Member of the Surbana Jurong Group

Northern Geelong Growth Area Flood Impact Assessment and Stormwater Management Strategy

Volume 1: Existing Conditions Report

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As outlined in the 'Northern and Western Geelong Growth Area, Technical Reports - Scope of Works' (May 2016) there are broadly three phases of the NWGGA project. These are;

- Phase 1 – Existing conditions modelling, followed by developed conditions modelling, and initial strategy development
- Phase 2 – Workshops and additional modelling to refine the strategy
- Phase 3 – Refinement of the strategy to a finalised document incorporating staging recommendations, costing, flood overlays and final reporting

The purpose of this report is to present the first component of Phase 1 - existing flooding conditions across the investigation area. Explicitly the peak flows (1 in 100 Annual Exceedance Probability (AEP)) and consequent inundation extents within the investigation area.

The investigation area comprises six separate catchments covering the whole NGGA study area and additional downstream regions. The six catchments are Cowies Creek (c153), Sunderlands Creek (c180), St Georges Drain (c155), Wharf Road Drain (c156), Elcho Drain (c157J) and Hovells Creek (c157AX). Due to the minimal area covering Sunderlands Creek catchment, the assessment of this catchment has been omitted from the whole investigation. Refer to Appendix A for the catchment boundaries.

Information required for this assessment has been obtained predominantly from CoGG, including previous catchment study reports, as constructed drawings, development proposal reports and digital data.

The methodology adopted in developing the existing flooding conditions modelling has been developed based on a combination of CoGG Guidance Notes together with an analysis approach agreed with CoGG and consistent with current industry standards.

The hydrologic assessments have been undertaken using various methods including the rational method (VicRoads version), the RORB rainfall runoff model and the Tuflow two dimensional hydraulic model. The models have been calibrated to ensure the resulting peak flows at key locations are matching or within reasonable range of the rational method estimates.

Using the adopted methods, estimated peak flows for the 1 in 100 AEP flood event were produced for the undeveloped (zero development) and existing conditions scenario.

The results of the existing conditions scenario for all five sub-catchments are included in Table 1-1.

Table 1-1 - 1 in 100 AEP Flood Existing Conditions Flow Summary

Location	Catchment Area (km^2)	RORB		Tuflow (Adopted)	
		Critical Duration (hr)	Peak Discharge (m^3/s)	Critical Duration (hr)	Peak Discharge (m^3/s)
Cowies Creek	20.7	12	42.6	12	35.3
St Georges Drain	7.5	2	44.7	2	24.5
Wharf Road	4.7	9	16.3	9	17.4
Elcho Drain	19.8	12	42.1	12	29.2
Hovells Creek	9.3	2	35.6	12	21.3

The Tuflow two dimensional hydraulic model has been used to determine the existing conditions inundation extents for the 1 in 100 AEP flood event.

The impacts of varying the hydraulic model parameters were tested extensively as part of the Tuflow modelling including the model grid size and roughness parameters. Generally, the analysis indicates that the model is most sensitive to the Manning's n roughness parameter

Previous hydrologic/hydraulic studies of the various catchments were available for most catchments. A comparison of the flows from this study and the most relevant flows from the other studies are presented. Differences have been noted and reasons for any differences have been discussed.

2. ABBREVIATIONS AND ACRONYMS

Table 2-1 - Abbreviations and Acronyms

Abbreviation/ Acronym	Description
A	Area
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
BoM	Bureau of Meteorology
CCTV	Closed Circuit Television
CL	Continuing Loss
COGG	City of Greater Geelong
DEM	Digital Elevation Map
DRB	Drainage Basin
DTM	Digital Terrain Model
Dwg	Drawing
DVA	Dandenong Valley Authority
H	Horizontal
IEAUST	Institute of Engineers Australia
IFD	Intensity Frequency Duration
IL	Initial Loss
km	Kilometres
m	Metres
mm	Millimetres
MW	Melbourne Water
Q	Flow
RCP	Reinforced Concrete Pipe
RB	Retarding Basin
SMEC	Snowy Mountains Engineering Corporation
TOR	Terms of Reference
V	Vertical
12D	Civil 3 dimensional modelling software
2D	Two dimensional

3. INTRODUCTION

3.1. General

SMEC Australia has been engaged by LB Rezoning Pty Ltd to establish a site based Flood Impact Assessment (FIA) and Stormwater Management Strategy (SMS) for the Northern Geelong Growth Areas project (NGGA). The study has been undertaken to accord with requirements set out by the City of Greater Geelong (COGG), who are committed to preparing an Integrated Infrastructure Delivery Plan (IIDP) and a Framework Plan for the region. The FIA and SMS form one of the technical input studies required to inform the IIDP.

The Study Area is approximately bound by the following landmarks:

- Staceys Road in the north;
- Bacchus Marsh Road in the east;
- Geelong Ring Road in the south; and
- Anakie Road in the south west.

An outline of the study area is presented as Figure 3-1.

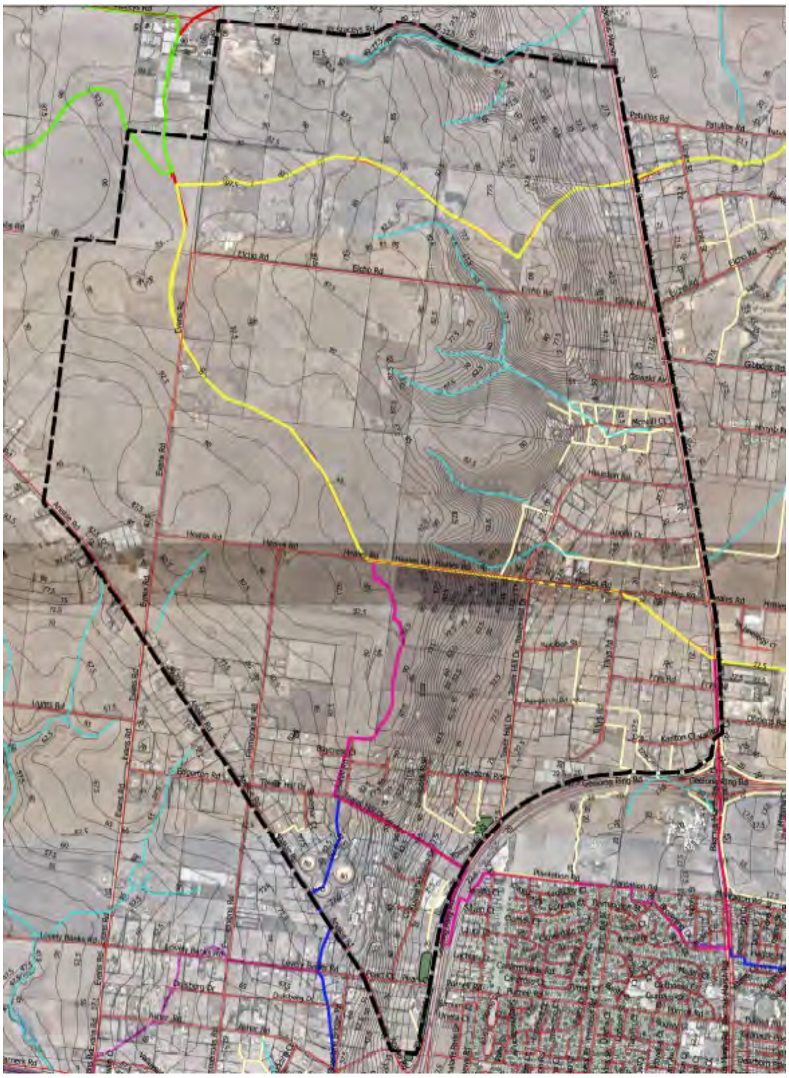

Figure 3-1: Study Area Boundary

The purpose of this report is to present outputs detailing the existing flooding conditions across the investigation area. The investigation area covers a greater extent than the study area and it has been defined as part of the project scope of work. The investigation area is detailed as a figure included as Appendix A.

CoGG have established that the purpose of the FIA and SMS is to achieve the following outcomes:

- Manage the storage and movement of storm and flood water runoff through the site and downstream areas.
- Create an attractive and active community urban environment.
- Develop runoff management infrastructure which appropriately balances amenity, efficiency and safety.
- Ensure that the strategy does not result in any adverse impacts within the study and downstream areas.
- Integration of the runoff management infrastructure with other land uses and access requirements.
- Practicality in construction and maintenance for the infrastructure elements.
- Advice of orderly sequencing of development.

The required scope of works for the study as required by CoGG and Corangamite Catchment Management Authority (CCMA) is described in the briefing documents as follows:

- CoGG (2016) Northern and Western Geelong Growth Areas Project, Technical Reports – Scopes of Work, May 2016.
- CoGG (2016) Northern and Western Geelong Growth Areas Project, Flood Impact Assessment and Stormwater Management Strategy – File Note, 2nd June 2016.

Broadly, the study encompasses several phases:

- Phase 1 – Existing conditions modelling, followed by developed conditions modelling, and initial strategy development
- Phase 2 – Workshops and additional modelling to refine the strategy
- Phase 3 – Refinement of the strategy to a finalised document incorporating staging recommendations, costing, flood overlays and final reporting

This report is concerned, with the Phase 1 hydrologic and flooding analysis for the catchment existing conditions.

4. AVAILABLE INFORMATION

4.1. General

The FIA and SMS requires a range of inputs which have been obtained from a variety of sources. The data has predominately been provided by CoGG. The variety of available data and the relevant source is described in detail below.

4.2. Digital Data

CoGG has provided digital data describing topography and available infrastructure as follows:

- NGGA study area and regional catchment boundaries
- Lidar for NGGA and catchment areas
  - Area North of Staceys Road and west of Evans Road, Year of capture 2014, vertical accuracy +/- 0.10 m
  - Balance of Study Area, Year of capture 2011, vertical accuracy +/- 0.10 m
- 0.5 m contours for NGGA and catchment areas
- Cadastre
- GIS layer of existing drainage pipe location and diameters in developed areas
- GIS layer detailing existing Retarding Basins, wetlands, sedimentation basins, and earthen channels

CoGG have indicated that the GIS infrastructure layer is constructed using the best available data, but it has not been rigorously vetted and its accuracy cannot be guaranteed. Furthermore, it is understood that the infrastructure layer may not capture all in ground assets.

4.3. Drawings

A variety of design drawings for various developments have been provided as follows:

- SKM/Abigroup (2009) Geelong Ring Road, as constructed drawings
- SMEC Urban (2013) Heales Road Lara Elcho Channel, preliminary design drawings
- SMEC Urban (2013) Geelong Ring Road Employment District, Elcho Basin Stage 1, preliminary design drawings
- SMEC Urban (2011) Geelong Ring Road Employment Precinct, Drainage, preliminary design drawings

To assist in the development of the terrain model, SMEC retrieved an electronic copy of the Elcho Drain surface from our file archives.

4.4. Reports

A variety of studies have been undertaken over the course of the past 20 years, analysing rainfall runoff processes of the NGGA and downstream areas. The relevant available reports are as follows:

- BMTWBM (2011) Kosciusko Avenue Main Drain Catchment Drainage/Flood Study, Report to CoGG, April 2011.
- Craigie, N.M. (2012) Lara West Growth Area (LWGA) Surface Water Management Strategy, Report to SMEC Urban, June 2012.

- GHD (2004) St Georges Road Main Drain Catchment Augmentation (Stage 1), Report to CoGG, April 2004.
- SKM (1999) Heales Road Industrial Estate – Drainage/Flood Study (Stage 2), Report to CoGG, November 1999.
- SMEC (2010) Heales Road Industrial Area Drainage Study, Report to CoGG, November 2010.
- Spiire (2014) Manzeene Village, Lara, Site Stormwater Management Plan Report, Report to CoGG, April 2014.
- Water Technology (2004) Geelong Bypass – Hydrology and Hydraulics – Stage 1 Assessment, Section 1 – Princes Freeway, Corio Interchange to Midland Highway, Report to VicRoads, January 2004.
- WBM (2006) Grand Park Hydraulic Modelling and Impact Report, Report to Bisinella Developments/TGM Group, November 2006.
- WBM Oceanics (2002) Lara Flood Study Stage 1, Report to CoGG, April 2002.

In addition to the above listed reports, important input data for the assessment are included in CoGG generated documents as follows:

- CoGG (2016) Northern and Western Geelong Growth Areas Project, Context Report, April 2016.
- CoGG (2016) Northern and Western Geelong Growth Areas Project, Technical Reports – Scopes of Work, May 2016.

4.5. Survey

A number of locations were identified where existing drainage infrastructure has the potential to influence the occurrence of flooding. Given the qualification which CoGG have expressed with regard to the accuracy of drainage infrastructure characteristics in their data base, some additional survey was commissioned. The infrastructure elements which have been surveyed as part of this project are as follows:

- 4 No. culverts under Bacchus Marsh Road between Heales Road and Windermere Road
- 6 No. culverts under Anakie Road between Lovely Banks Road and Geelong-Ballan Road
- 4 No. culverts under Geelong Ballan Road between Anakie Road and Staceys Road
- Railway bridge crossing over Cowies Creek.
- Elcho Drain culverts under Buckingham Street and Canterbury Road West in the Grand Lakes development.

A sketch showing surveyed culvert locations is included as Appendix B.

4.6. Aerial Imagery

No aerial imagery has been provided with the data supplied by CoGG. For the purposes of map production, SMEC has used images supplied by Nearmap.

4.7. Flow Data

It is understood that there are no flow gauges within the CoGG region and there are no flood records that could assist in undertaking model validation or calibration for the investigation area. The only available regional flow data has been captured for relatively large catchments and are not generally relevant to this study.

5. METHODOLOGY

5.1. General

This section outlines the methods applied in developing the existing conditions layout. The methods have been defined based on a combination of specific Guidance Notes developed by CoGG together with an analysis approach agreed with CoGG consistent with current industry standards.

The following standards and guidance notes were utilised for the development of the existing condition modelling methodology:

- CoGG (2012) Design Notes No.2 – Stormwater Detention Storage Design, August 2012.
- Melbourne Water (2015) Constructed Wetlands Design manual, Part D: Design Tools, Resources and Glossary (Final Draft).
- IEAust (1997) Australian Rainfall and Runoff, A Guide to Flood Estimation.

It may be noted that IEAust (1997) is currently undergoing revision and a select number of books are available in draft form for review. At the time of report writing, there are no draft chapters available for review directly related to urban hydrology (Book 9). There are draft books and chapters available which cover rainfall runoff (Books 3, 4 and 5) and hydraulic modelling (Book 6). It is recognised that the draft books have yet to be adopted for use and are still subject to industry review. For the purposes of developing the methodology for this study, IEAust (1987) has been applied. It is considered that the aspect of the draft books with greatest potential to impact upon the outcomes of this study is the proposal to apply ensembles of temporal patterns and losses.

Notwithstanding this potential impact, it is recognised that there is little direct evidence of peak stormwater or flood flows within the CoGG region. It is considered that this lack of data represents the greatest limitation to improving the accuracy of rainfall runoff process simulations. It is judged that any alteration in the rainfall runoff modelling approach nominated in the new version of Australian rainfall and Runoff is unlikely to improve the confidence in any rainfall runoff modelling outcomes until such a time as it can be corroborated by measured flow data.

5.2. Hydrology

5.2.1. Approach

This section presents the general hydrologic analysis methods applied to each of the sub areas within the investigation area. The hydrology has been considered separately for each of the investigation sub areas as listed below (Sub areas are detailed in a figure attached as Appendix A):

- Cowies Creek Sub-Area (C153).
- St Georges Drain (C155).
- Wharf Rd Drain (C156).
- Elcho Drain (C157J).
- Hovells Creek (C157AX).

The methodology has been developed with the aim of ensuring consistency across the investigation area to the extent possible. Different aspects of the catchment hydrology have been modelled using the rational method (VicRoads Version), the RORB rainfall runoff model and the Tuflow two dimensional hydraulic model.

The rational method has been applied to provide an indication of the 1 in 100 Annual Exceedance Probability (AEP) flow that should be expected at sub-catchment outlets and at various locations within each sub area.

The RORB model has been calibrated to the rational method to ensure that the flow magnitude can be achieved using model parameters which are considered to be within an acceptable range. The model has also been used to provide an indication of the flow variability within the various sub areas.

The Tuflow model has been calibrated to provide flows which are consistent with those from the rational method/RORB model. The model has been used to ensure that any hydraulic controls on flow generation and attenuation are captured within the modelling process and appropriately accounted for.

In an attempt to achieve consistency across all sub catchments, modelling was initially conducted for sub catchments with zero development. The Cowies Creek catchment was considered in the first instance and relevant RORB and Tuflow modelling parameters were derived. To the extent practicable, these same characteristics were preserved for modelling of the remaining sub areas.

Once the zero development conditions modelling was completed, impervious areas were introduced to the model which are consistent with the existing conditions and the RORB and Tuflow models were re-run to provide existing conditions outputs.

In cases where there are differences between RORB and Tuflow modelling outputs, preference has generally been given to applying the Tuflow model outcomes. This is because the Tuflow model explicitly simulates waterway and floodplain hydraulic conditions and should provide a superior definition of relative flows within a sub catchment in comparison to the approach applied in the RORB software package.

5.2.2. Regional Flood Estimates

Application of a regional flood estimation method is necessary as part of the modelling process, as there is no flow data available to assist in model calibration. A variety of methods are available to assess regional flows including the following:

- Urban Rational Method
- Rural Rational method
- Rural Rational Method (VicRoads form)
- Regional Flood Frequency Estimate (RFFE) process developed as part of the new from Ball et.al. (2016)
- Regression relationships such as those described in Grayson et.al. (1996)
- Application of default parameters to a RORB catchment model.

CoGG (2012) expresses a preference for applying the VicRoads version of the rational method when estimating 1 in 100 AEP flows. In cases where the above equations have been reconciled against observed flow data, (eg IEAust 2009) a substantial degree of uncertainty has been observed.

It is observed that the VicRoads rational method is a commonly applied regional formula and in addition there is little or no objective documentation to predict peak flows from small catchments in the region. On this basis, it is considered reasonable to utilise the VicRoads rational method as the initial basis for establishing design flows in the investigation area.

It is also considered to be prudent to compare the results from the VicRoads rational method against the RFFE estimation procedure as this is likely to become the default recommendation in the final version of the updated Australian Rainfall and Runoff document. It should be recognised, however, that the RFFE method is most likely to produce relevant outcomes in circumstances where the observed regional flow data is from catchments with similar characteristics to the catchment being studied. It is observed with respect to this study that the closest observed data is remote from the NGGA and catchment areas are typically much larger.

5.2.3. RORB Model

5.2.3.1. General

A RORB runoff routing model has been used to simulate the hydrologic performance of the various sub areas and to evaluate the internal flow distribution of each sub area.

