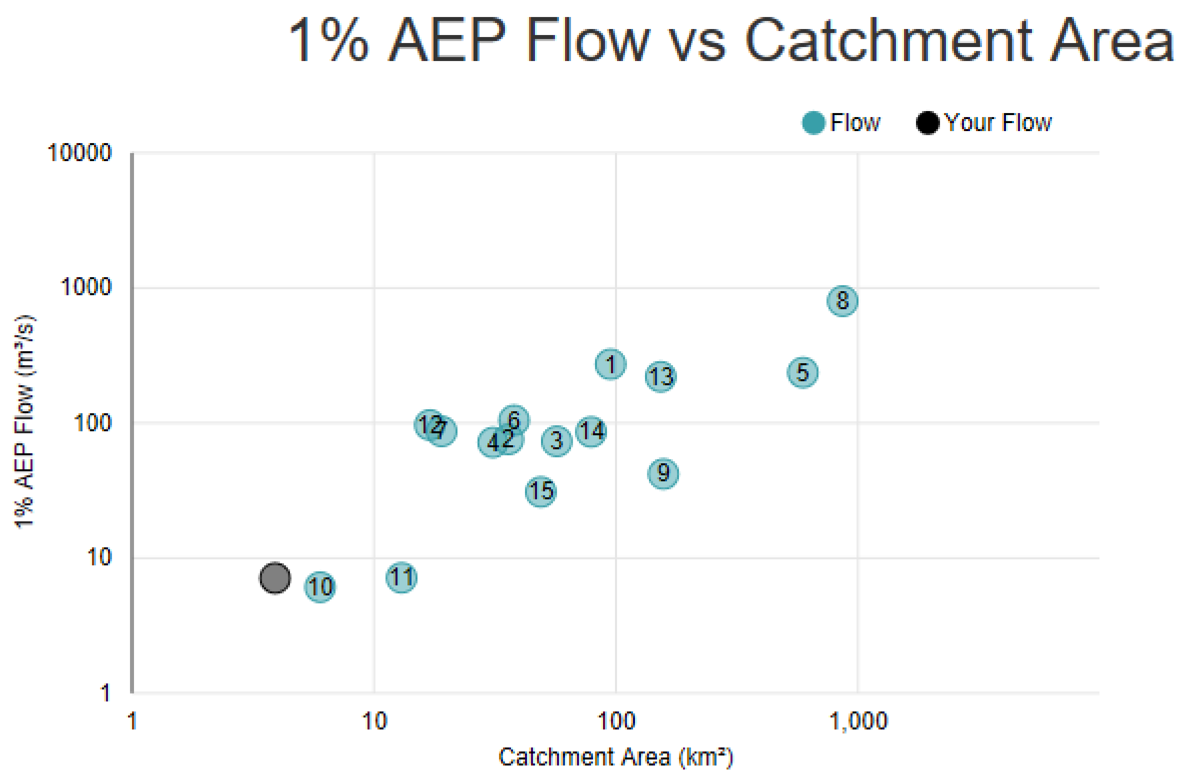
Statistics

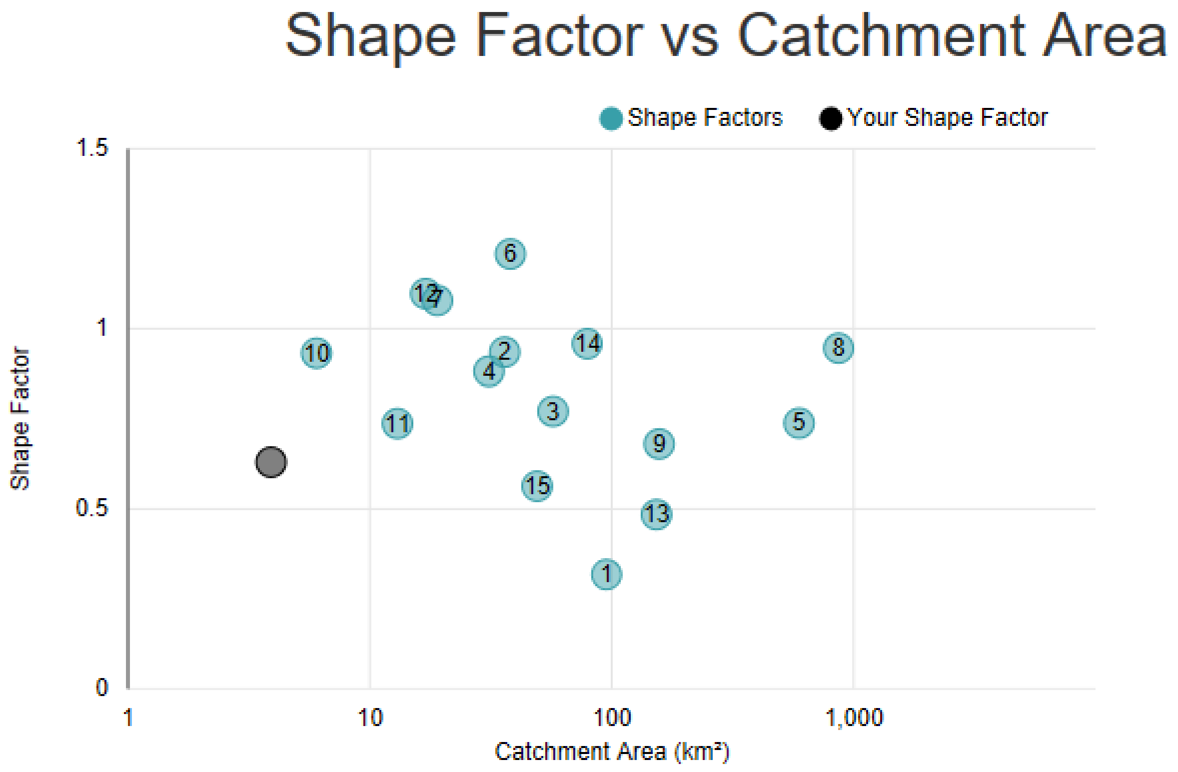
Variable	Value	Standard Dev
Mean	0.323	0.654
Standard Dev	0.664	0.217
Skew	0.141	0.030