RORB (Laurensen et al 2010) runoff routing characteristics along with the catchment stream network are represented by the parameters k_c and m. The effect of the catchment in delaying the runoff response from rainfall is represented by k_c and the non-linearity in the storage discharge relationship for the catchment is represented by m. The delay throughout the catchment is assumed by the model to be related to stream length or some other suitable measure identified by the modeller. The model also incorporates a loss model to account for rainfall lost to groundwater stores, evaporation and various other sinks.

The approach typically preferred by CoGG in selecting the RORB rainfall runoff parameters and loss values is described below. While rainfall and runoff parameters and losses are considered separately below, the peak recorded runoff is dependent on both. Therefore, when a parameter is altered, due consideration must be given to the impact on other parameters or losses.

5.2.3.2. Calibration Approach

The preferred method for setting RORB parameters is to calibrate to observed rainfall and runoff data. There is no such data available for this study and instead the model has been calibrated to rational method estimates.

5.2.3.3. Model Schematic

The RORB model schematic takes the form of a sub area layout diagram overlying a stream network diagram. In addition, the stream network is defined with respect to its efficiency as being natural, excavated, lined or drowned. The model sub area hard surface development is also characterised within the model as a proportion impervious. The imperviousness affects the manner in which losses are modelled over each sub area.

A consistent sub area size of around 10 ha has been targeted for the investigation area. This sub area size provides consistency across the various models and also ensures that there are a number of sub areas upstream of critical output points along the study area boundary.

For the purposes of preparing the undeveloped conditions model, a natural stream type has been selected (Code type 1). In preparing the existing conditions model, a natural stream type has been used for areas which are rural or sparsely developed. A more developed stream type has been selected in more densely developed areas (Code type 2).

The undeveloped model has had zero imperviousness assumed over the entire catchment. The existing conditions model has had imperviousness included representing the full degree of development allowed by the current planning overlays. The proportion imperviousness applied for differing development types has been assigned in accordance with the recommendations nominated in CoGG (2012a).

5.2.3.4. Rainfall Runoff Parameters

Laurensen et al (2010) recommend the approach for selecting RORB runoff routing model parameters k_c and m is to calibrate the catchment file utilising rainfall and runoff data for selected historical storm events. There is no such data available for this study and instead, a variety of regional equations have been applied to identify the range of values which would be considered typical. It should be recognised that the regional equations are relatively coarse and do not take account of a range of site specific features that would influence the catchment delay such as catchment shape, catchment slope and soil depth. The following equations are typically applied in Victoria and Tasmania:

- $k_c = 1.53 \times A^{0.55}$ (A = area in km²) Recommended by Melbourne Water for Dandenong Creek and Westernport Catchments
- $k_c = 1.19 \times A^{0.56}$ (A = area in km²) Recommended by Melbourne Water for Yarra and Maribyrnong Catchments
- $k_c = 2.57 \times A^{0.45}$ (Annual Rainfall > 800 mm, Hansen et. al. 1986)
- $k_c = 0.49 \times A^{0.65}$ (Annual Rainfall < 800 mm, Hansen et. al. 1986)

Although the m parameter can vary in the range of 0.6 to 1 it is recommended in IEAUST (1997) that a value of 0.8 be used in the absence of evidence supporting an alternative value and this value has been adopted for use in this study.

5.2.3.5. Loss Modelling

RORB offers two loss models for use with the software package;

- An initial and continuing loss model where the initial loss is measured in millimetres and represents the loss that occurs before runoff commences. The continuing loss is measured in millimetres per hour and occurs throughout the course of the rainfall event.
- An initial and a constant proportional loss model where the initial loss is measured in millimetres and represents the loss that occurs before runoff commences. The proportional loss is represented by a runoff coefficient and represents the portion of the catchment that contributes rainfall to the runoff hydrograph. Therefore, over each measured time increment of rainfall the rainfall is reduced by multiplying the rainfall by the runoff coefficient.

The initial and proportional loss models in an urban context have been recently investigated and reported on in IEAust (2014). The report concludes that from the data reviewed, overall, the proportional loss model preforms better than the continuing loss model, but the results are neither consistent enough nor strong enough to draw a clear conclusion. Overall, either model is considered appropriate for application. The continuing loss model is preferred in circumstances where flood frequency curves for extreme floods are required, predominately due to the presence of an agreed convention for extrapolating losses into the extreme range.

CoGG (2012) expresses a preference for using the initial and continuing loss model and that is the model which has been adopted for use in this study.

The recommended approach to selecting loss values is as follows:

- Where adequate rainfall and runoff data for the catchment under consideration is available, the loss values that appear to best represent the observed rainfall and runoff characteristics for the catchment are utilised.
- In circumstances where there is little or no rainfall and runoff data for a given catchment, but there is data available for an adjacent catchment with similar form and degree of urbanisation, then the loss values of the adjacent catchment are applied to the catchment under consideration.

- Where there is no rainfall and runoff data available either on the catchment under consideration or another similar catchment nearby, then regional loss values are adopted.

Regional loss values have been suggested by CoGG for the Cowies Creek as follows:

- Initial Loss (IL): 15 mm
- Continuing Loss (CL): 2 mm/hr

Loss definitions as suggested by CoGG are understood to be informed by the regional soil maps a copy of which is included as Appendix C.

Alternative losses have been suggested for the Moorabool/Sunderlands Creek catchment (20 mm IL and 3 mm/hr CL). Notwithstanding this recommendation, the Cowies Creek losses have been applied to sub catchment C180 since the soil map indicates that the portion of that catchment modelled within the investigation area is similar to Cowies Creek.

The soil map indicates that all of the study area and the majority of the investigation area is of a similar soil type and as such, consistent losses have been assumed as the starting position for the assessment.

Note that the actual loss values selected will depend upon how well the peak modelled output matches with regional peak flow estimates.

Impervious areas are internally set by the RORB model to have zero loss.

5.2.4. Tuflow Model

A Tuflow two dimensional hydraulic model has been used to assist in defining the catchment hydrology. Separate Tuflow models were developed which describe the entirety of each of the sub catchments within the investigation area. The model setup is described in further detail in Section 5.3.

To assist in undertaking the hydrology analysis, two variants of the Tuflow model have been produced. The first represents undeveloped conditions and the second represents existing conditions. The two models are identical with the exception of the assumed losses. The inflow hydrographs inputs to the undeveloped model have IL and CL losses removed from the entire catchment. The existing conditions inflow hydrographs have a lesser loss since there is no loss from impervious areas.

The Cowies Creek Tuflow model was established first and the grid size and cell roughness parameters were altered and extensively tested to provide a reasonable match to the rational method/RORB modelled flows at the catchment outlet. Similar grid sizes and roughness parameters were then used for the modelling of the undeveloped model of the remaining catchments.

The Tuflow model outputs are considered to be of particular relevance in modelling catchments drainage to the south and east. The topography off the escarpment down to Bacchus Marsh Road is very steep relative to the region to the east of that road and substantial relative attenuation of flows is to be expected. This attenuation would not be captured in either rational method or RORB modelling processes which utilise catchment area and stream lengths respectively to simulate relative delay/attenuation.

It is considered that Bacchus Marsh Road should be the focus for achieving consistency between the various modelled flows and to the east of Bacchus Marsh Road, the Tuflow model outcomes should be relied upon for estimating runoff.

5.2.5. Hydrology Analysis

5.2.5.1. Flood Frequency Analysis

Given that there is no stream flow data available, runoff estimates for 1 in 100 AEP peak flows have been estimated using the VicRoads rational method and the RFFE approach as described above.

CoGG (2012) provides guidance on how the rational method should be applied and that method has been adopted for use in this study.

RFFE has been computed utilising the relevant website (accessed 21/10/16) as follows:

http://rffe.arr-software.org/

The Model requires inputs of latitude and longitude for both the catchment centroid and outlet along with the catchment area.

5.2.5.2. Design Rainfall Estimation

Various aspects of the design rainfall were applied in the analysis as follows:

- 1 in 50 and 1 in 100 AEP design rainfalls and others more frequent were estimated using the on line Bureau of Meteorology (BoM) website tool located at http://www.bom.gov.au/water/designRainfalls/ifd/index.shtml. It may be noted that currently there are two IFD relationships available on this website, being 1987 and 2013 data sets. The 1987 data set has been applied for use in this analysis since the VicRoads rational method was derived using that data set.
- The selection of the design temporal patterns presented in IEAust (1987) were adopted for use in this analysis.
- A uniform spatial pattern was adopted for use in the analysis consistent with the recommendations of IEAust (1997).

5.3. Hydraulic Modelling

5.3.1. General

A hydraulic model is required to establish the inundation extent associated with existing flood conditions. A Tuflow two dimensional hydraulic model has been developed for this study, the important features of which are discussed below.

5.3.2. Model Setup

Tuflow models require either a Digital Elevation Model (DEM) or a Digital Terrain Model (DTM) as input. A DEM is an unmodified elevation surface at a defined grid spacing derived from some input data set, while a DTM is typically considered to be a refined DEM with additional finer scale elements such as break lines, infrastructure or drains incorporated into the surface. For the purposes of this analysis, it is considered that a DTM should be developed since there are a range of features important to the modelled outcomes that are not picked up in a DEM based surface.

A DTM has been developed for each of the individual sub catchments utilising the Lidar supplied by CoGG as a base and modified using a combination of survey and infrastructure information from the CoGG database supplied for the study. In particular, survey has been obtained for culverts located at the study area boundary and also a select number of additional locations within the larger investigation area such as the Cowies Creek outlet and the Grand Lakes development culverts.

In some cases, the grid cells from the DTM are higher than the culvert invert levels. In such circumstances, the grid cell elevation adjacent to the culvert has been modified to ensure that it is at or below the culvert invert.

A grid spacing of 10 m was adopted for the model for the purposes of identifying the critical duration storm event. The critical duration events were subsequently modelled using a 5 m grid spacing. This process was implemented to minimise the run times for the various models. One dimensional elements were included to represent culverts crossings and bridges.

Time steps set to ensure that the courant number is less than 1. As a maximum, the time step was set at a value in seconds which is half the grid size in metres.

5.3.3. Boundary Conditions

5.3.3.1. General

The boundary conditions used in the model encompass inflows and downstream water surface controls.

5.3.3.2. Inflows

Inflows to the model have been introduced as hydrographs from the RORB model, located at the centroid of each of the RORB catchment sub areas. Note that the inflow hydrographs have not been attenuated within RORB and therefore, the Tuflow hydraulic routing functions are being used to simulate all delay processes occurring within the catchment. As discussed above, losses have been removed from the input hydrographs. The loss removal process has occurred within RORB.

All of the undeveloped and existing conditions models (except Sunderlands Creek) were run for four durations, being the 2 hour, 9 hour, 12 hour and 48 hour events. It is judged that those four durations would provide outcomes which are close to the critical conditions in each of the modelled sub areas.

5.3.3.3. Downstream Water levels

The downstream water level boundary conditions have been evaluated separately for each sub catchment taking into consideration all available information inclusive of existing flood mapping reports, design drawings, survey, Lidar and CoGG infrastructure database.

In general, the approach has been to utilise either a modelled depth flow relationship or hydraulic control where appropriate, otherwise the downstream surface slope has been used to provide the control assuming normal depth.

5.3.4. Roughness Parameters

Roughness elements have been defined throughout the model with land use types being allocated a particular roughness as detailed in Table 5-1.

Table 5-1 - Tuflow Modelling Roughness Parameters

Surface Element	Adopted Roughness (Manning's n)
Undeveloped Conditions Model	
Floodplains/natural overland flow paths	0.2 for water depth ≤ 30 mm 0.045 for water depth ≥ 100 mm For water depth in between, a linear interpolation is applied
Existing Conditions Model	
Floodplains/natural overland flow paths	0.2 for water depth ≤ 30 mm 0.045 for water depth ≥ 100 mm For water depth in between, a linear interpolation is applied
Urban Areas	0.2
Low density residential	0.075
Culvert/pipe lining	0.012
Roads	0.016

Throughout the course of the project, a number of approaches to defining the model roughness have been trialled. It has been found that an acceptable consistency of approach across the various modelled sub areas is obtained when applying a Manning's n value which varies with depth.

6. COWIES CREEK SUBCATCHMENT C153

6.1. Site Description

The Cowies Creek catchment is located in the south western portion of the study area. The outlet of the Cowies Creek catchment is located at the railway bridge crossing. The total area of the catchment included in the investigation is 20.7 km^2, of which the study area comprises around 28%. There are seven separate waterways or drainage lines which discharge flows across the study area boundary, predominately across Anakie Road. A map of the sub catchment is included as Figure 6-1.

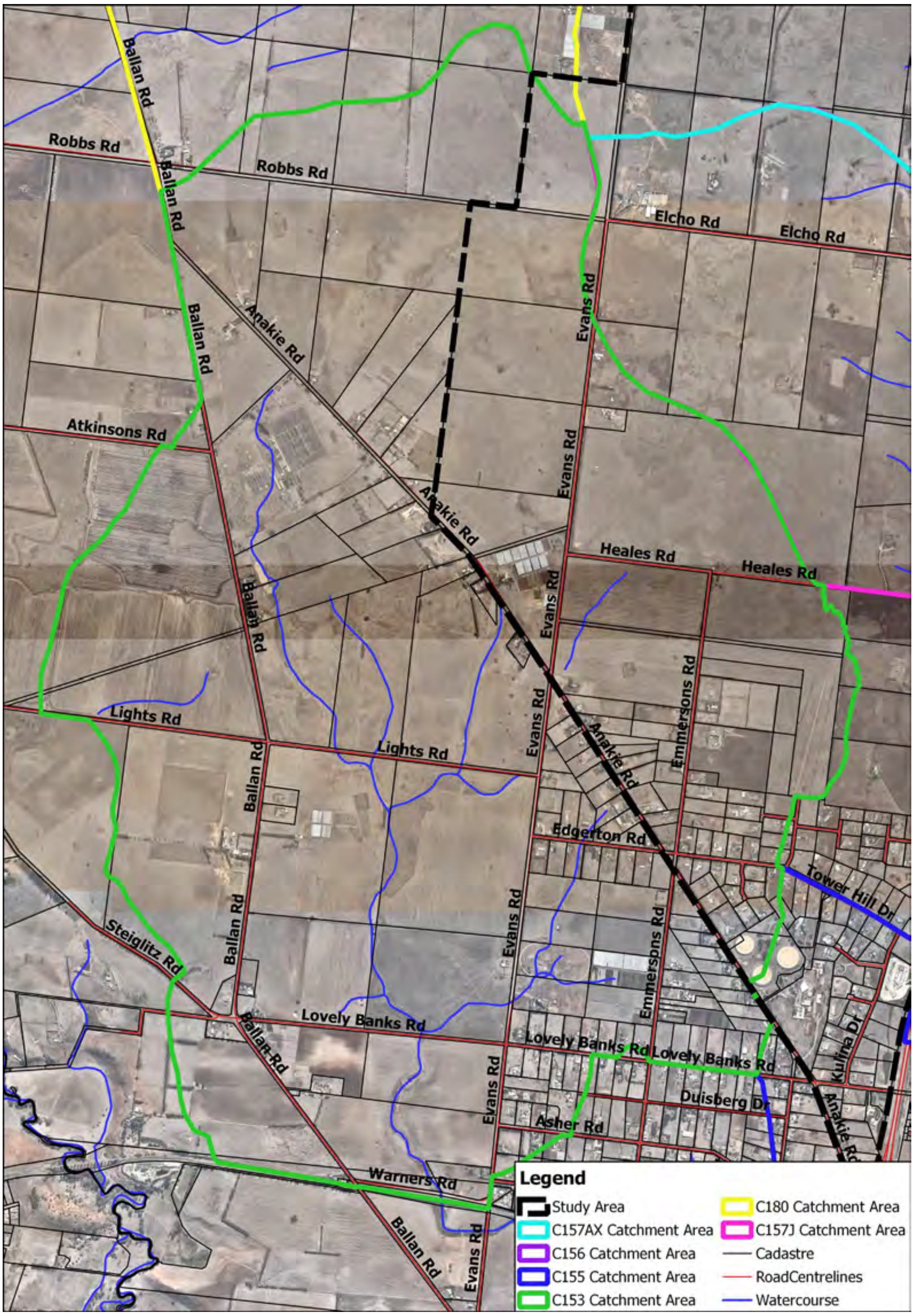

Figure 6-1: Cowies Creek Sub Area

It may be noted that the above figure represents a departure from the area presented in CoGG supplied documentation. During the course of modelling, it has been determined that the actual catchment area extends further to the north and encompasses part of the catchment which has previously be identified as draining to Sunderlands Creek.

6.2. Adjacent Sunderlands Creek

Sunderlands Creek is a sub catchment covering a very small area of the study area which is located to the north of Cowies Creek catchment boundary. An assessment of this sub catchment indicates that there is only a very small portion of that catchment located within the study area boundary (~4 ha). Of that area, the majority is controlled by a combination of an exclusion overlay and an electricity easement, which effectively excludes that area from potential development. As a result, no separate investigation of this catchment has been undertaken as part of this investigation. Should conditions change and this area become a subject of further development, a separate catchment investigation should be undertaken in the future.

6.3. Hydrology

6.3.1. RORB Catchment Model

6.3.1.1. General

A RORB model has been developed for the Cowies Creek catchment. As described above, the model was calibrated by first generating the model without any impervious areas included in the catchment file. The model was calibrated to a rational method flow estimate and the impervious areas were subsequently placed into the model for the design runs.

For this catchment, a natural reach type was selected for both undeveloped and existing conditions models.

6.3.1.2. Sub-Catchment Area

The Cowies Creek sub catchment model has an area of 20.7 km^2, which was delineated into 260 sub-catchment areas ranging in size between 4.5 and 11 ha. A schematic of the RORB model is included in Appendix D1.

The fraction impervious value for the undeveloped condition model is zero. The fraction impervious values applied to the existing conditions model were developed using the existing zonings. The basis upon which the RORB model impervious fractions were selected is included as a figure in Appendix D2.

6.3.1.3. Delay Parameters

K_C Value

The range of k_C values which are plausible for the catchment have been estimated using a variety of methods as detailed in Table 6-1.

Table 6-1 - Estimate k_C parameter equations

ESTIMATE	EQUATION	Kc
Dandenong Creek and Westernport Catchments	$k_c = 1.53 * A^{0.55}$ ▪ A = Area (km^2)	8.1
Yarra and Maribyrnong Catchments	$k_c = 1.19 * A^{0.56}$ ▪ Where A = Area (km^2)	6.49
Vic(MAR>800mm)	$k_c = 2.57 * A^{0.45}$ ▪ A = Area (km^2)	10.0
Vic(MAR<800mm)	$k_c = 0.49 * A^{0.65}$ ▪ A = Area (km^2)	3.5

The results from the analysis indicate the range of kc values that would be expected for the catchment are between 3.5 and 10.0.

m Value

An m value of 0.8 has been adopted as discussed above.

Initial Loss and Runoff Coefficient

Losses of 15 mm IL and 2 mm/hr CL as discussed above have been adopted for use and only varied if necessary to achieve an appropriate match with the target calibration flow.

6.3.2. Flow Estimation Comparison

The RORB model has been calibrated to the VicRoads rational method flows with a 1 in 100 AEP. The rainfall runoff process conceptualisation is different between RORB and the rational method. The result of these differences is that the modelled outputs are not consistent across different probability events, durations and catchment areas.

The approach taken in calibrating the RORB model has been to ensure to the extent possible that the 1 in 100 AEP outflows do not vary by more than 30% form the VicRoads rational method estimates. A range of locations within the catchment have been considered when undertaking the calibration. The details of the modelled outcomes are presented in Appendix D3 and the outcome are displayed graphically for both the 1 in 100 AEP and the 1 in 5 AEP event as Figure 6-2 and Figure 6-3.

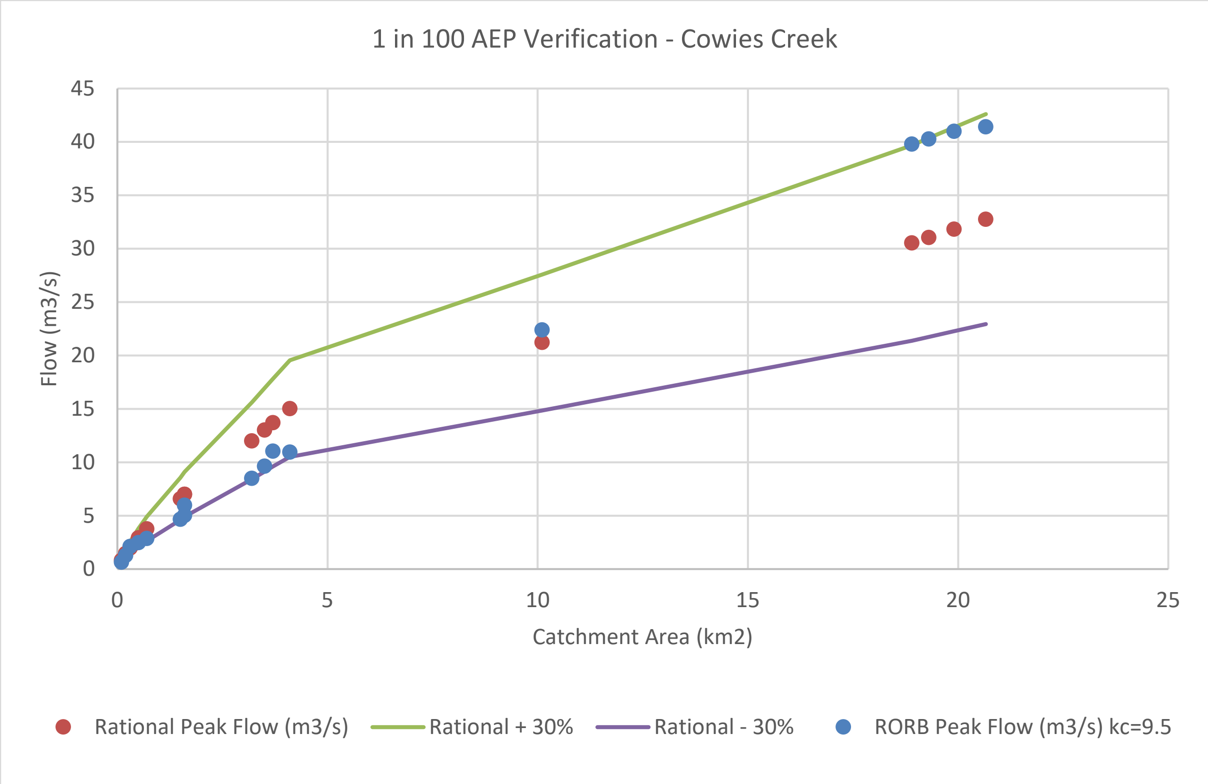

Figure 6-2: 1 in 100 AEP flow comparisons RORB vs VicRoads rational method

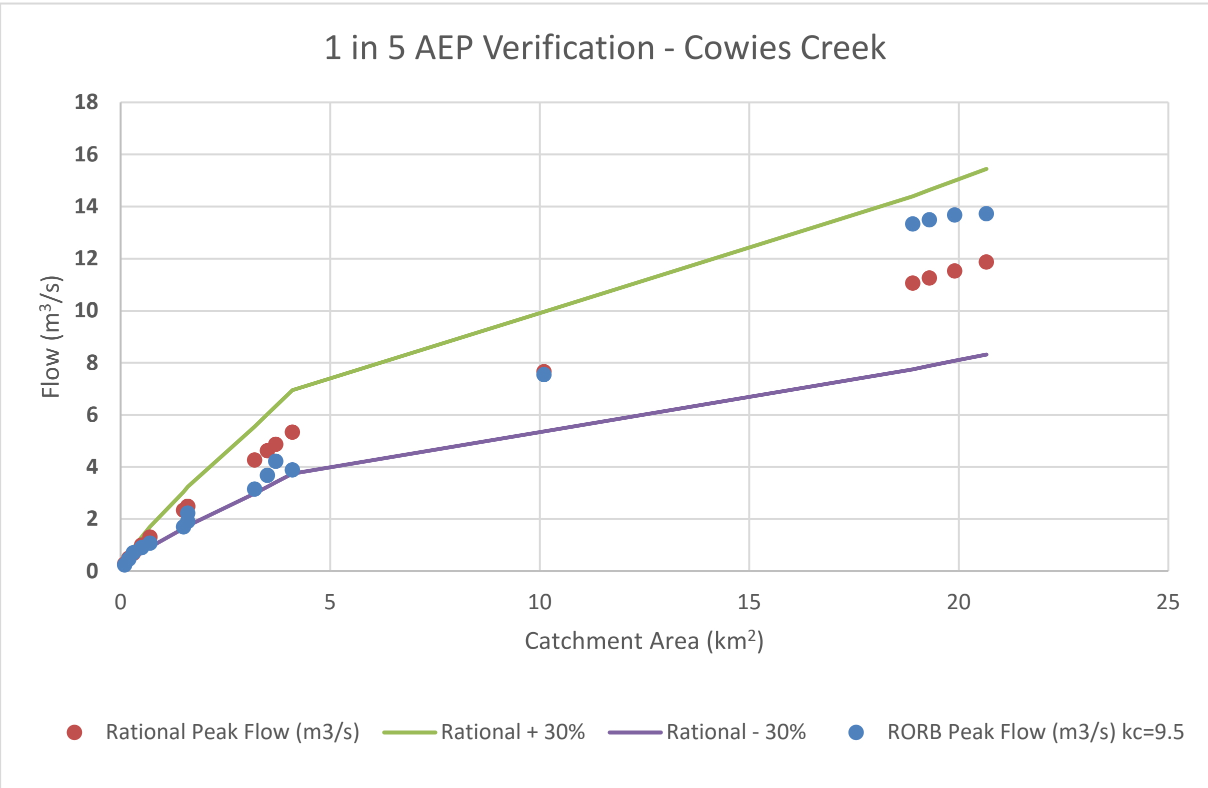

Figure 6-3: 1 in 5 AEP flow comparisons RORB vs VicRoads rational method

It may be observed from the above figure that the gradient of the runoff versus catchment area graph is different for the two modelling procedures. The RORB delay parameters is the predominate variable used for calibration and the parameter set detailed in Table 6-2 was found to offer the best compromise fit to the target flows.

Table 6-2 - Adopted RORB parameters

Parameter	Value
m	0.8
K_C	9.5
K_c/d_{av}	2.26
Initial Loss (mm)	15.0
Continuing Loss (mm/hr)	2.0

In addition to the VicRoads rational method and RORB modelled outcomes, both Tuflow and RFFE outcomes have been generated. The locations within the model considered to be of greatest relevance in the study are the overall catchment outlet and also the locations where waterways or flow paths cross the study area boundary. The flow at each of those locations has been reported upon comparatively in Table 6-3.

Table 6-3 – 1 in 100 AEP Flow estimation comparisons, undeveloped models

Location	Catchment Area (km^2)	RORB Critical Duration (hr)	Discharge (m^3/s)			
			VicRoads	RORB	RFFE	Tuflow
2 (Anakie Rd)	1.5	9	6.6	4.7		7.2 (9hr)
4 (Anakie Rd)	1.6	9	7.0	5.0		8.7 (9hr)
5 (Anakie Rd)	1.6	3	7.0	6.0		5.4 (2hr)
6 (Anakie Rd)	0.2	2	1.4	1.3		2.4 (2hr)
7 (Anakie Rd)	0.3	1	2.0	2.1		2.7 (2hr)
Outlet	20.7	12	32.8	41.4	29	33.9 (12hr)

The location numbers referred to in the above table are detailed on the RORB sub area catchment map included as Appendix D1.

From the above table, it is observed that the RFFE estimates represent a relatively minor departure from the VicRoads rational method estimate and the RORB model provide an estimate at the upper limit of the flow range. The Tuflow discharges are similar in comparison to the rational method. In the case of location 4 and 6, the difference in peak discharges can be explained by the difference in the contributing catchment area between the two methods. The RORB model has defined boundaries for waterway flow lines which are fixed for all flood events. The Tuflow model evaluates the catchment boundaries in a more flexible manner. In gently grading topography, cross boundary flows may commence that can be variable with flow. The Tuflow model simulates the interaction and allows for interchange of flows across boundaries that otherwise act as barriers in the RORB model. This phenomenon can lead to differences in modelled peak flows between the two software packages

It should also be recognised that the Manning's n value for the floodplain used in the Tuflow model has been programmed to vary with water depth to produce a reasonable match with the target VicRoads rational method estimate. The issue is discussed further below.

6.4. Hydraulics

6.4.1. Model Development

Two separate Tuflow models have been developed as part of the existing conditions modelling, being an undeveloped and an existing conditions model. The main difference between the two models is the proportion imperviousness, which has been set to zero for undeveloped conditions. The proportion of imperviousness for existing conditions has been based upon the zonings for the various areas as detailed as a figure included as Appendix D2.

The model has been set up with a grid size of 10 m to determine the critical duration and 5 m for the final existing conditions run. The respective time steps for the models are 4 seconds and 1 second.

- A range of 1 dimensional elements have been incorporated into the model as follows:
- 6 No. surveyed culverts under Anakie Road.
- Survey details of the Railway bridge crossing.
- 3 No. Council culverts from GIS data.

2 No. retarding basins were located within Study Area Boundary as follows:

- 122-N-RB (15 Baycrest Close Lovely Banks) - capacity is unknown.
- 119-N-RB (390 Tower Hill Road Lovely Banks) – 1050 m^3 capacity.

Of the two, only 119-N-RB only has been included in the model, since it is judged that 122- N-RB does not have a significant impact on flows due to its small capacity.

A thematic image of the DTM used in Tuflow modelling is included as Appendix D4. Manning's n roughness parameters applied across the model are presented as Appendix D5.

The RORB model results indicate that the critical duration storm for various sub areas vary between 1 hour and 12 hours. The Tuflow model has been run for the 1 hour, 2 hour, 9 hour, 12 hour and 48 hour duration events.

The downstream model boundary has been developed as follows:

- The modelled area has been extended a distance of around 200 m downstream of the investigation area defined in the Scope of Works.
- The railway bridge crossing and its retarding effects have been incorporated into the model as a 1D element, being a bridge. The effect of the bridge element in the Tuflow model was validated by setting up an independent HECRAS model of that bridge.
- A downstream hydraulic slope of 0.005 m/m has been adopted based upon the measured grade of the Cowies Creek waterway.

6.4.2. Existing Conditions Model

The existing conditions model has been set up and run in both RORB and Tuflow models. The comparative outflows from the two models are presented in Table 6-4.

Table 6-4 – 1 in 100 AEP Existing Conditions Flow Comparisons

Location	Catchment Area (km^2)	RORB		Tuflow (Adopted)	
		Critical Duration (hr)	Discharge (m^3/s)	Critical Duration (hr)	Discharge (m^3/s)
2 (Anakie Rd)	1.5	9	4.7	9	7.4
4 (Anakie Rd)	1.6	9	5.1	9	8.8
5 (Anakie Rd)	1.6	3	6.2	2	5.6
6 (Anakie Rd)	0.2	2	1.4	2	2.6
7 (Anakie Rd)	0.3	1	2.4	2	2.8
Outlet	20.7	12	42.6	12	35.3

It is proposed that the Tuflow flows should be adopted for use in the modelling.

Inundation maps have been included as Appendix D6 detailing inundation depths.

6.5. Sensitivity Assessment

The impact of varying various hydraulic model parameters were extensively tested as part of the Cowies Creek modelling. The results are presented in Table 6-5.

Table 6-5 – Tuflow Outflow Sensitivity Assessment

Model Conditions	Manning's n		Grid Size (m)	Duration (hr)	Discharge (m^3/s)	
	Waterway	Floodplain			250 m D/S of Lovely Banks Road	Railway line D/S (outlet)
Undeveloped	0.04	0.04	10	2	59.5	51.4
Undeveloped	0.04	0.07	10	2	48.0	44.6
Undeveloped	0.04	0.15	10	2	28.4	28.6
Existing	0.04	0.04	10	2	61.4	53.4
Existing	0.04	0.07	10	2	51.0	42.9
Existing	0.04	0.15	10	2	30.1	30.3
Undeveloped	0.04	0.07	10	9	59.1	56.5
Undeveloped	0.04	0.15	10	9	40.6	40.5
Existing	0.04	0.07	10	9	59.9	57.7
Existing	0.04	0.07	5	9	59.9	49.6
Existing	0.04	0.08	10	9	57.0	55.3
Existing	0.04	0.15	10	9	41.7	41.9
Existing	Variable with depth		10	9	55.7	35.5
Existing	Variable with depth		5	9	59.8	32.8
Undeveloped	Variable with depth		5	9	58.0	32.1
Undeveloped	Variable with depth		5	12	51.2	34.0
Existing	Variable with depth		5	12	51.7	35.3

The various runs demonstrate that the model is most sensitive to Manning's n values in the region of interest closer upstream of the railway line and closer to Lovely Banks Road. Downstream of Lovely Banks Road, the flow results are complicated by the presence of storage areas which retard flows to some extent depending upon Manning's n values and grid size. It may be appreciated that using a fixed Manning's n values which correspond to recommended values in the text result in peak flows which are well in excess of the VicRoads rational method estimates. The Manning's n parameter is the only mechanism by which the flows can be substantially reduced. The results above indicate that a fixed roughness value of 0.15 is required to reduce the peak flow at the outlet to a value which is within 30% of the rational method. Comparing the peak flows at the study area boundary (Anakie Road) with the rational method shows a closer correlation.

An alternate method of applying a varying Manning's n with water depth were tested to address this concern. The alternate method resulted in peak flow values within the reasonable range of the rational method. As a result, this method was adopted.

The adopted varying Manning's n for rural catchments are listed below.

- 0.2 for water depth up to 30 mm
- 0.045 for water depth at 100 mm or greater
- For water depth in between, a linear interpolation between 0.2 and 0.045 was applied

These parameters were also applied to the other catchments to ensure consistency across the study.

It is important to note that the bottom two results in Table 6-5 are influenced by the change in modelling approach to the railway bridge to more accurately represent the storage effect. Earlier model runs represented the opening as two cell widths, where the latter two runs represent the opening as a bridge.

6.6. Other Studies

There are two existing reports which have investigated the Cowies Creek catchment, being WaterTechnology (2009) and WaterTechnology (2004). In addition, CoGG has also previously developed their own model of the catchment. Outputs from those studies have been provided to allow a comparison with the outputs from the current study.

Outputs from WaterTechnology (2009) are only available for a point downstream with a larger catchment area. The flows have been extrapolated down to the smaller catchment area associated with this study applying the following equation as described in Grayson et.al. (1996):

$$Q_{(Smaller\ Catchment)} = Q_{(Larger\ Catchment)} \times (A_{(Smaller\ Catchment)} / A_{(Larger\ Catchment)})^{0.7}$$

Where:

Q = Discharge (m^3/s)

A = Catchment Area (km^2)

Note: Exponent can vary between 0.5 and 0.85. If data is available, the exponent may be calibrated, otherwise, 0.7 is typically applied.

The modelling described in WaterTechnology (2009) comprised a RORB model which was validated against a 1 in 100 AEP flow estimate derived using the VicRoads rational method procedure. Details of the analysis as supplied by CoGG are as follows:

- Catchment Area: 25.7 km^2.
- 1 in 100 AEP flow estimate: 46.6 m^3/s (9 hour critical duration).

Extrapolating the WaterTechnology (2009) reported outcomes to the location associated with this study results in a 1 in 100 AEP flow estimate of 40.1 m^3/s.

WaterTechnology (2004), estimates the design flow for Cowies Creek at the Geelong Ring Road to be 40.6 m^3/s. This estimate was derived through an application of the VicRoads rational method. The catchment area in WaterTechnology (2004) was estimated to be 23.2 km^2. Extrapolating the flow to the location associated with this study results in a 1 in 100 AEP flow estimate of 37.8 m^3/s.

The design flood estimate developed internally by CoGG using their own model incorporated the following characteristics:

- XP2D model with Tuflow engine & coupled Laurenson lumped hydrology.
- Runs undertaken with a 4 m grid and a 1.5 s time step.
- Manning's n of predominately 0.045 and a sub area schematisation which is coarser than that applied in this study.

The key outcomes relative to this study are presented in Table 6-6.

Table 6-6 – 1 in 100 AEP Flow Existing Conditions Study Comparison

Location	Discharge (m^3/s)					
	WaterTechnology (2009)	WaterTechnology (2004)	This Study (RORB)	CoGG		This Study (Tuflow)
				2h	9h	
2 (Anakie Rd)			4.7	4		7.4
4 (Anakie Rd)			5.0	7		8.8
5 (Anakie Rd)			6.0	6		5.6
6 (Anakie Rd)			1.3	1		2.6
7 (Anakie Rd)			2.1	2		2.8
Outlet	40.1	37.5	41.4	32	46	35.3

7. ST GEORGES DRAIN SUBCATCHMENT C155

7.1. Site Description

The St Georges Drain catchment is located in the south eastern portion of the study area. The outlet catchment is located along Princes Hwy. The total area of the catchment included in the investigation is 7.5 km^2, of which the study area comprises around 15%. There are two separate waterways or drainage lines which discharge flows across the study area boundary at the Geelong Ring Road. A map of the sub catchment is included as Figure 7-1.