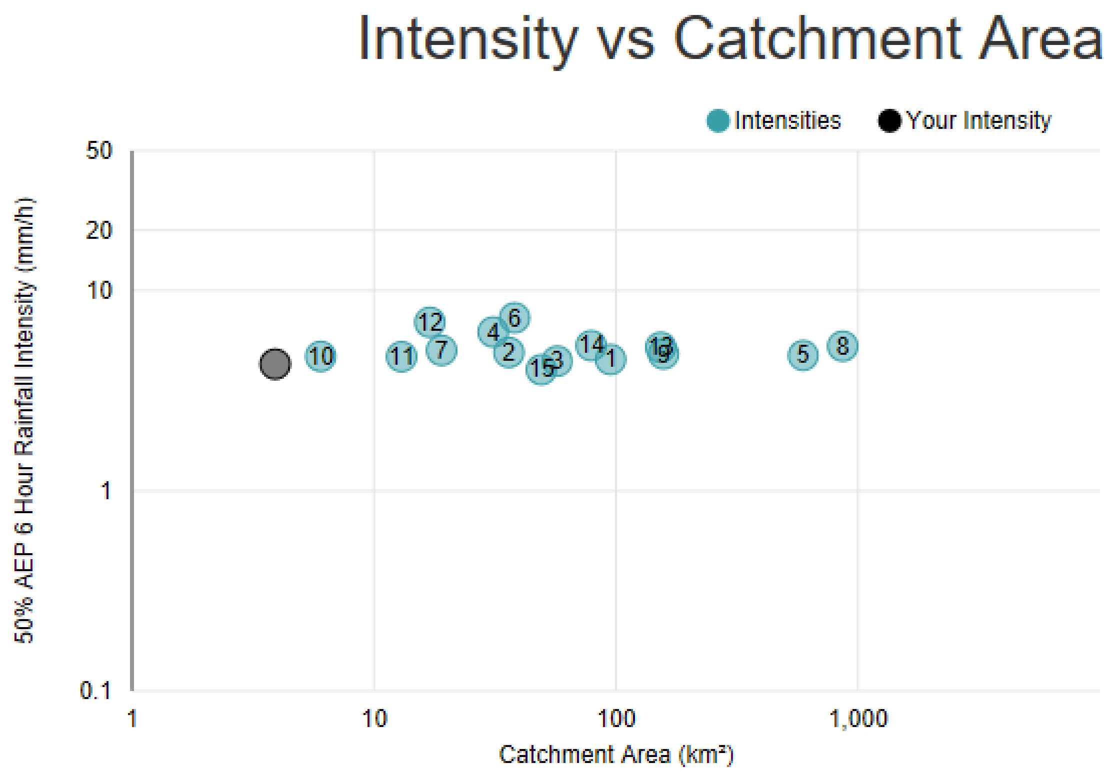
Note: These statistics come from the nearest gauged catchment. Details.

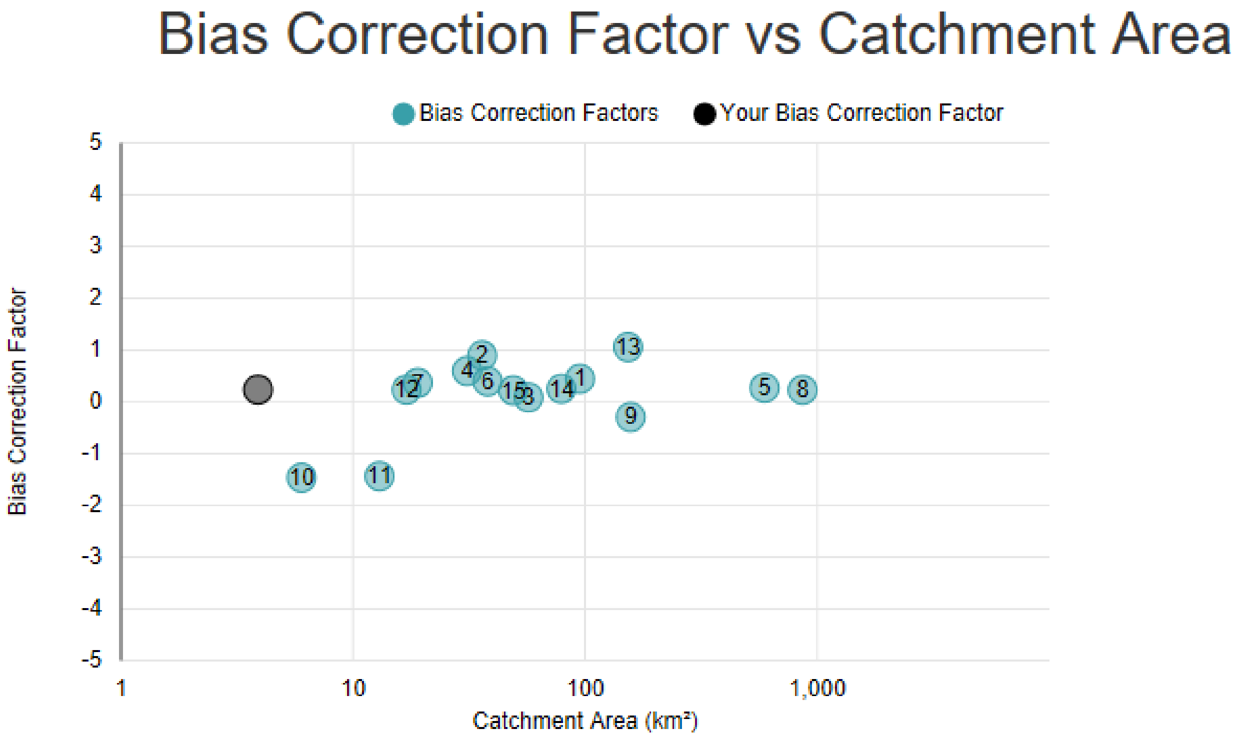
Correlation		
1.000		
-0.330	1.000	
0.170	-0.280	1.000

Note: These statistics are common to each region. Details.