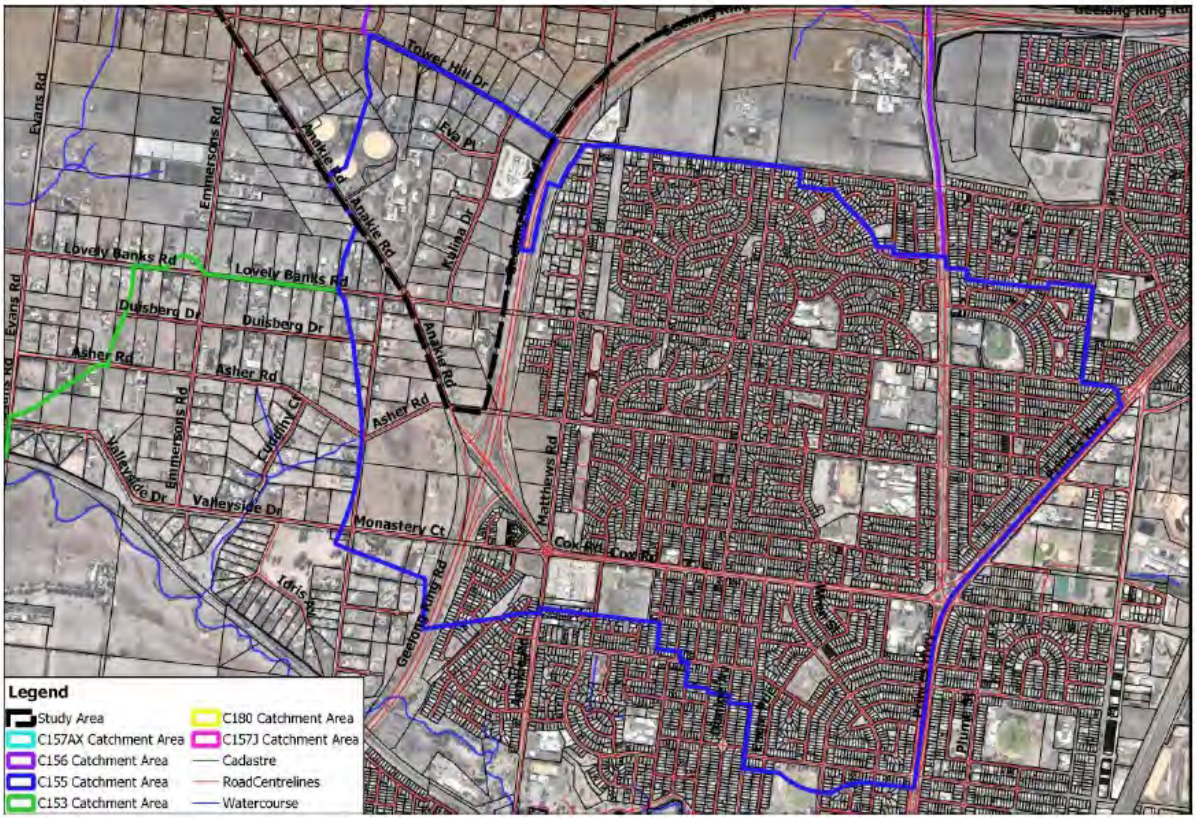

Figure 7-1: St Georges Drain Sub Area

There are two studies describing the investigation area being Water Technology (2004) and GHD (2004).

7.2. Hydrology

7.2.1. RORB Catchment Model

7.2.1.1. General

A RORB model has been developed for the St Georges Drain catchment. As described above, the model was calibrated by first generating the model without any impervious areas included in the catchment file. The model was calibrated to a rational method flow estimate and the impervious areas were subsequently placed into the model for the design runs.

For this catchment, a natural reach type was selected for the undeveloped conditions model and a mixture of natural and excavated, unlined waterway reach types were applied in the existing conditions model. The relative locations of different reach types are displayed in Figure 7-2.

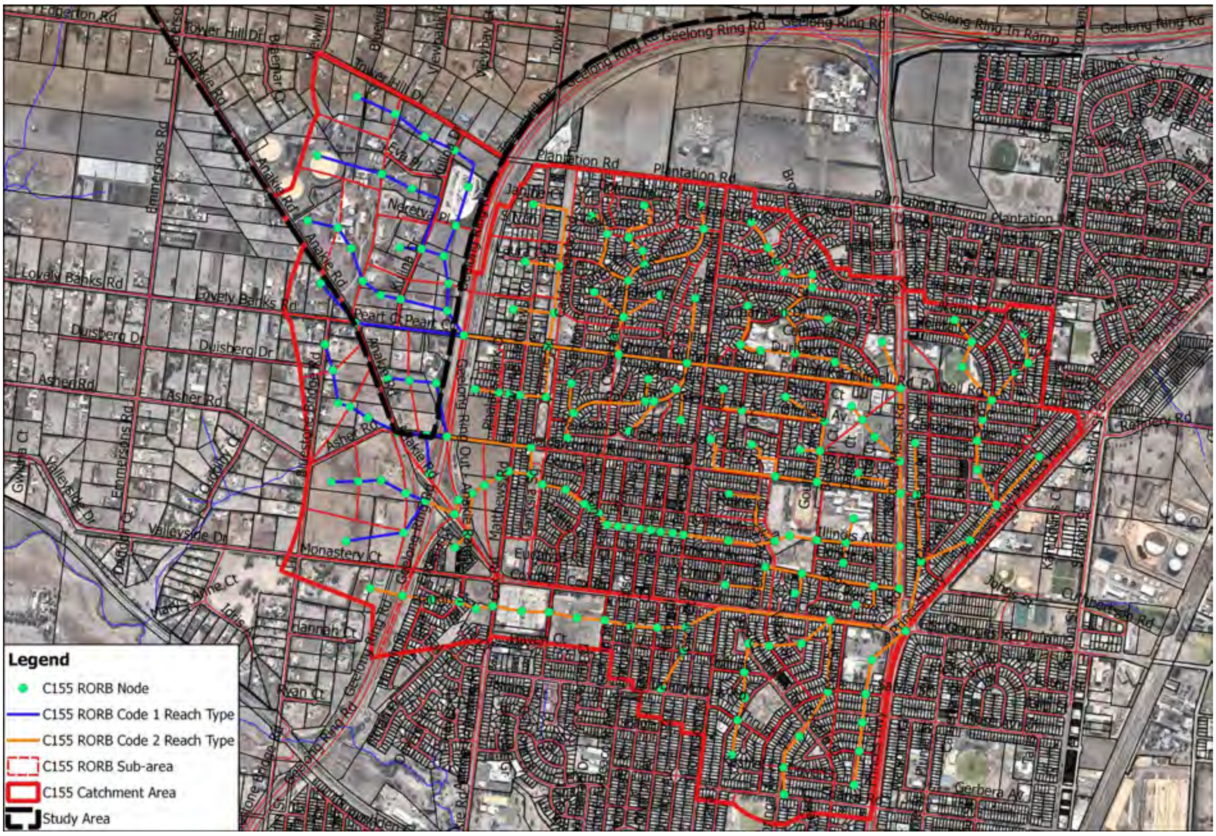

Figure 7-2: RORB Existing Conditions Reach type

7.2.1.2. Sub-Catchment Area

The St Georges Drain catchment model has an area of 7.5 km^2, which was delineated into 102 sub-catchment areas ranging in size between 3.7 and 10 ha. A schematic of the RORB model is included in Appendix E1.

The fraction impervious value for the undeveloped condition model is zero. The fraction impervious values applied to the existing conditions model were developed using the existing zonings. The basis upon which the RORB model impervious fractions were selected is included as a figure included in Appendix E2.

7.2.1.3. Delay Parameters

K_C Value

The range of k_C values which are plausible for the catchment have been estimated using a variety of methods as detailed in Table 7-1.

Table 7-1 - Estimate k_C parameter equations

ESTIMATE	EQUATION	Kc
Dandenong Creek and Westernport Catchments	$k_c = 1.53 * A^{0.55}$ ▪ A = Area (km^2)	4.6
Yarra and Maribyrnong Catchments	$k_c = 1.19 * A^{0.56}$ ▪ Where A = Area (km^2)	3.7
Vic(MAR>800mm)	$k_c = 2.57 * A^{0.45}$ ▪ A = Area (km^2)	6.4
Vic(MAR<800mm)	$k_c = 0.49 * A^{0.65}$ ▪ A = Area (km^2)	1.8

The results from the analysis indicate the range of kc values that would be expected for the catchment are between 1.8 and 6.5.

m Value

An m value of 0.8 has been adopted as discussed above.

Initial Loss and Runoff Coefficient

Losses of 15 mm IL and 2 mm/hr CL as discussed above have been adopted for use and only varied if necessary to achieve an appropriate match with the target calibration flow.

7.2.2. Flow Estimation Comparison

The RORB model has been calibrated to the VicRoads rational method flows with a 1 in 100 AEP. The rainfall runoff process conceptualisation is different between RORB and the rational method. The result of these differences is that the modelled outputs are not consistent across different probability events, durations and catchment areas.

The approach taken in calibrating the RORB model has been to ensure to the extent possible that the 1 in 100 AEP outflows do not vary by more than 30% form the VicRoads rational method estimates. A range of locations within the catchment have been considered when undertaking the calibration. The details of the modelled outcomes are presented in Appendix E3 and the outcome are displayed graphically for both the 1 in 100 AEP and the 1 in 5 AEP event as Figure 7-3 and Figure 7-4.

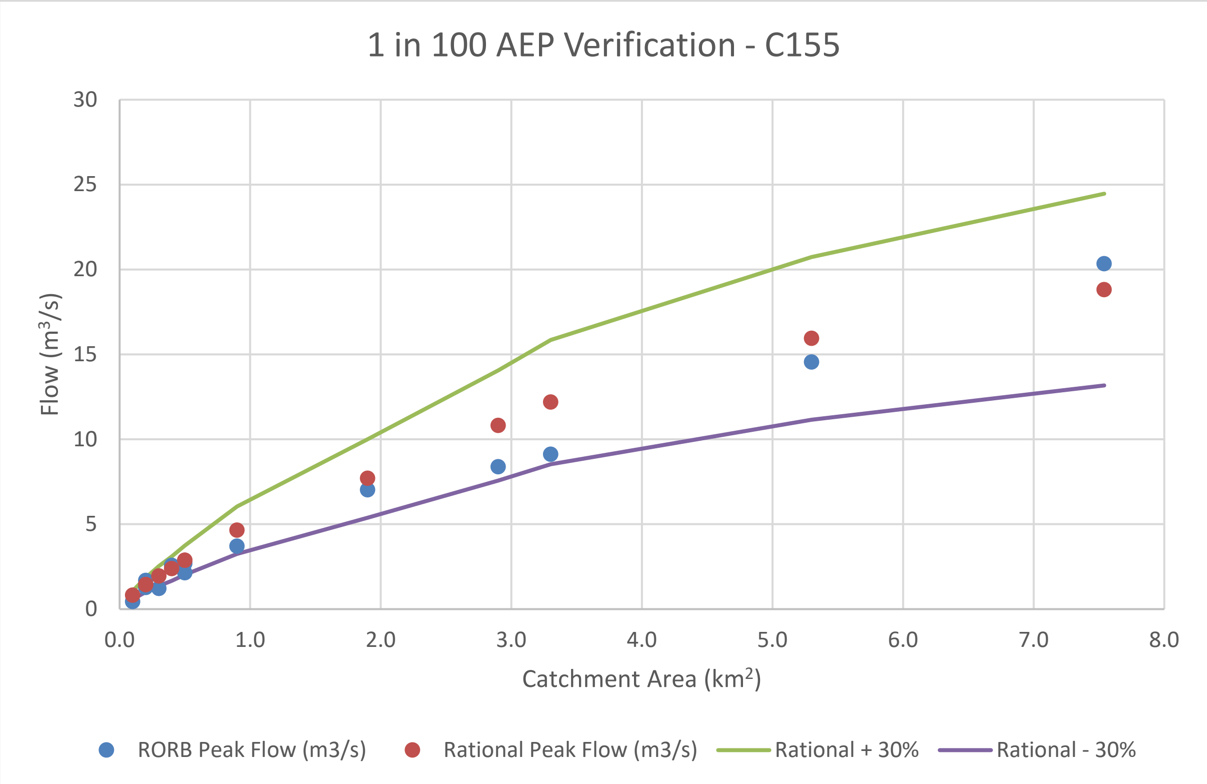

Figure 7-3: 1 in 100 AEP flow comparisons RORB vs VicRoads rational method

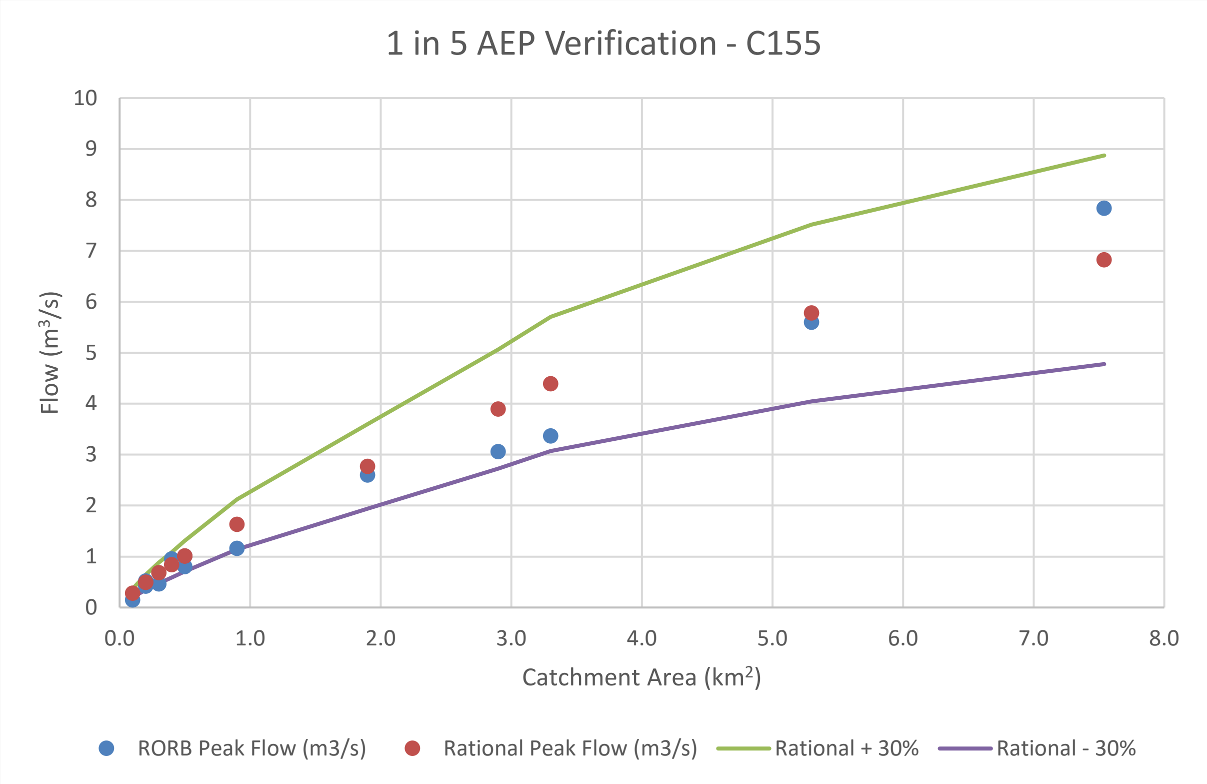

Figure 7-4: 1 in 5 AEP flow comparisons RORB vs VicRoads rational method

It may be observed from the above figure that the gradient of the runoff versus catchment area graph is different for the two modelling procedures. The RORB delay parameters is the predominate variable used for calibration and the parameter set detailed in Table 7-2 was found to offer the best compromise fit to the target flows.

Table 7-2 - Adopted RORB parameters

Parameter	Value
m	0.8
K_C	4.0
K_c/d_{av}	1.84
Initial Loss (mm)	15.0
Continuing Loss (mm/hr)	2.0

The K_c/d_{av} is 20% lower than the Cowies Creek figure of 2.26. The smaller figure suggests that this catchment is more efficient than Cowies at generating runoff.

In addition to the VicRoads rational method and RORB modelled outcomes, both Tuflow and RFFE outcomes have been generated. The locations within the model considered to be of greatest relevance in the study are the overall catchment outlet and also the locations where waterways or flow paths cross the study area boundary. The flow at each of those locations has been reported upon comparatively in Table 7-3.

Table 7-3 – 1 in 100 AEP Flow estimation comparisons, undeveloped models

Location	Catchment Area (km^2)	RORB Critical Duration (hr)	Discharge (m^3/s)			
			VicRoads	RORB	RFFE	Tuflow
1 (Geelong Ring Road)	0.9	2	4.6	3.7	2.16	3.0 (2hr)
2 (Geelong Ring Road)	0.4	2	2.4	2.6		2.2 (9hr)
Outlet	7.5	12	18.8	20.3	12.5	21.9 (9hr)

The locations numbers referred to in the above table are detailed on the RORB sub area catchment map included as Appendix E1.

From the above table it is observed that the Tuflow flow estimates are similar in magnitude to the VicRoads rational method and RORB flows. Differences in flows at Geelong Ring Road are considered to be due to:

- The ability of Tuflow to simulate rainfall runoff processes as discussed for the Cowies Creek catchment.
- The differences in assumed contributing areas for the RORB model as opposed to those estimated by Tuflow.

7.3. Hydraulics

7.3.1. Model Development

Two separate Tuflow models have been developed as part of the existing conditions modelling, being an undeveloped and an existing conditions model. The proportion imperviousness varies between the two models and has been set to zero for undeveloped conditions. The proportion of imperviousness for existing conditions has been based upon the zonings for the various areas as detailed as a figure included as Appendix E2. In addition, the existing conditions model includes underground trunk drainage, whereas the undeveloped conditions model does not include any drainage.

The model has been set up with a grid size of 10 m to determine the critical duration and 5 m for the final existing conditions run. The respective time steps for the models are 4 seconds and 2 seconds.

A range of 1 dimensional elements have been incorporated into the model as follows:

- 3 No. culverts under the Geelong Ring Road, identified in Anakie Road.
- Retarding basin 111-N-RB
- Trunk drainage lines with a diameter of 600 mm or greater. (Includes 10 no. sub branches)

Number of smaller retarding basins are included in the investigation area, but outside of the study area boundary. These retarding basins have not been included in the modelling.