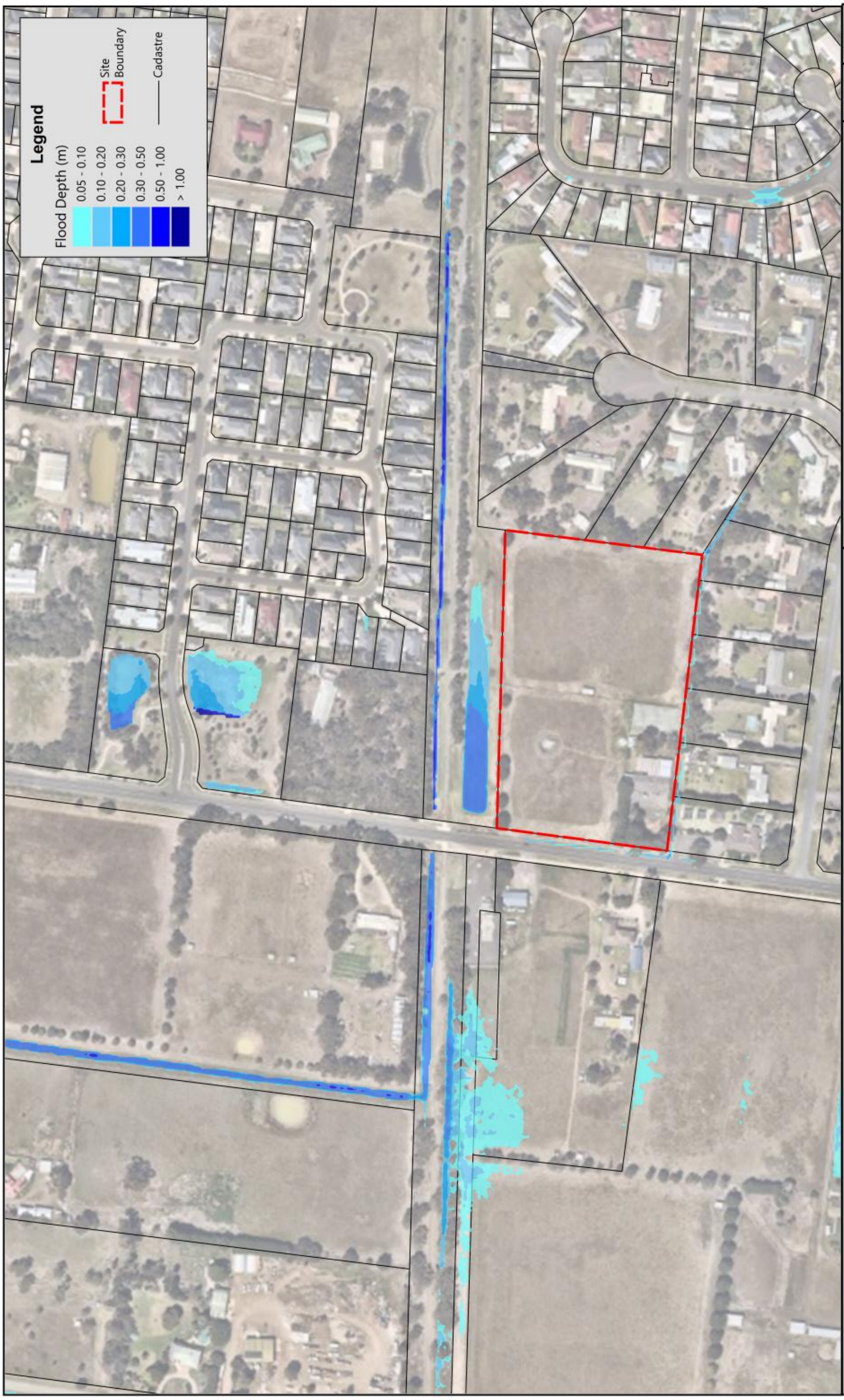

1% AEP Flow vs Catchment Area
Flow
Your Flow
1% AEP Flow (m³/s)
10000
1000
100
10
1
1
10
100
1,000
Catchment Area (km²)
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15


Shape Factor vs Catchment Area
Shape Factors
Your Shape Factor
Shape Factor
1.5
1
0.5
0
1
10
100
1,000
Catchment Area (km²)
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15


Intensity vs Catchment Area
Intensities
Your Intensity
50% AEP 6 Hour Rainfall Intensity (mm/h)
50
20
10
1
0.1
1
10
100
1,000
Catchment Area (km²)
10
11
12
7
4
6
2
15
3
14
1
13
9
5
8


Bias Correction Factor vs Catchment Area
Bias Correction Factors
Your Bias Correction Factor
Bias Correction Factor
5
4
3
2
1
0
-1
-2
-3
-4
-5
1
10
100
1,000
Catchment Area (km²)
10
11
12
7
4
2
6
15
3
14
1
13
9
5
8

9.2 Appendix B: Flood Extent Maps for Events up to 2% AEP (Existing and Developed)



TITLE:

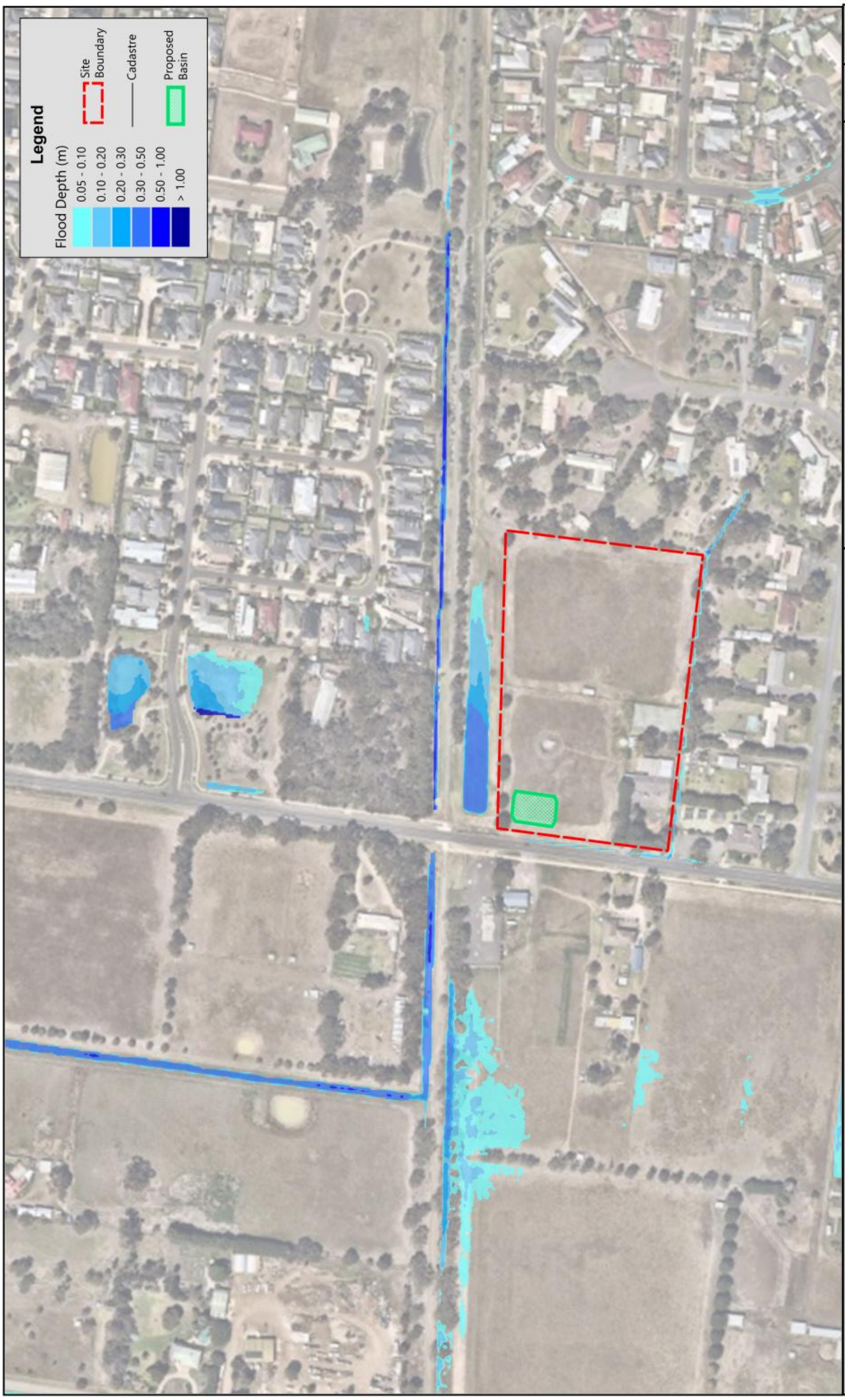
50% AEP Flood Depth (> 50mm) - Existing Site
31-49 Melaluka Road, Leopold

18076-01

Date: 7 February 2019

FIGURE: **50-1**

REVISION: **013**



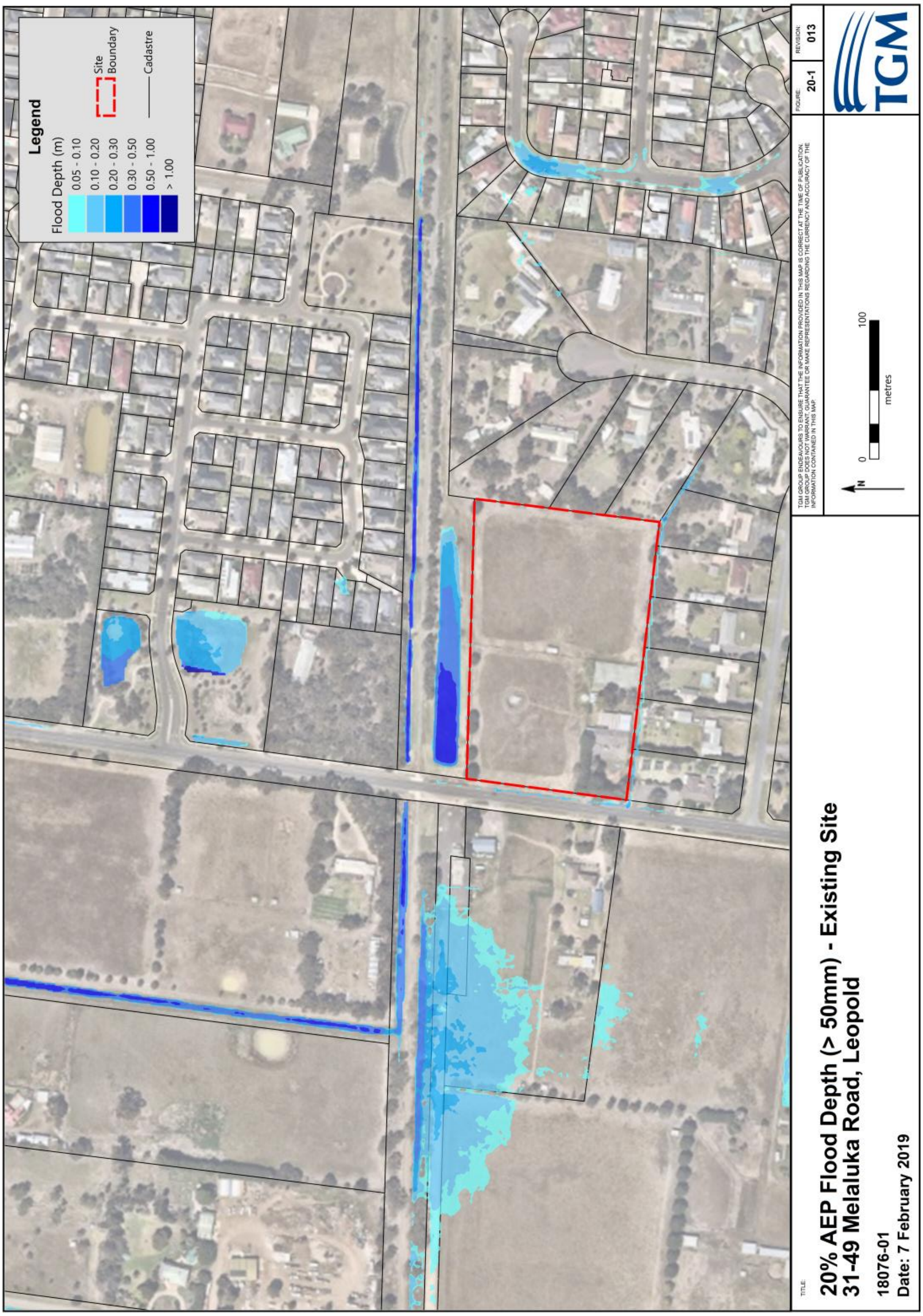
TITLE:

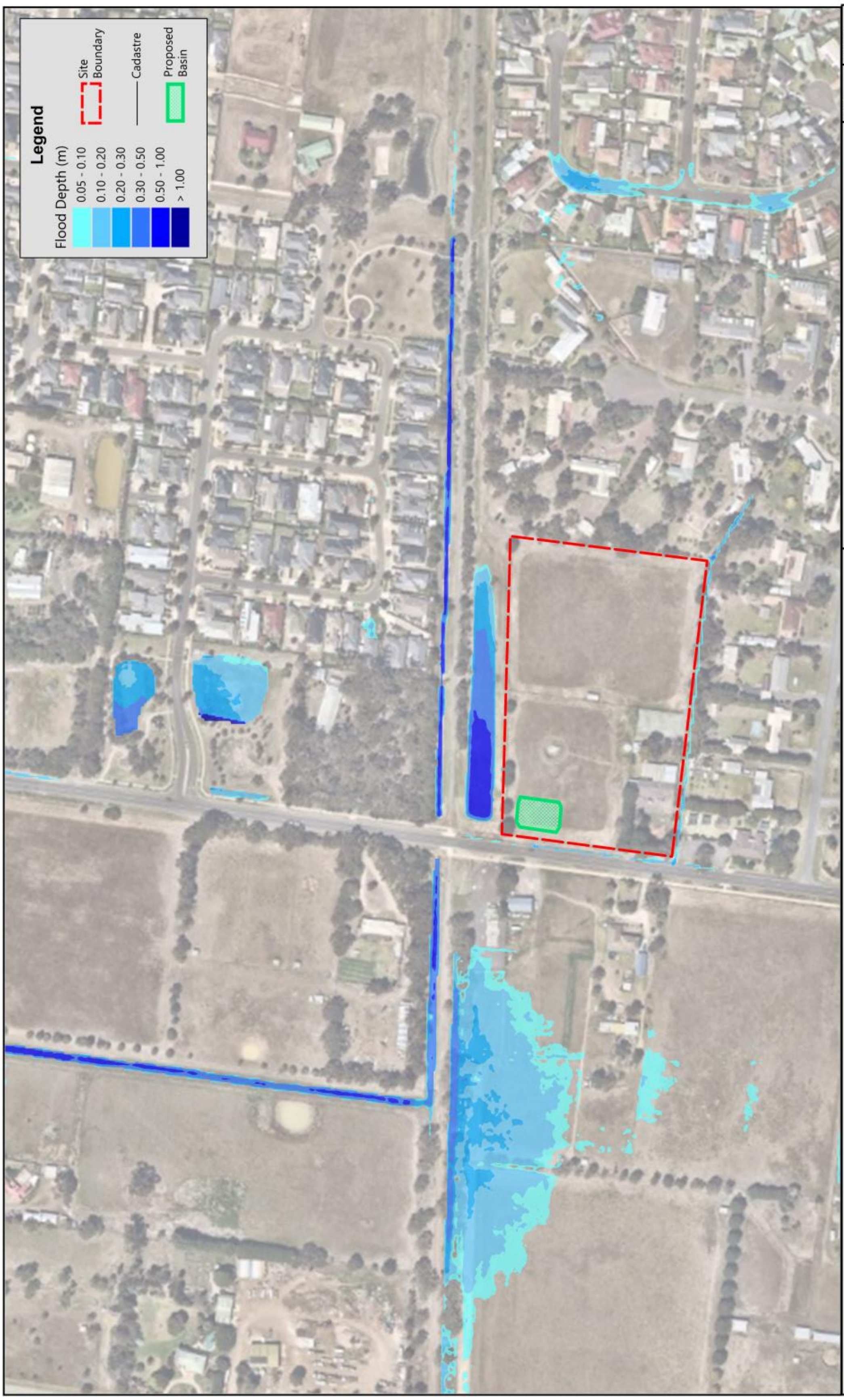
50% AEP Flood Depth (> 50mm) - Developed Site
31-49 Melaluka Road, Leopold