A thematic image of the DTM used in Tuflow modelling is included as Appendix E4. Manning's n roughness parameters applied across the model are presented as Appendix E5.

The RORB model results indicate that the critical duration storm for various sub areas vary between the 2 hour and 12 hours. The Tuflow model has been run for the 1 hour, 2 hour, 9 hour, 12 hour and 48 hour duration events.

The downstream model boundary has been developed using the following criteria:

- The modelled area has been extended a distance of around 150 m downstream of the investigation area defined in the Scope of Works.
- A downstream hydraulic slope of 0.003 m/m has been adopted based upon the measured grade of the downstream floodplain.

Information available from other studies is as follows:

- Modelling described in GHD (2014) indicates that at the intersection of Princes Highway and Detroit Crescent, a surface flow of 4.1 m^3/s corresponds to a surface elevation of 9.2 mAHD and a flow of 13.8 m^3/s corresponds to a surface elevation of 9.26 mAHD.

7.3.2. Existing Conditions Model

The existing conditions model has been set up and run in both RORB and Tuflow models. The comparative outflows from the two models are presented in Figure 7-4Table 7-4.

Table 7-4 – 1 in 100 AEP Existing Conditions Flow Comparisons

Location	Catchment Area (km^2)	RORB		Tuflow (Adopted)	
		Critical Duration (hr)	Discharge (m^3/s)	Critical Duration (hr)	Discharge (m^3/s)
1	0.9	2	4.1	2	3.5
2	0.4	2	2.9	2	2.2
Outlet	7.5	2	44.7	2	24.5

In contrast to all other catchment, the Tuflow modelled outcomes for this catchment are substantially lower at the outlet than the RORB modelled outcomes. The RORB model flows suggest a more substantial increase in runoff between undeveloped and existing conditions. This pattern is not evident at the Geelong Ring Road, however, where the Tuflow model outcomes are higher at location 1 than the RORB model outcomes. The variation in relative flows is considered to be due to the different modelling assumptions between the two models. More specifically is due to the alteration in the reach definition in RORB. The reach type changes at the Geelong Road interface from natural to excavated, unlined.

The RORB model simulates a single runoff routing element with flows that increase in a regular monotonic fashion. The Tuflow model incorporates two separate catchment elements being the piped network and the overland flow network. The Tuflow model outcomes will produce a relatively quick response time up until the capacity of the drainage network and from that point on, the catchment response will be slower. This is especially the case for this catchment since overland flows are not directed to a waterway, but instead are conveyed through a less hydraulically efficient street network.

Inundation maps have been included as Appendix E6 detailing inundation depths. It may be noted that the inundation maps indicate that there is some water which discharges into the adjacent Wharf Road sub catchment. This phenomenon has been reviewed and it has been found that:

- The flow can occur in both directions, i.e. flows from the Wharf Road catchment can discharge to the St Georges Drain at the same time that flows from the St Georges Drain catchment flow to the Wharf Road sub catchment.
- The magnitude and timing of the flow in both directions are similar. So, the hydraulic interchange between the two models is essentially equal.

Further quantification of the peak water surface interaction between the two catchments is presented in the figures below.

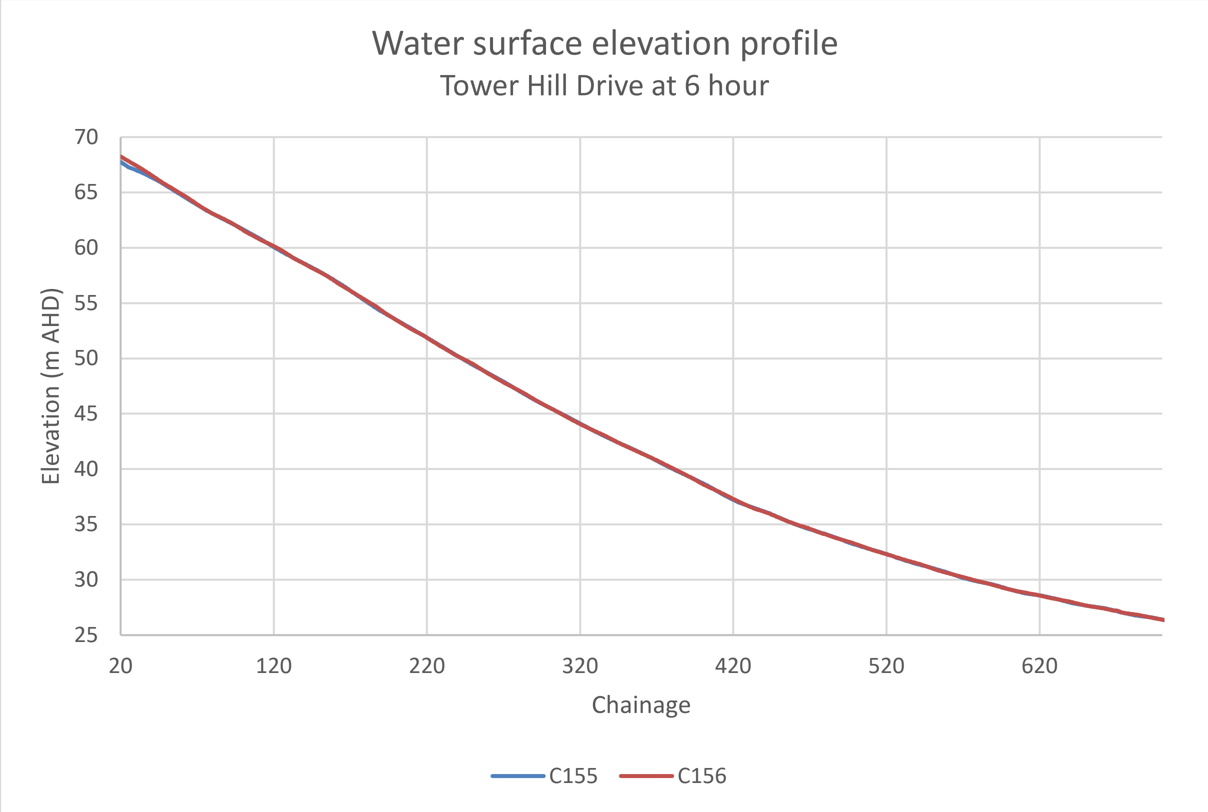

Figure 7-5: Water surface elevation profile of C155 and C156

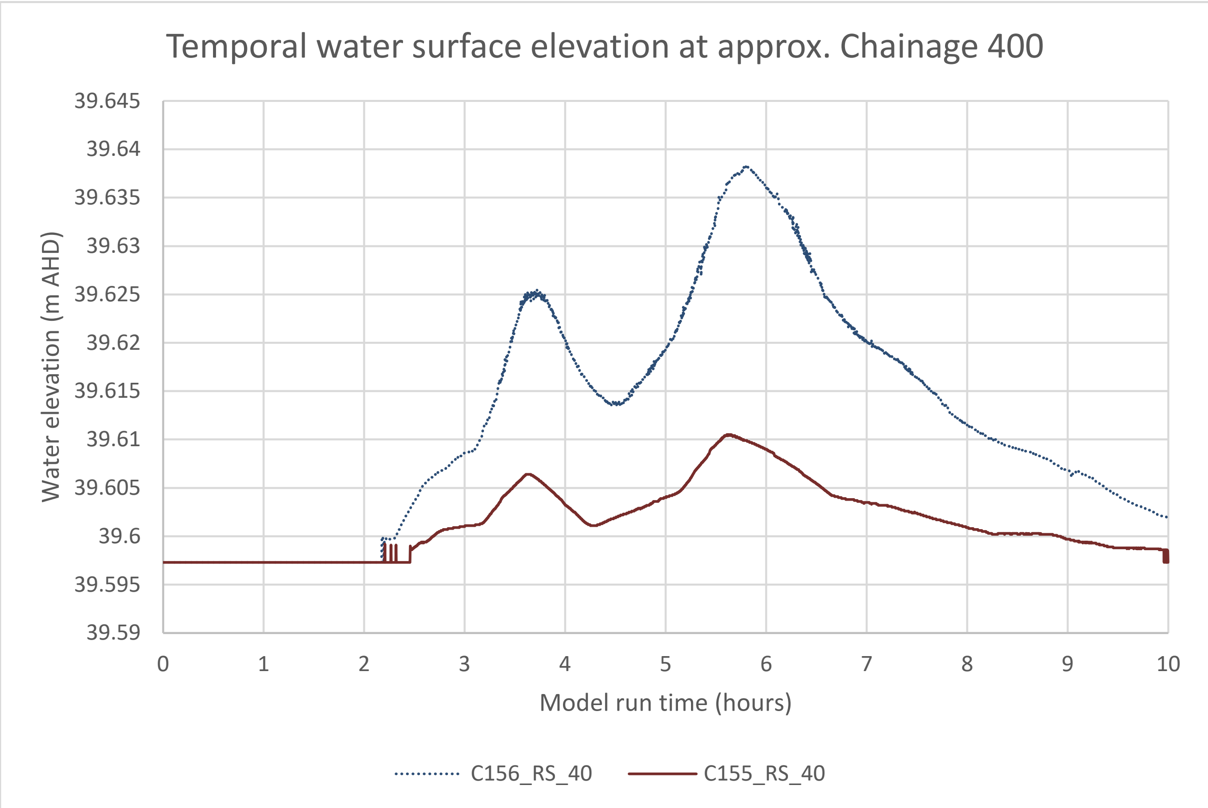

Figure 7-6: Comparison of temporal water surface elevation C155 & C156

The two figures above show a comparison of the Tuflow peak water surface elevation along Tower Hill Drive (model boundary). Figure 7-5 shows the water surface profile along the model interface with the difference in levels at 6 hour of the simulation. Figure 7-6 shows the head difference between two water levels against the entire simulation time at a specific location. The average head difference is 30mm at the model boundary. The hydraulic head has been converted into an equivalent flow and incorporated into the final Tuflow model input. The resulting cross flows are small. Therefore, there is a negligible effect on the flood extent and peak flows to the overall study outcomes and does not warrant further investigation at this stage of the catchment investigation.

It is important to note that the modelling undertaken does not incorporate the existing drainage network in a high level of detail as would typically be required of a flood study. The modelling undertaken for this study is intended to be used to assess the comparative impact of development on the downstream area.

With respect to the modelled flood levels, the results should be considered to be preliminary. A flood study to determine peak water surface levels would typically include more detailed modelling inclusive of additional survey and a more complete representation of the drainage network.

At the time when a Precinct Structure Plan (PSP) is developed for the region, consideration may be given to combining the St Georges and Wharf Road catchments. The imperative to combine the catchments would be contingent upon the proposed PSP boundaries along with any topographic modifications such as fill placement which may modify catchment flow paths. In particular, it would be important to combine the catchments if any retardation infrastructure is proposed in the vicinity of the catchment boundary.

7.4. Sensitivity Assessment

The impact of varying various hydraulic model parameters was tested as part of the modelling. A variety of alternative runs which were undertaken trialling different grid sizes and roughness parameters and the results are presented in Table 7-5.

Table 7-5 – Tuflow Outflow Sensitivity Assessment

Model Conditions	Manning's n		Grid Size (m)	Duration (hr)	Discharge (m^3/s)
	Waterway	Floodplain			
Undeveloped	0.04	0.04	10	2	22.2
Undeveloped	0.04	0.07	10	2	17.4
Existing	0.04	0.04	10	2	24.7
Existing	0.04	0.07	10	2	23.0
Existing	0.04	0.07	5	2	24.7
Existing	0.04	0.07	10	9	22.8
Existing	0.04	0.07	5	9	23.2
Existing	varies (as for Cowies Creek)		5	2	24.5

Overall, the outflows estimated by the model at the downstream extent are found to be relatively insensitive to the tested parameters. It is considered likely that this is due to the fact that the majority of the 1 in 100 AEP flows are carried in the stormwater pipe system rather than as overland flows. The stormwater pipe system is largely unaffected by the various trialled parameters.

7.5. Other Studies

There are two existing reports which have investigated the St Georges Drain catchment, being GHD (2014) and WaterTechnology (2004). Of those two reports, only GHD (2014) has provided flows that may be used as a point of comparison for our project.

The most obvious flow comparison point available in GHD (2014) is located close to the intersection of Bacchus Marsh road and Princes Highway. GHD (2014) constructed both a RORB model of the catchment and a quasi 2 dimensional hydraulic model – XP-UDD2000. Two estimates are available from the report being RORB and a XP-UDD2000 outputs. Of the two, the XP-UDD2000 is considered to be more accurate since it explicitly accounts for attenuation of flows through the flatter urbanised portion of the catchment.

RORB parameters from the GHD (2014) differ to those applied in this study in the following respects:

- kc/dav = 1.46 which is 20% lower than the figure used in this study. This figure would result in a lesser degree of attenuation in comparison with the outputs from this study.
- IL = 12.5 mm. This figure is similar to that adopted for this study.
- Proportional Loss (PL) varies between 0.3 and 0.6 depending upon storm duration. This study has applied a continuing loss rather than a proportional loss. The impact of using the alternate loss model is unclear without a targeted analysis.

The key outcomes relative to this study are presented in Table 7-6.

Table 7-6 – 1 in 100 AEP Flow Existing Conditions Study Comparison

Location	Catchment Area (km²)	GHD (2014)		This Study	
		Critical Duration (hr)	Discharge (m³/s)	Critical Duration (hr)	Discharge (m³/s)
Outlet (RORB)	7.5	NA	52.5	2	44.7
Outlet (Hydraulic Model)	7.5	2	26.9 (Total) 13.8 (Surface) 13.1 (Piped)	2	24.5 (Total) 14.5 (Surface) 10.0 (Piped)

From the above table, it is observed that the RORB models from the two studies are producing similar flows, but both are higher than the hydraulic model flows. The RORB flow is quite high for the study catchment when considering the catchment area and the catchment surface grade.

The attenuated flows at the downstream end of the model are consistent between both studies.

8. WHARF ROAD SUBCATCHMENT C156

8.1. Site Description

The Wharf Road catchment is located in the south western portion of the study area. The outlet of catchment is located at the Bacchus Marsh Road south of the Geelong Ring Road. The total area of the catchment included in the investigation is 4.7 km^2, of which the study area comprises around 85%. There are three separate culverts which discharge flows across the study area boundary, at Geelong Ring Road. A map of the sub catchment is included as Figure 8-1.

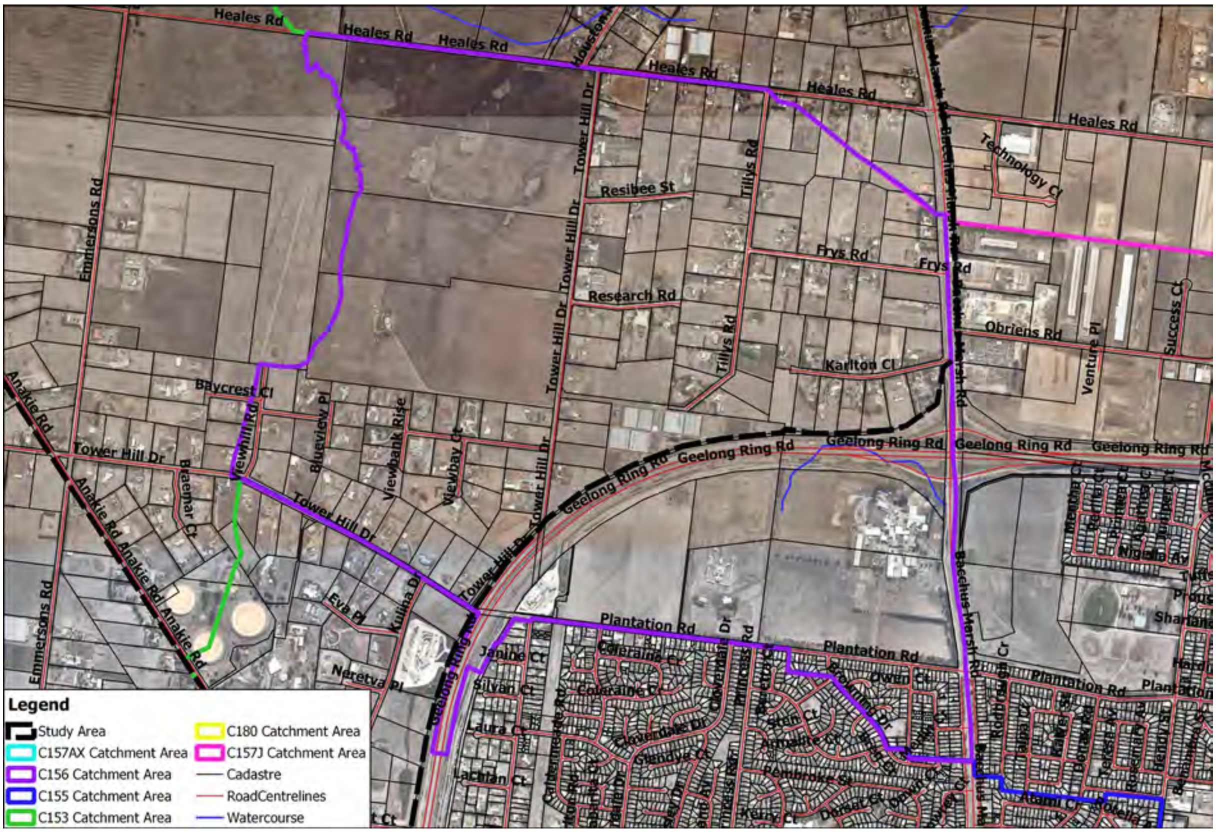

Figure 8-1: Wharf Road Sub Area

There are two studies describing the investigation area being Water Technology (2004) and BMTWBM (2011).

8.2. Hydrology

8.2.1. RORB Catchment Model

8.2.1.1. General

A RORB model has been developed for the Wharf Road catchment. As described above, the model was calibrated by first generating the model without any impervious areas included in the catchment file. The model was calibrated to a rational method flow estimate and the impervious areas were subsequently placed into the model for the design runs.

For this catchment, a natural reach type was selected for the undeveloped conditions model and a mixture of natural and excavated, unlined waterway reach types were applied in the existing conditions model. The relative locations of different reach types are displayed in Figure 8-2.