18076-01
Date: 7 March 2019

FIGURE: **2-7**
REVISION: **014**

TGM


Legend
Flood Depth (m)
0.05 - 0.10
0.10 - 0.20
0.20 - 0.30
0.30 - 0.50
0.50 - 1.00
> 1.00
Site Boundary
Cadastre
TGM GROUP ENDEAVOURS TO ENSURE THAT THE INFORMATION PROVIDED IN THIS MAP IS CORRECT AT THE TIME OF PUBLICATION. TGM GROUP DOES NOT WARRANT, GUARANTEE OR MAKE REPRESENTATIONS REGARDING THE CURRENCY AND ACCURACY OF THE INFORMATION CONTAINED IN THIS MAP.
FIGURE: 20-1
REVISION: 013
TGM
N
0
100
metres
TITLE:
20% AEP Flood Depth (> 50mm) - Existing Site
31-49 Melaluka Road, Leopold
18076-01
Date: 7 February 2019



TITLE:

20% AEP Flood Depth (> 50mm) - Developed Site
31-49 Melaluka Road, Leopold

18076-01

Date: 7 March 2019

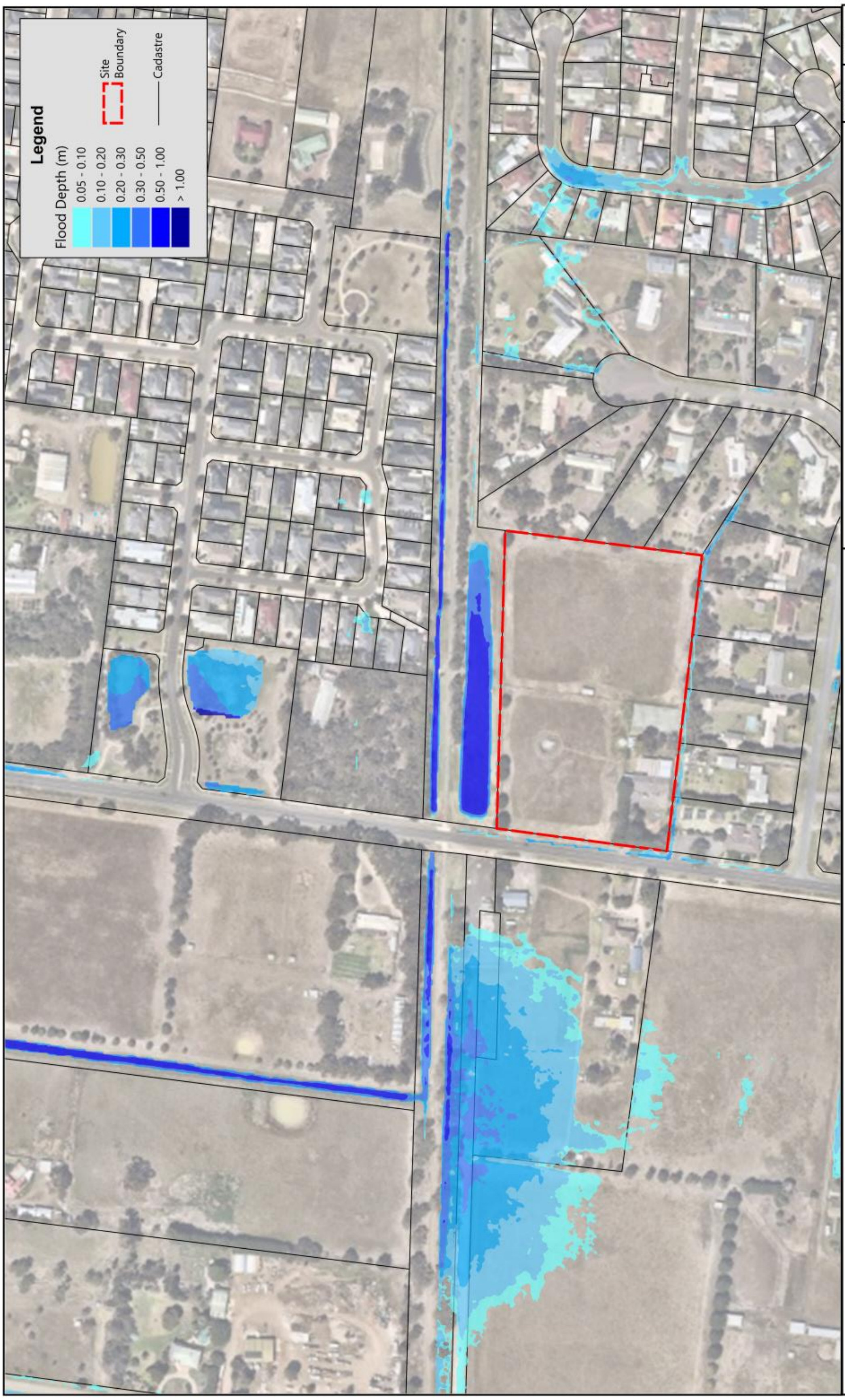
TGM GROUP ENDEAVOURS TO ENSURE THAT THE INFORMATION PROVIDED IN THIS MAP IS CORRECT AT THE TIME OF PUBLICATION. TGM GROUP DOES NOT WARRANT, GUARANTEE OR MAKE REPRESENTATIONS REGARDING THE CURRENCY AND ACCURACY OF THE INFORMATION CONTAINED IN THIS MAP.