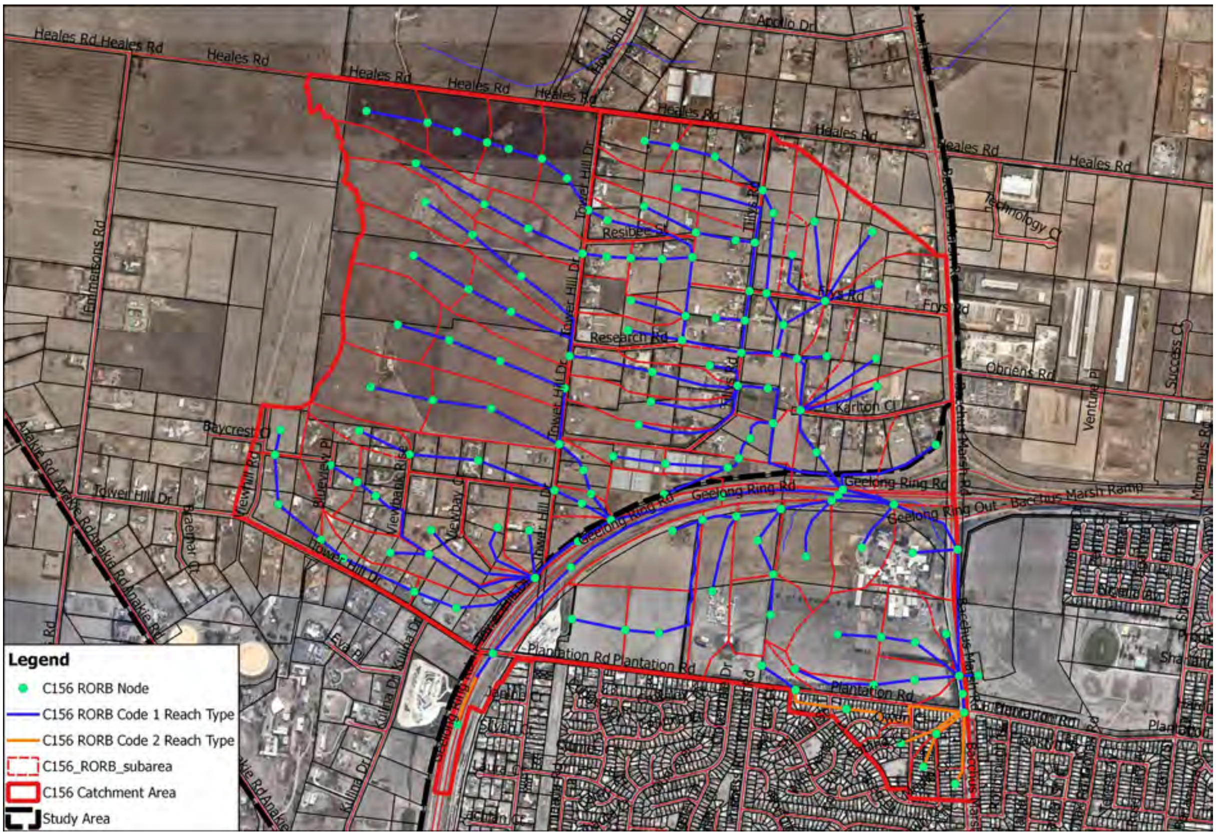

Figure 8-2: RORB Existing Conditions Reach type

8.2.1.2. Sub-Catchment Area

The Wharf Road sub catchment model has an area of 4.7 km^2, which was delineated into 84 sub-catchment areas ranging in size between 2.0 and 10 ha. A schematic of the RORB model is included in Appendix F1.

The fraction impervious value for the undeveloped condition model is zero. The fraction impervious values applied to the existing conditions model were developed using the existing zonings. The basis upon which the RORB model impervious fractions were selected is included as a figure included in Appendix F2.

8.2.1.3. Delay Parameters

<u>**K_C Value**</u>

The range of k_C values which are plausible for the catchment have been estimated using a variety of methods as detailed in Table 8-1.

Table 8-1 - Estimate k_C parameter equations

ESTIMATE	EQUATION	Kc
Dandenong Creek and Westernport Catchments	$k_c = 1.53 * A^{0.55}$ ▪ A = Area (km^2)	3.6
Yarra and Maribyrnong Catchments	$k_c = 1.19 * A^{0.56}$ ▪ Where A = Area (km^2)	2.8
Vic(MAR>800mm)	$k_c = 2.57 * A^{0.45}$ ▪ A = Area (km^2)	5.2
Vic(MAR<800mm)	$k_c = 0.49 * A^{0.65}$ ▪ A = Area (km^2)	1.3

The results from the analysis indicate the range of kc values that would be expected for the catchment are between 1.3 and 5.2.

m Value

An m value of 0.8 has been adopted as discussed above.

Initial Loss and Runoff Coefficient

Losses of 15 mm IL and 2 mm/hr CL as discussed above have been adopted for use and only varied if necessary to achieve an appropriate match with the target calibration flow.

8.2.2. Flow Estimation Comparison

The RORB model has been calibrated to the VicRoads rational method flows with a 1 in 100 AEP. The rainfall runoff process conceptualisation is different between RORB and the rational method. The result of these differences is that the modelled outputs are not consistent across different probability events, durations and catchment areas.

The approach taken in calibrating the RORB model has been to ensure to the extent possible that the 1 in 100 AEP outflows do not vary by more than 30% form the VicRoads rational method estimates. A range of locations within the catchment have been considered when undertaking the calibration. The details of the modelled outcomes are presented in Appendix F3 and the outcome are displayed graphically for both the 1 in 100 AEP and the 1 in 5 AEP event as Figure 8-3 and Figure 8-4.

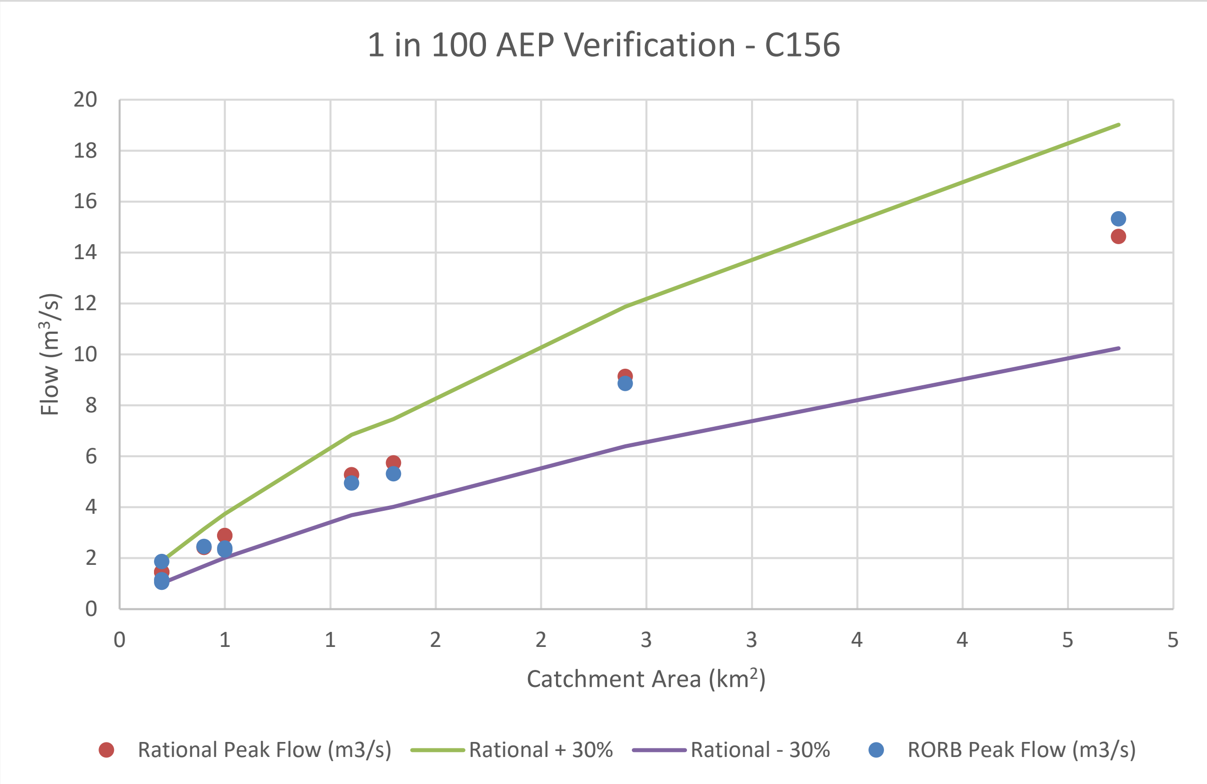

Figure 8-3: 1 in 100 AEP flow comparisons RORB vs VicRoads rational method

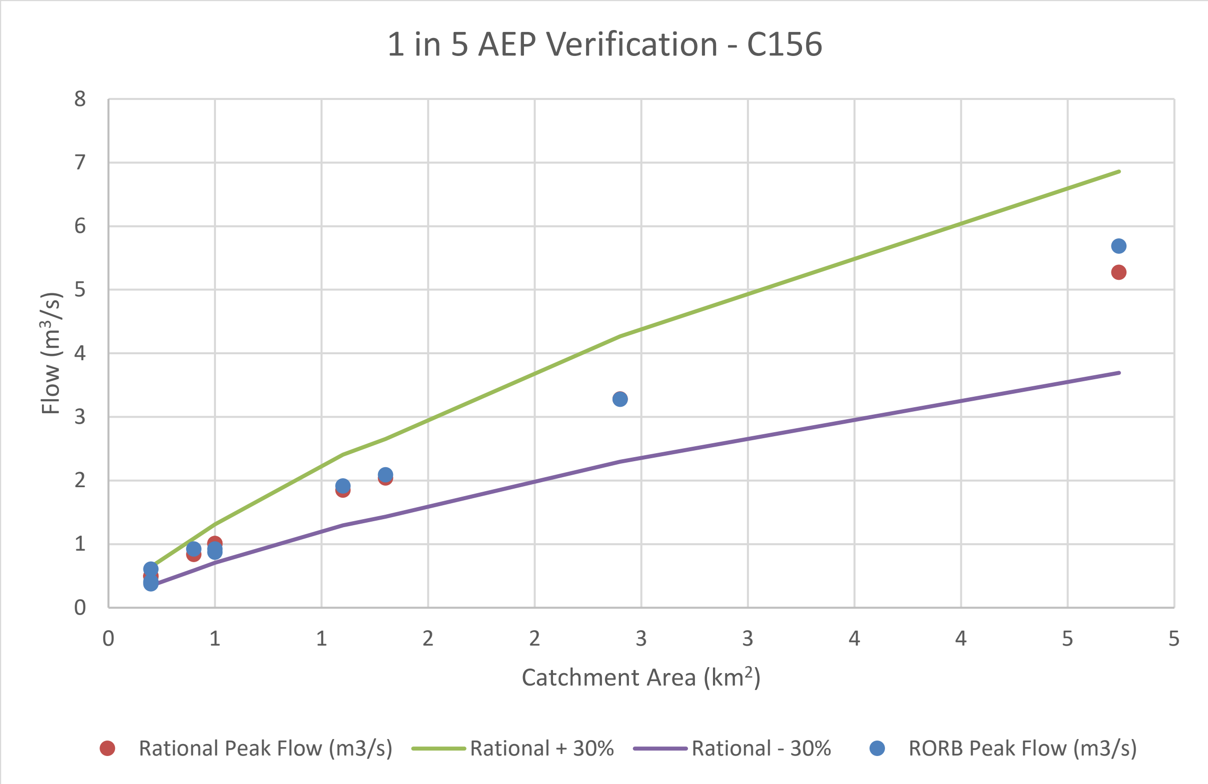

Figure 8-4: 1 in 5 AEP flow comparisons RORB vs VicRoads rational method

The RORB delay parameters is the predominate variable used for calibration and the parameter set detailed in Table 8-2 was found to offer the best compromise fit to the target flows.

Table 8-2 - Adopted RORB parameters

Parameter	Value
m	0.8
Kc	3.5
K_c/d_{av}	1.6
Initial Loss (mm)	15.0
Continuing Loss (mm/hr)	2.0

The K_c/d_{av} figure for Wharf Road is lower than the Cowies Creek figure of 2.26. This outcome suggests that this catchment is more efficient than the Cowies Creek catchment and that flows will attenuate less rapidly per unit stream length.

In addition to the VicRoads rational method and RORB modelled outcomes, both Tuflow and RFFE outcomes have been generated. The locations within the model considered to be of greatest relevance in the study are the overall catchment outlet and also the locations where waterways or flow paths cross the study area boundary. The flow at each of those locations has been reported upon comparatively in Table 8-3.

Table 8-3 – 1 in 100 AEP Flow estimation comparisons, undeveloped models

Location	Catchment Area (km^2)	RORB Critical Duration (hr)	Discharge (m^3/s)			
			VicRoads	RORB	RFFE	Tuflow
1 (Ring Rd)	0.50	9	2.9	2.4	1.4	2.6 (2hr)
2 (Ring Rd)	1.10	2	5.3	5.0	2.5	3.5 (9hr)
3 (Ring Rd))	2.40	9	9.1	8.9	4.7	9.5 (9hr)
Outlet	4.7	9	14.6	15.3	8.8	16.1 (9hr)

The locations numbers referred to in the above table are detailed on the RORB sub area catchment map included as Appendix F1.

From the above table it is observed that the Tuflow outcomes are lower at location 2 and higher at the outlet than the VicRoads and RORB modelled outcomes but not unreasonably so. This is due to similar reasons discussed for the St Georges and Cowies Creek catchments. It is considered that the Tuflow model provides a stronger basis for establishing flow paths and concentration points than RORB. Notwithstanding differences in peak flow estimates, Tuflow provides a stronger basis for relative flow estimation throughout the catchment compared to the other two models.

8.3. Hydraulics

8.3.1. Model Development

Two separate Tuflow models have been developed as part of the existing conditions modelling, being an undeveloped and an existing conditions model. The main difference between the two models is the proportion imperviousness, which has been set to zero for undeveloped conditions. The proportion of imperviousness for existing conditions has been based upon the zonings for the various areas as detailed as a figure included as Appendix F2.

The model has been set up with a grid size of 10 m to determine the critical duration and 5 m for the final existing conditions run. The respective time steps for the models are 4 seconds and 2 seconds.

A range of 1 dimensional elements have been incorporated into the model as follows:

- 3 No. culverts under the Geelong Ring Road.
- 1 No. culvert under Bacchus Marsh Road.
- 1 No. culvert under Karlton Close.
- 2 No. retarding basins, 122-N-RB & 108-N-RB.

A single small retarding basin is included in the investigation area, but outside of the study area boundary and has not been included in the modelling.

A thematic image of the DTM used in Tuflow modelling is included as Appendix F4. Manning's n roughness parameters applied across the model are presented as Appendix F5.

The RORB model results indicate that the critical duration storm for various sub areas vary between 2 hours and 9 hours. The Tuflow model has been run for the 2 hour, 3 hour, 9 hour, 12 hour and 48 hour duration events.

The downstream model boundary has been developed using the following criteria:

- The modelled area has been extended a distance of around 530 m downstream of the investigation area defined in the Scope of Works.

- A downstream hydraulic slope of 0.005 m/m has been adopted based upon the measured grade of the downstream floodplain.

Information available from other studies is as follows:

- Modelling described in BMTWBM (2011) indicates that at the Bacchus Marsh Road culvert, a flow 22.1 m^3/s corresponds to a flood depth of 16.1 mAHD.

8.3.2. Existing Conditions Model

The existing conditions model has been set up and run in both RORB and Tuflow models. The comparative outflows from the two models are presented in Table 8-4.

Table 8-4 – 1 in 100 AEP Existing Conditions Flow Comparisons

Location	Catchment Area (km^2)	RORB		Tuflow (Adopted)	
		Critical Duration (hr)	Discharge (m^3/s)	Critical Duration (hr)	Discharge (m^3/s)
1 (Ring Rd)	0.50	2	3.0	2	3.3
2 (Ring Rd)	1.10	3	5.2	9	3.7
3 (Ring Rd))	2.40	9	10.4	9	10.1
Outlet	4.7	9	16.3	9	17.4

As per the modelling methodology, the flows from the Tuflow model have been adopted for use in this study.

Inundation maps have been included as Appendix F6 detailing inundation depths.

8.4. Sensitivity Assessment

The impact of varying various hydraulic model parameters was tested as part of the modelling. A variety of alternative runs which were undertaken trialling different grid sizes and roughness parameters and the results are presented in Table 8-5.

Table 8-5 – Tuflow Outflow Sensitivity Assessment

Model Conditions	Manning's n		Grid Size (m)	Duration (hr)	Discharge (m^3/s)
	Waterway	Floodplain			
Undeveloped	0.04	0.04	10	2	19.1
Undeveloped	0.04	0.07	10	2	13.6
Undeveloped	0.04	0.07	10	3	14.9
Undeveloped	0.04	0.07	10	9	16.4
Existing	0.04	0.04	10	2	18.5
Existing	0.04	0.07	10	2	16.6
Existing	0.04	0.07	10	3	15.3
Existing	0.04	0.07	10	9	17.1
Existing	0.04	0.07	5	9	17.9
Existing	varies (as for Cowies Creek)		5	9	17.4

The outcomes presented above indicates that the peak flows are mildly sensitive to the Manning's n vales and that the grid size has some impact upon predicted peak flows. In this catchment, reducing the grid size has had the effect of slightly increasing the peak flows in contrast to other sub catchments where reducing grid size has resulted in a reduction of peak outflows.

8.5. Other Studies

There are two existing reports which have investigated the Wharf Road Drain catchment, being BMTWBM (2011) and WaterTechnology (2004). Of those two reports, only BMTWBM (2011) has provided flows that may be used as a point of comparison for our project. Flows from WaterTechnology (2004) are not clearly labelled in order that a direct comparison may be made at the Geelong Ring Road. In addition, the report appears to provide two sets of flows for culvert locations which appear to vary widely.

The most obvious flow comparison point available in BMTWBM (2011) is located at the Bacchus Marsh Road culvert. BMTWBM (2011) reports on development of both a RORB model of the catchment and a 2 dimensional hydraulic model Tuflow. The RORB model outputs are used as inputs to the Tuflow model at the Bacchus Marsh Road culvert.

RORB parameters in BMTWBM (2011) differ to those applied in this study in the following respects:

- The RORB model has been calibrated to the rational method, it is presumed that the VicRoads version was applied. The model was calibrated to a point at the downstream end of their model, corresponding to a catchment area of around 9.56 km^2, or double the catchment for this study.
- Reach type 2 was applied to the modelling rather than the reach type 1 used in this study.
- The kc delay parameter was set using the $k_c = 1.19 * A^{0.56}$ equation. This equation results in a shorter relative delay when compared to the equation applied in this study.
- A Proportional Loss (PL) of 0.6 was applied. This study has applied a continuing loss rather than a proportional loss. The impact of using the alternate loss model is unclear without a targeted analysis.

The key outcomes relative to this study are presented in Table 8-6.