N

0 200 metres

FIGURE: **2-6**

REVISION: **014**



TITLE:

10% AEP Flood Depth (> 50mm) - Existing Site
31-49 Melaluka Road, Leopold

18076-01
Date: 7 February 2019

FIGURE: **1-1**
REVISION: **013**

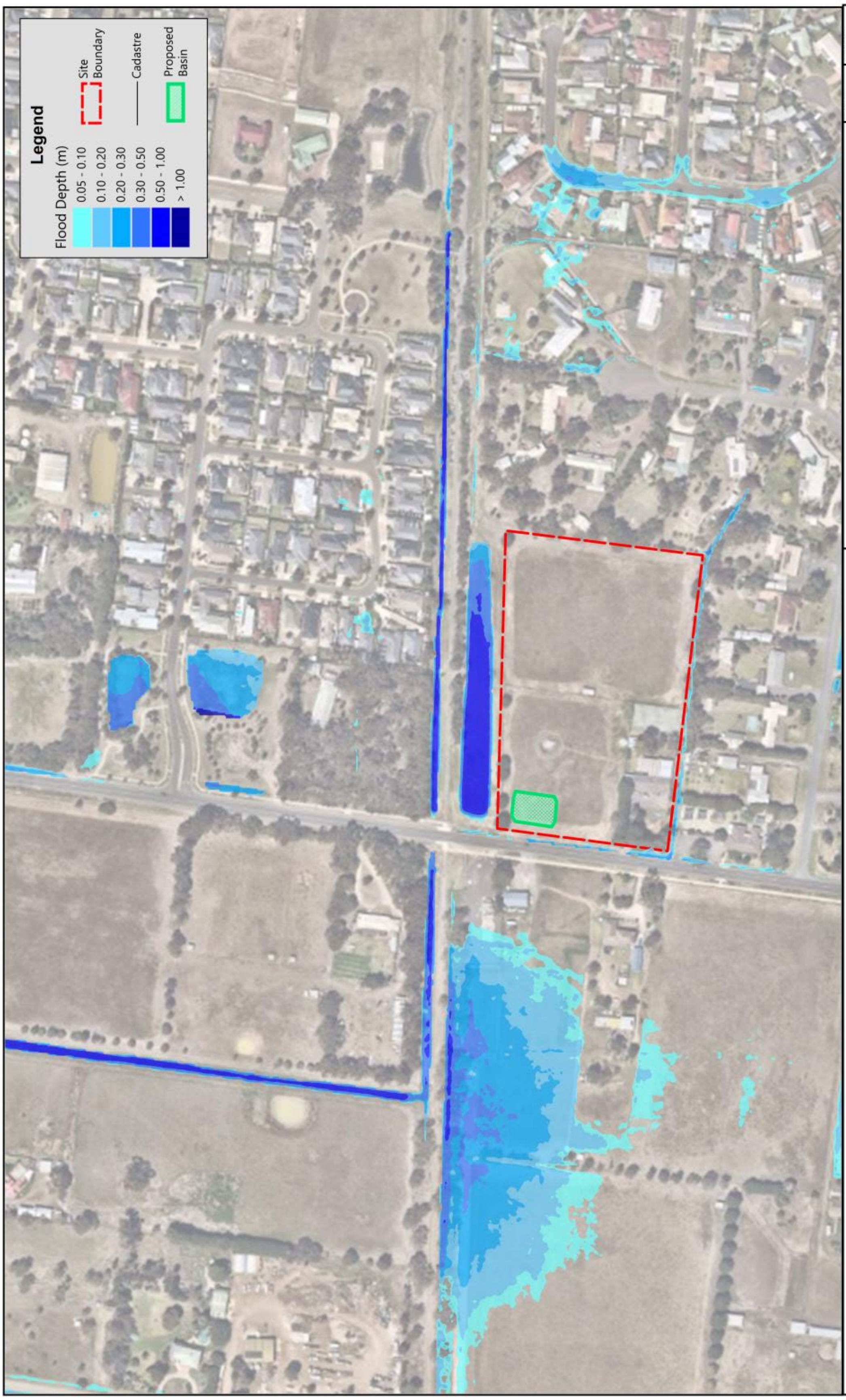
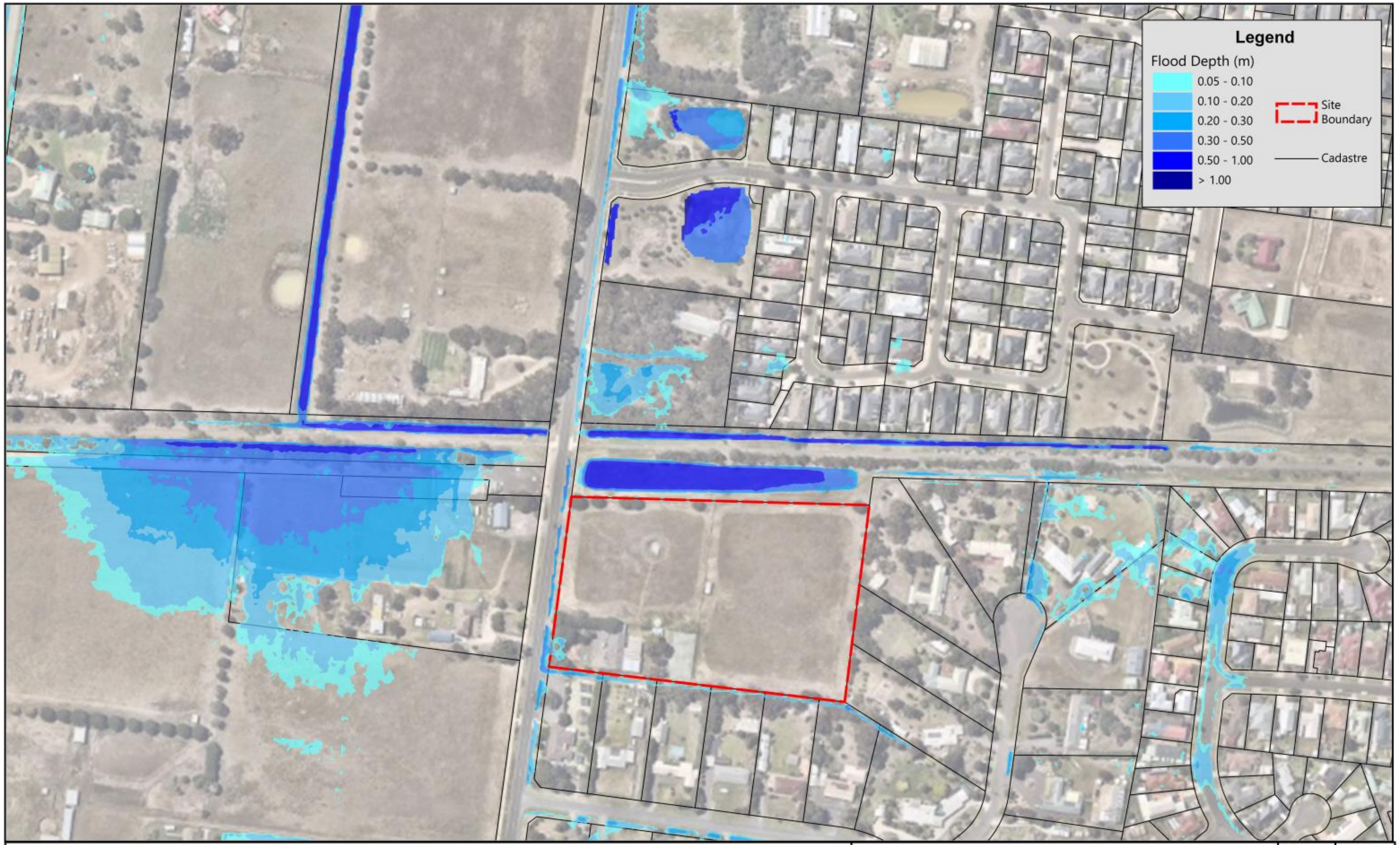