Table 8-6 – 1 in 100 AEP Flow Existing Conditions Study Comparison

Location	Catchment Area (km^2)	Discharge (m^3/s)	
		BMTWBM (2011)	This Study
Outlet	4.7	22.2	17.4

Notwithstanding the potential to reduce the overall catchment flows, it is observed that the flow estimate from this study is consistent with that estimated in BMTWBM (2011).

8.6. Downstream Impacts

Additional hydraulic modelling in Tuflow was completed for the downstream areas of the Wharf Road catchment, as part of the existing conditions assessment at the request of CoGG. The aim of this extension to modelling is to clarify any potential impacts of development to regions further downstream.

8.6.1. Model Extension Setup

The total extent of the hydraulic modelling for the Wharf Road catchment is shown in Figure 8-5. CoGG nominated that the downstream boundary location be set at Princess Hwy.

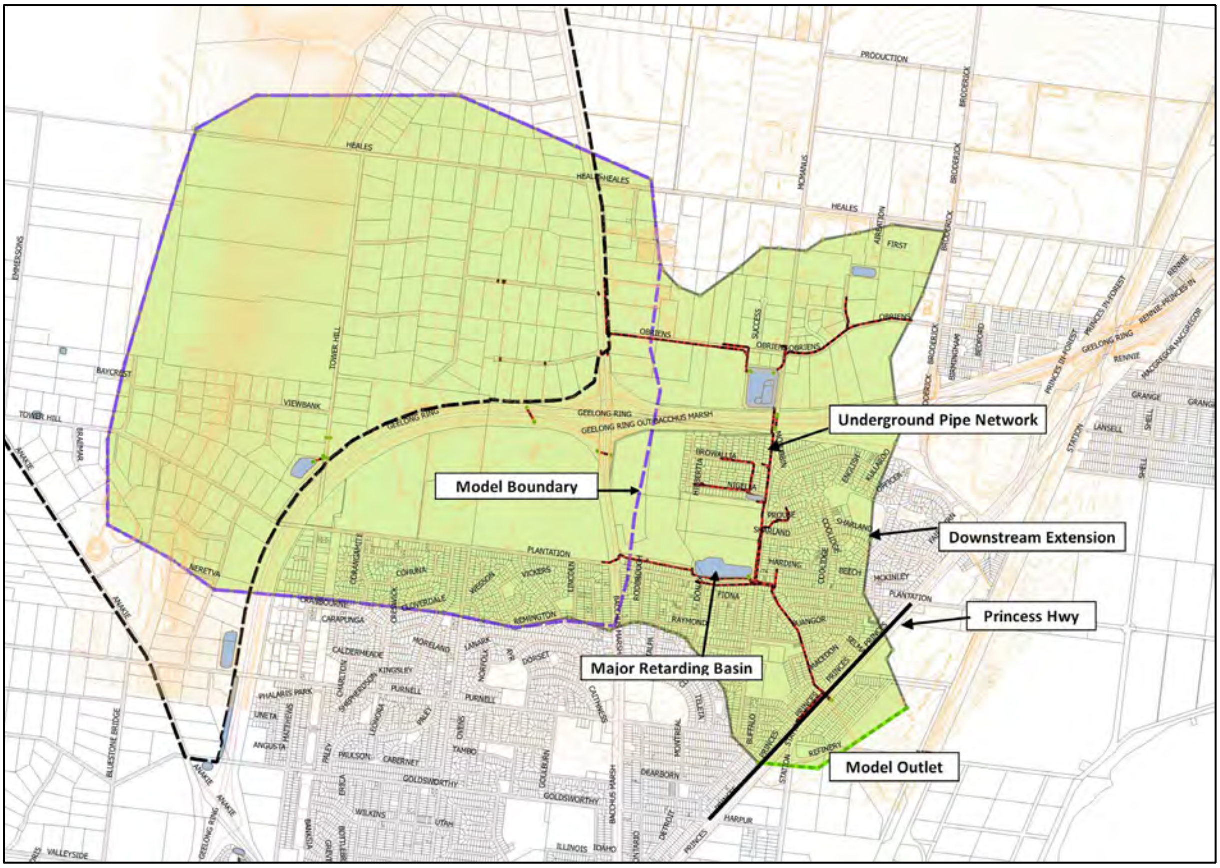

Figure 8-5: Wharf Road Downstream Model Extension

The original model extent is delineated by the purple dotted line in the above figure. The additional modelled area has been set up differently to the upstream model and the models for other sub catchments. It has been set up as an independent model with its upstream input being the outflow hydrograph from the upstream model. In addition, the rainfall input to the model has been represented using a ‘Rainfall Excess on Grid’ approach. A single event only has been modelled, being the critical duration storm for the upstream catchment, the 9 hour 1 in 100 AEP event.

Like the St Georges Road catchment, the main trunk drainage for the existing underground network was included in the model down to a nominal pipe diameter of 600 mm. It is judged that the smaller diameter pipe network will not have a considerable impact on flood conditions in the 1 in 100 AEP event. Two large retarding basins within the modelled areas were included in the model.

The drainage pipe invert levels have been extracted from CoGG GIS data. In cases where the invert levels and pit types were not available, the invert levels were estimated based on the available Lidar data assuming a typical pit and cover depth. Some judgement on the invert levels of the retarding basin outlets have been made to allow the retarding basin to function as per intended.

The Manning’s roughness parameters applied in the downstream areas are consistent with the main model which are described in Table 5-1.

It should be noted that the results from the model extension should not be relied upon for estimating accurate absolute inundation extents or levels in the downstream areas. The explicit aim of the additional modelling is to provide a comparative analysis of the effects of the upstream development on the existing downstream areas.

8.6.2. Existing Conditions Downstream

The modelling results shows that the two major retarding basins located adjacent to the Geelong Ring Road and Plantation Road (110-N-RB) have a significant effect on the flood extent in a 1 in 100 AEP flood event. The underground drainage line along Streeton Close and Kosciusko Avenue convey much of the flows from the two storage basins. The model has calculated the peak flow to be 9.5 m^3/s in the 1950 mm diameter pipe with approximately 2 m^3/s conveyed halfway along Kosciusko Ave. The peak outflow at the model extension boundary was 10.8 m^3/s.

The model indicates that surcharging occurs along the main drainage lines with concentrations near the corner of Plantation Road and Henry Street and additionally upstream of the Princess Hwy embankment formation. Notable widespread flooding also occurs between Bacchus Marsh Road and the large retarding basin 110-N-RB.

An inundation map of the downstream catchment areas is shown in Appendix F7.

9. ELCHO DRAIN SUBCATCHMENT C157J

9.1. Site Description

The Elcho Drain catchment is located in the eastern portion of the study area. The outlet of catchment is located at the culvert crossing of Forest Road South. The total area of the catchment included in the investigation is 19.8 km^2, of which the study area comprises around 40%. There are three separate waterways or drainage lines which discharge flows across the study area boundary at Bacchus Marsh Road. A map of the sub catchment is included as Figure 9-1.

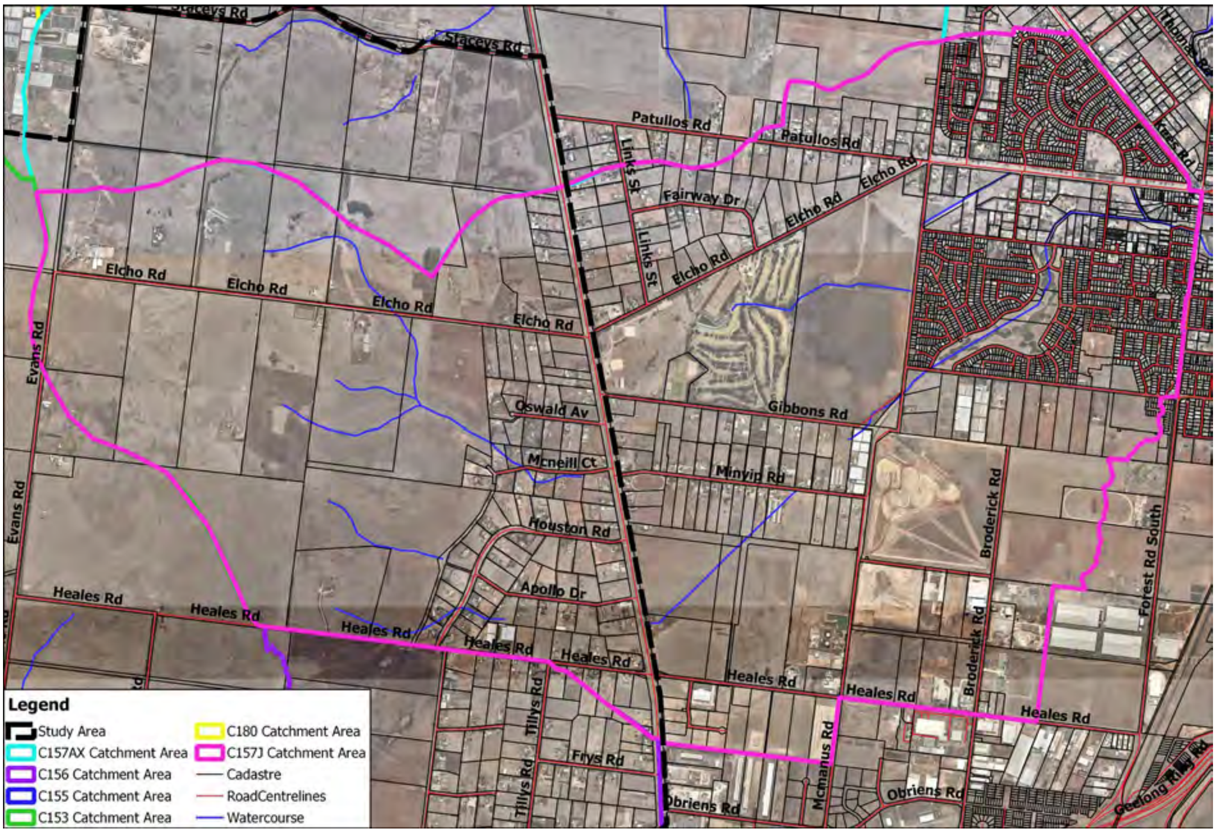

Figure 9-1: Elcho Drain Sub Area

There are three studies describing the investigation area being SKM (1999), SMEC (2010) and WBM (2006).

9.2. Hydrology

9.2.1. RORB Catchment Model

9.2.1.1. General

A RORB model has been developed for the Elcho Drain catchment. As described above, the model was calibrated by first generating the model without any impervious areas included in the catchment file. The model was calibrated to a rational method flow estimate and the impervious areas were subsequently placed into the model for the design runs.

For this catchment, a natural reach type was selected for the undeveloped conditions model and a mixture of natural and excavated, unlined waterway reach types were applied in the existing conditions model. The relative locations of different reach types are displayed in Figure 9-2.

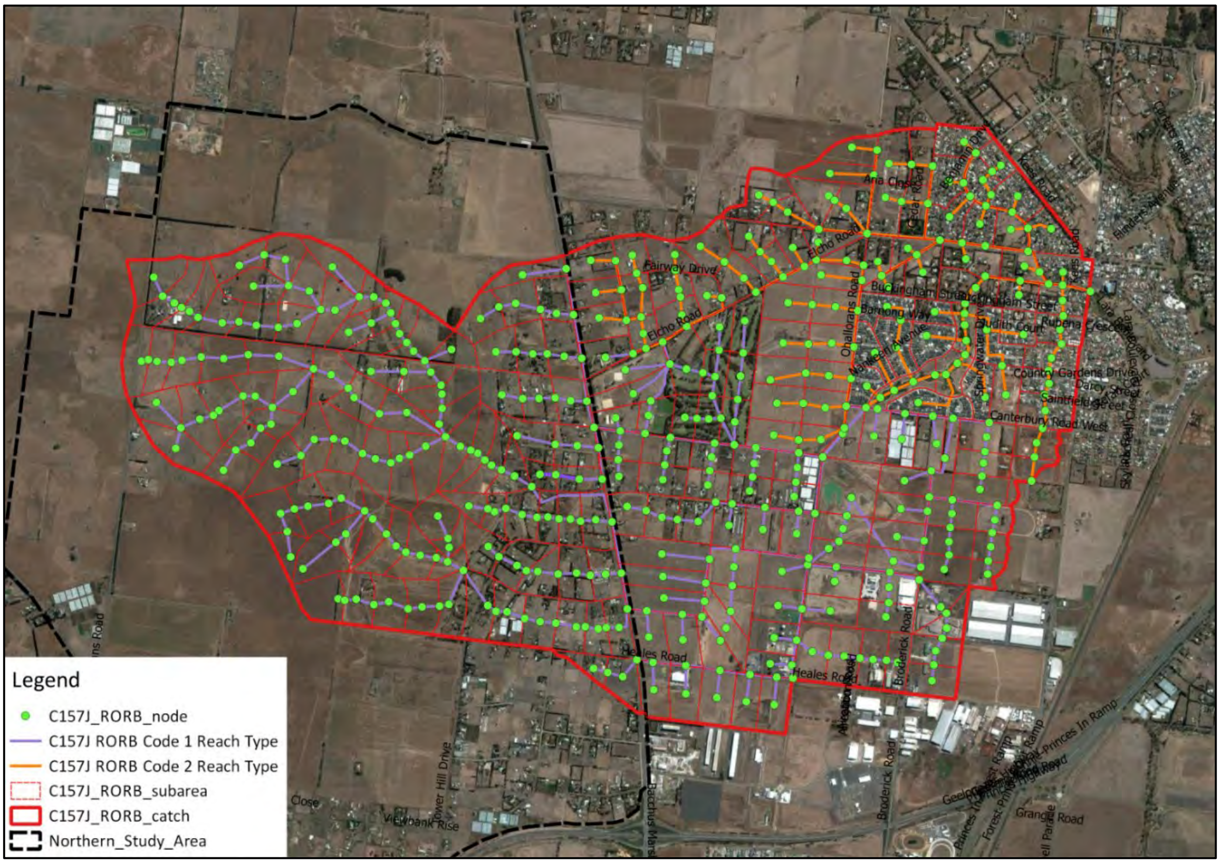

Figure 9-2: RORB Existing Conditions Reach type

9.2.1.2. Sub-Catchment Area

The Elcho Drain sub catchment model has an area of 19.8 km^2, which was delineated into 265 sub-catchment areas ranging in size between 6.5 and 17 ha. A schematic of the RORB model is included in Appendix G1.

The fraction impervious value for the undeveloped condition model is zero. The fraction impervious values applied to the existing conditions model were developed using the existing zonings. The basis upon which the RORB model impervious fractions were selected is included as a figure included in Appendix G2.

9.2.1.3. Delay Parameters

<u>K$_C$ Value</u>

The range of k_C values which are plausible for the catchment have been estimated using a variety of methods as detailed in Table 9-1.

Table 9-1 - Estimate k_C parameter equations

ESTIMATE	EQUATION	Kc
Dandenong Creek and Westernport Catchments	$k_c = 1.53 * A^{0.55}$ ▪ A = Area (km^2)	7.9
Yarra and Maribyrnong Catchments	$k_c = 1.19 * A^{0.56}$ ▪ Where A = Area (km^2)	6.33
Vic(MAR>800mm)	$k_c = 2.57 *A^{0.45}$ ▪ A = Area (km^2)	9.8
Vic(MAR<800mm)	$k_c = 0.49 *A^{0.65}$ ▪ A = Area (km^2)	3.4

The results from the analysis indicate the range of kc values that would be expected for the catchment are between 3.4 and 10.0.

m Value

An m value of 0.8 has been adopted as discussed above.

Initial Loss and Runoff Coefficient

Losses of 15 mm IL and 2 mm/hr CL as discussed above have been adopted for use and only varied if necessary to achieve an appropriate match with the target calibration flow.

9.2.2. Flow Estimation Comparison

The RORB model has been calibrated to the VicRoads rational method flows with a 1 in 100 AEP. The rainfall runoff process conceptualisation is different between RORB and the rational method. The result of these differences is that the modelled outputs are not consistent across different probability events, durations and catchment areas.

The approach taken in calibrating the RORB model has been to ensure to the extent possible that the 1 in 100 AEP outflows do not vary by more than 30% form the VicRoads rational method estimates. A range of locations within the catchment have been considered when undertaking the calibration. The details of the modelled outcomes are presented in Appendix G3 and the outcome are displayed graphically for both the 1 in 100 AEP and the 1 in 5 AEP event as Figure 9-3 and Figure 9-4.