FIGURE: 2-5
REVISION: 014

TITLE:
10% AEP Flood Depth (> 50mm) - Developed Site
31-49 Melaluka Road, Leopold

18076-01
Date: 7 March 2019



TITLE:

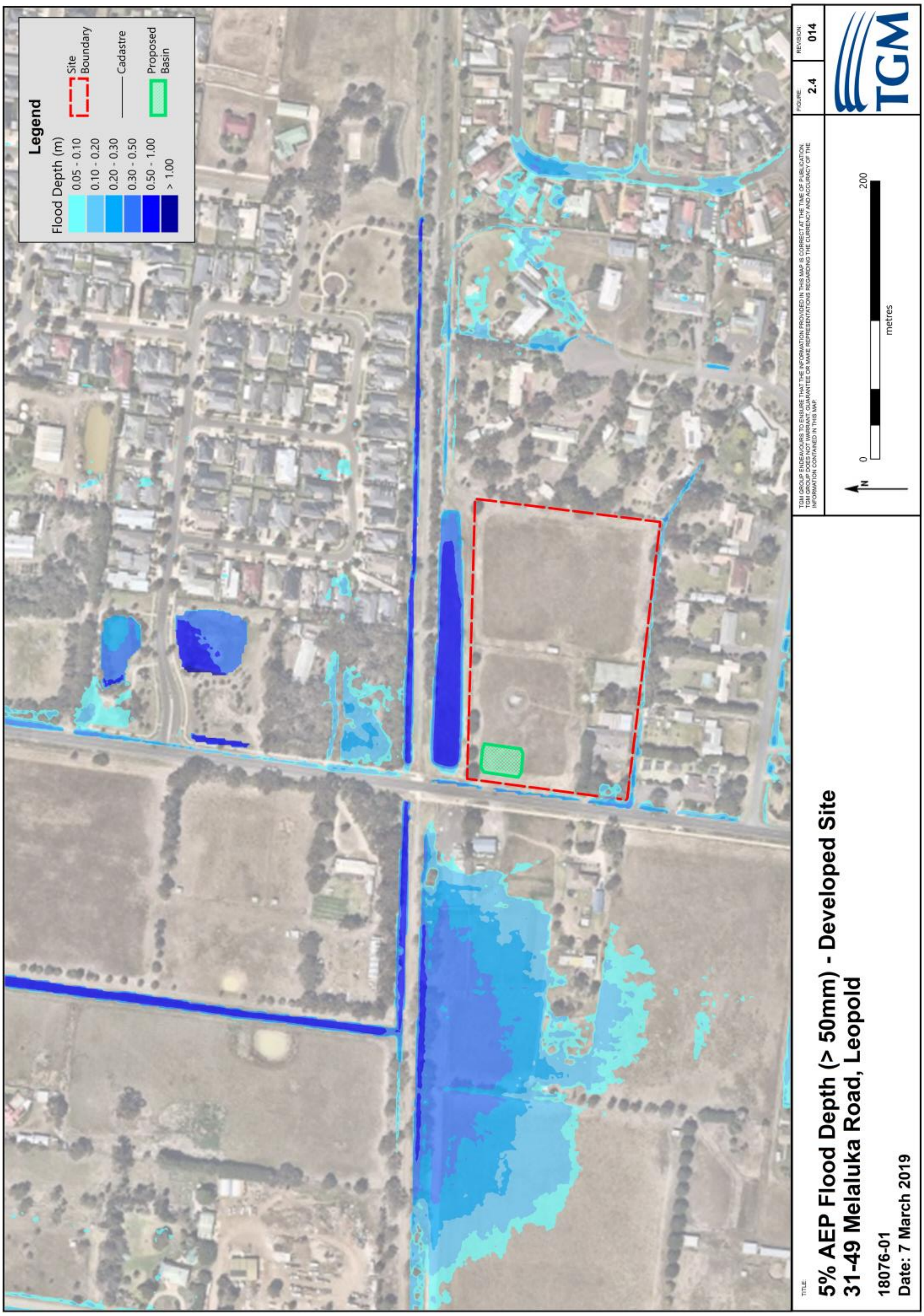
5% AEP Flood Depth (> 50mm) - Existing Site
31-49 Melaluka Road, Leopold

18076-01
Date: 7 February 2019

FIGURE: 5-1

REVISION: 013

TGM



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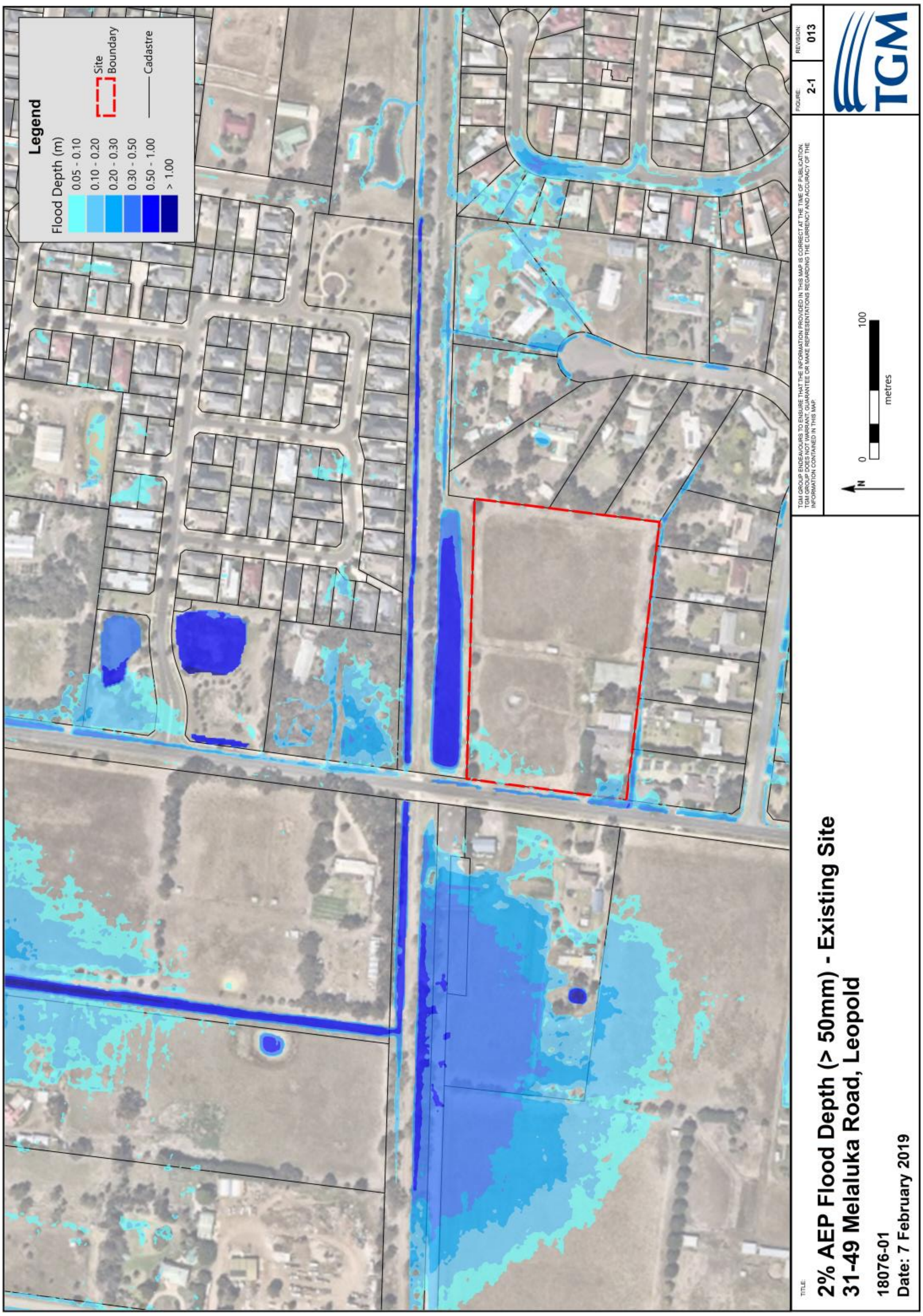
TITLE:

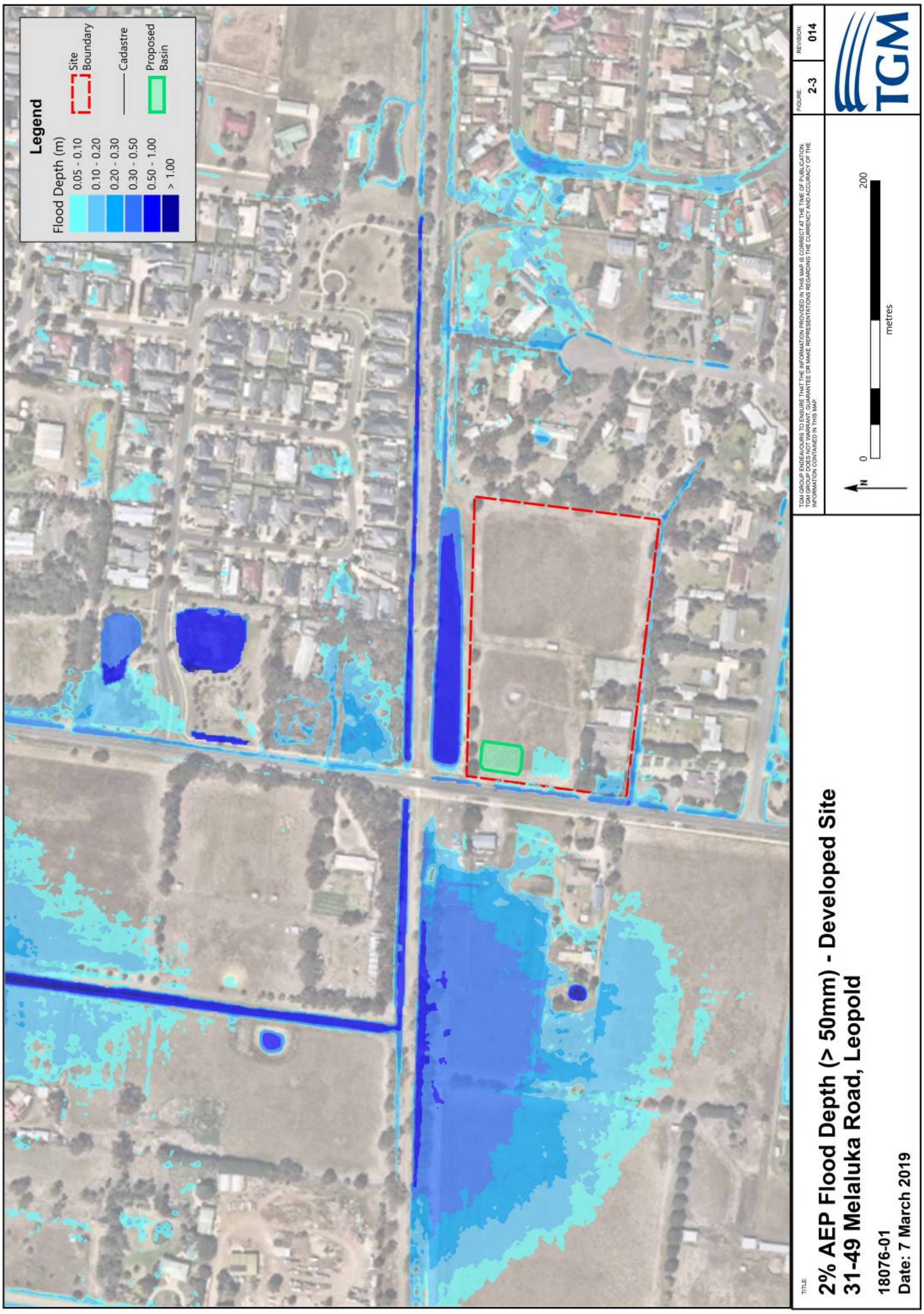
5% AEP Flood Depth (> 50mm) - Developed Site
31-49 Melaluka Road, Leopold

18076-01
Date: 7 March 2019



2% AEP Flood Depth (> 50mm) - Existing Site
31-49 Melaluka Road, Leopold

18076-01
Date: 7 February 2019



2% AEP Flood Depth (> 50mm) - Developed Site
31-49 Melaluka Road, Leopold

18076-01
Date: 7 March 2019