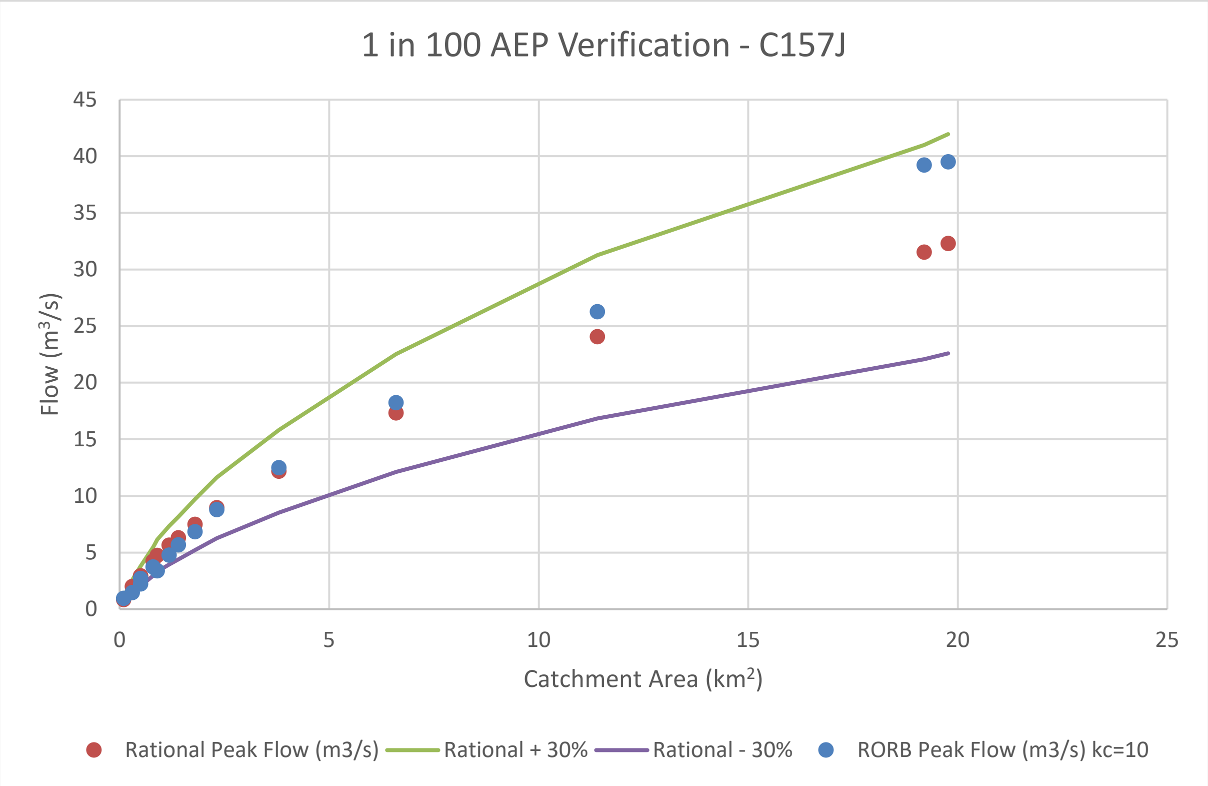

Figure 9-3: 1 in 100 AEP flow comparisons RORB vs VicRoads rational method

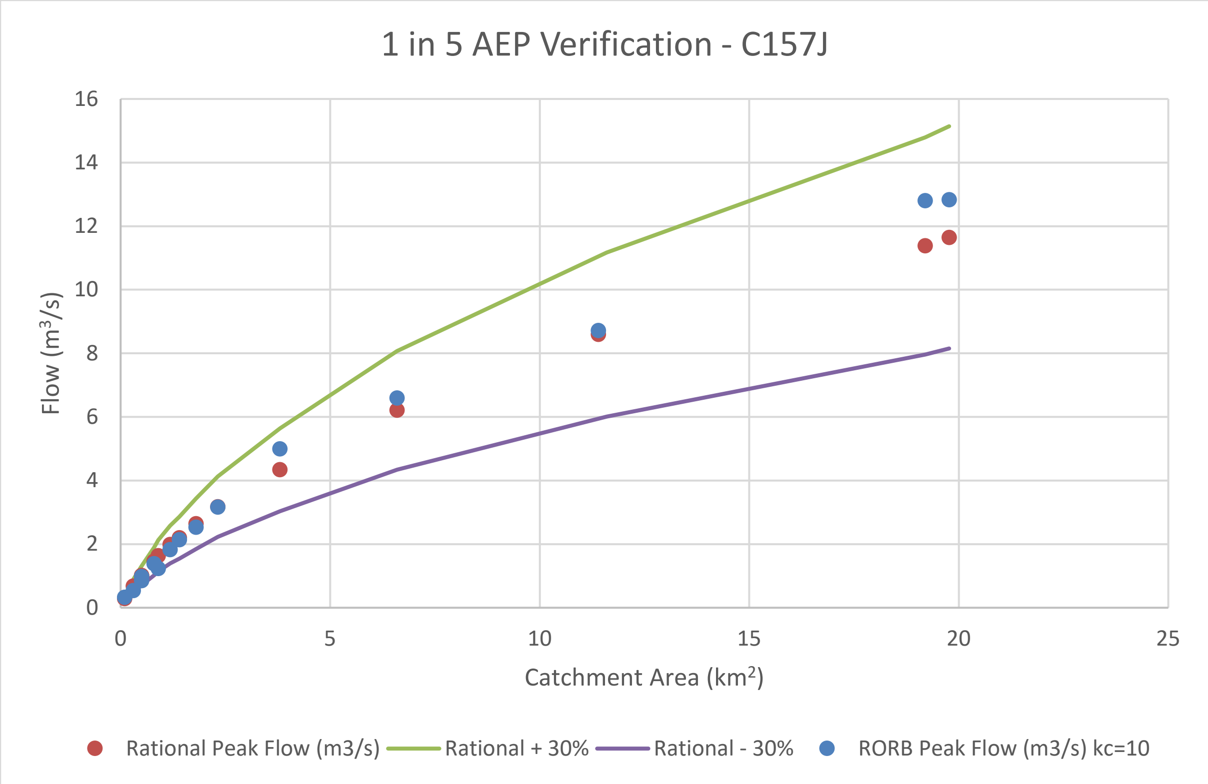

Figure 9-4: 1 in 5 AEP flow comparisons RORB vs VicRoads rational method

It may be observed from the above figure that the gradient of the runoff versus catchment area graph is different for the two modelling procedures. The RORB delay parameters is the predominate variable used for calibration and the parameter set detailed in Table 9-2 was found to offer the best compromise fit to the target flows.

Table 9-2 - Adopted RORB parameters

Parameter	Value
m	0.8
Kc	10.0
K_c/d_{av}	1.94
Initial Loss (mm)	15.0
Continuing Loss (mm/hr)	2.0

The K_c/d_{av} parameter is around 15% lower than the Cowies Creek figure of 2.26, indicating that this catchment has a similar runoff efficiency.

In addition to the VicRoads rational method and RORB modelled outcomes, both Tuflow and RFFE outcomes have been generated. The locations within the model considered to be of greatest relevance in the study are the overall catchment outlet and also the locations where waterways or flow paths cross the study area boundary. The flow at each of those locations has been reported upon comparatively in Table 9-3.

Table 9-3 – 1 in 100 AEP Flow estimation comparisons, undeveloped models

Location	Catchment Area (km^2)	RORB Critical Duration (hr)	Discharge (m^3/s)			
			VicRoads	RORB	RFFE	Tuflow
1 (Bacchus Marsh Rd)	1.18	2	5.6	4.7	2.7	8.8 (2hr)
2 (Bacchus Marsh Rd)	6.60	12	17.3	18.2	11.3	14.3 (9hr)
3 (Bacchus Marsh Rd)	0.10	1	0.8	0.9	NA	0.5 (9hr)
Outlet	19.8	12	32.3	39.5	32.3	28.5 (12hr)

The locations numbers referred to in the above table are detailed on the RORB sub area catchment map included as Appendix G1.

From the above table, it is observed that while the VicRoads rational method and RORB model outcomes are similar, there is substantial variability with the Tuflow modelled outcomes. Overall, the Tuflow and RORB model flows are not consistent across various sub areas.

The differences in discharges are due to differences as discussed for the Cowies Creek, St Georges and Wharf Road catchments. It is considered that Tuflow provides a superior approach to identifying contributing catchment areas at various locations. In the case of location 1 and 2, the difference could be attributed to the flow split occurring at McNeill Court. The difference in the flow path between Tuflow and RORB is shown in Figure 9-5.

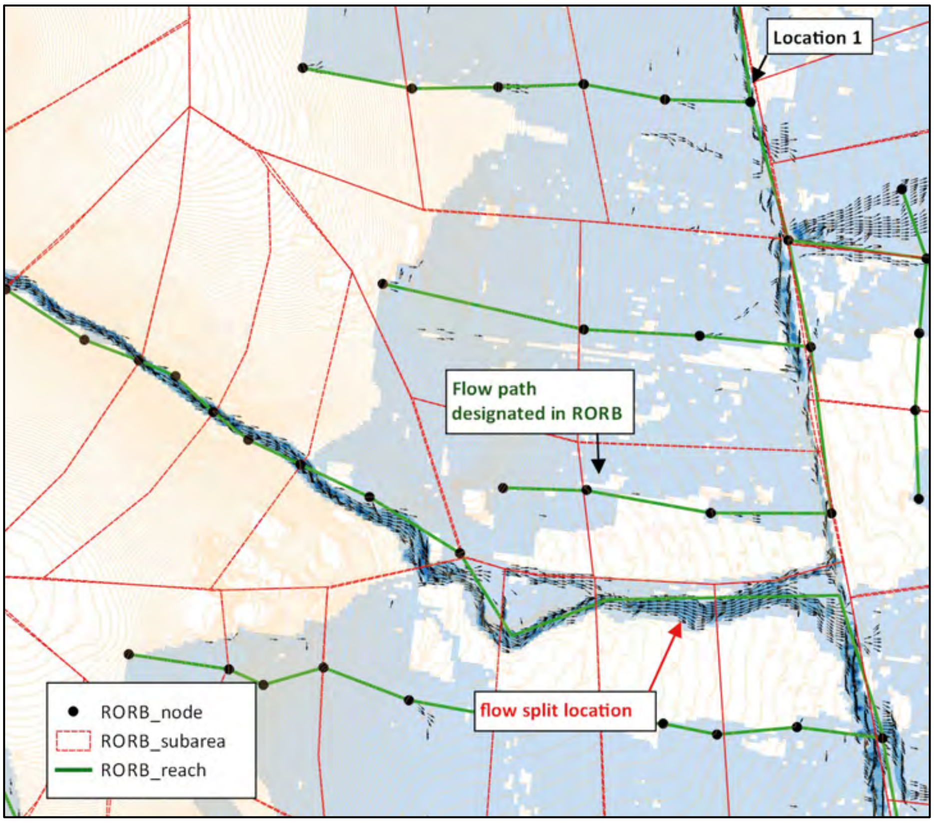

Figure 9-5: Flow path difference between Tuflow and RORB

The outputs from Tuflow show the water predominantly flowing to the south following the RORB reach alignment. It is also evident that a portion of the runoff is flowing towards the north. Contrary to Tuflow, the RORB model does not capture this flow split and the model carries the total flows towards south. This results in a lower Tuflow discharge at location 2 and higher discharge at location 1. As will be further discussed in the following section, Tuflow is able to account for the flow split and distribute flows based on the model topography.

Notwithstanding this observation, it is observed that overall, Tuflow indicates smaller peak runoffs at the catchment outlet.

9.3. Hydraulics

9.3.1. Model Development

Two separate Tuflow models have been developed as part of the existing conditions modelling, being an undeveloped and an existing conditions model. The main difference between the two models is the proportion imperviousness, which has been set to zero for undeveloped conditions. The proportion of imperviousness for existing conditions has been based upon the zonings for the various areas as detailed as a figure included as Appendix G2.

The model has been set up with a grid size of 10 m to determine the critical duration and 5 m for the final existing conditions run. The respective time steps for the models are 4 seconds and 2 seconds.

A range of 1 dimensional elements have been incorporated into the model as follows:

- 6 No. surveyed culverts under Bacchus Marsh Road (including culvert set near McNeill Ct).
- 2 No. surveyed culverts through the Grand Lakes development under Canterbury Road West and Buckingham Street.

A range of control lines in the form of 'thin z lines' have been incorporated into the surface layer to represent the flow split which occurs at Bacchus Marsh Road in the vicinity of McNeill Court. The locations of the control lines are shown on Figure 9-6.

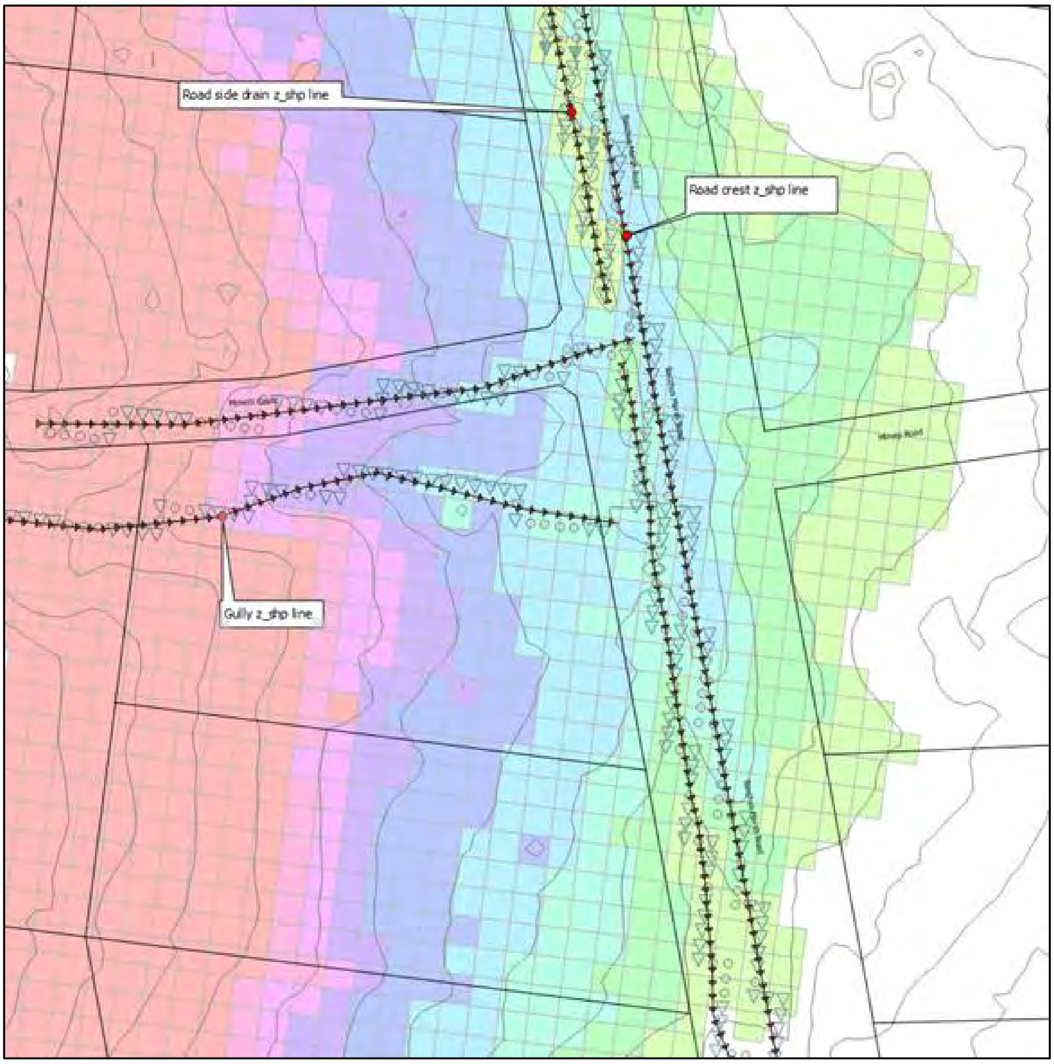

Figure 9-6: Location of Surface Control Lines

A thematic image of the DTM used in Tuflow modelling is included as Appendix G4. Manning's n roughness parameters applied across the model are presented as Appendix G5.

The RORB model results indicate that the critical duration storm for various sub areas vary between the 1 hour and 12 hours. The Tuflow model has been run for the 1 hour, 2 hour, 9 hour, 12 hour and 48 hour duration events.

The downstream model boundary has been developed using the following criteria:

- The modelled area has been extended a distance of around 200 m downstream of the investigation area defined in the Scope of Works.
- Downstream of the Forest Road South culvert, a weir/wetland embankment with a crest elevation of 6.5 mAHD controls water levels. This has been captured in the model.
- A hydraulic slope of 0.003 m/m has been adopted downstream of the weir/wetland embankment based upon the measured grade in Hovells Creek from the Hovells Creek sub catchment analysis.

9.3.2. Existing Conditions Model

The existing conditions model has been set up and run in both RORB and Tuflow models. The comparative outflows from the two models are presented in Table 9-4.

Table 9-4 – 1 in 100 AEP Existing Conditions Flow Comparisons

Location	Catchment Area (km^2)	RORB		Tuflow (Adopted)	
		Critical Duration (hr)	Discharge (m^3/s)	Critical Duration (hr)	Discharge (m^3/s)
1 (Bacchus Marsh Rd)	5.20	2	5.1	2	8.9
2 (Bacchus Marsh Rd)	2.60	9	18.2	9	14.6
3 (Bacchus Marsh Rd)	0.10	2	1.0	2	0.5
Outlet	19.8	12	42.4	12	29.2

As per the modelling methodology, the flows from the Tuflow model have been adopted for use in this study.

Of particular interest in this catchment is the flow split which occurs at Bacchus Marsh Road in the vicinity of McNeill Court. SKM (1999) reports that of the flows reaching this point, approximately 15% is diverted to the south, while 85% is directed north. The Tuflow modelling indicates that the flow split is 64% to the south and 36% diverted to the north during peak flow event. The modelled outcomes suggest that the flow split is different to that presented in SKM (1999). Inundation maps have been included as Appendix G6 detailing inundation depths.

It may be observed that the inundation map indicates some overland flows from Elcho Drain catchment are directed into the Hovells Creek catchment near Bacchus Marsh and O'Hallorans Road. A review of the modelled outcomes, indicates the flows amount to less than 0.3 m^3/s. This flow is judged to have a negligible impact on the project outcomes and has not been considered further as part of the study.

It should be highlighted that this study relies on topographic data from 2011 and 2014. It is likely that terrain changes in the Elcho Drain catchment may have occurred since that time. The impacts of any such changes are not represented in the modelling associated with this report.

9.4. Sensitivity Assessment

The impact of varying various hydraulic model parameters was tested as part of the modelling. A variety of alternative runs which were undertaken trialling different grid sizes and roughness parameters and the results are presented in Table 9-5.

Table 9-5 – Tuflow Outflow Sensitivity Assessment

Model Conditions	Manning's n		Grid Size (m)	Duration (hr)	Discharge 150 m D/S of Forest Road South (m^3/s)
	Waterway	Floodplain			
Undeveloped	0.04	0.04	10	48	45.0
Undeveloped	0.04	0.04	10	12	45.0
Undeveloped	0.04	0.07	10	12	26.9
Existing	0.04	0.04	10	12	46.2
Existing	0.04	0.07	10	12	31.3
Existing	0.04	0.07	5	12	30.7
Existing	varies (as for Cowies Creek)		5	12	29.2

The outcomes presented above indicate that the peak outflows are sensitive to the Manning's n values and to a greater extent than is the case for other sub catchments in this study. In adopting a larger Manning's n value in the floodplain regions, the flow estimate provides a closer estimate to the target VicRoads rational method estimate.

It may be noted that some of the runs above have outflows above 40 m^3/s. Those runs represent modelled outcomes prior to the Geelong Bacchus Marsh Road being defined by a control line running along the alignment of the road (a thin Z line). The inclusion of that feature resulted in a different spilt of flows occurring in the vicinity of the Geelong Bacchus Marsh Road/McNeil Court intersection.

9.5. Other Studies

There are three existing reports which have investigated the Elcho Drain catchment, being SKM (1999), SMEC (2010) and WBM (2006). Of the three reports, SKM (1999) is considered to be of greatest relevance since both SMEC (2010) and WBM (2006) have adopted flows from SKM (1999). SMEC (2010) reports on some flows which are not included in the SKM (1999) report. WBM (2006) while using flows from the SKM (1999) analysis, does not report on any flow magnitudes.

SKM (1999) was informed by a hydrology report which is understood to have been undertaken by SKM in 1998. The hydrology report was not available for review as part of this study. There are two points for which flow estimates are available, being the downstream boundary at Forest Road South and Elcho Drain at Bacchus Marsh Road.

The hydrology analysis as referred to in SKM (1999) was modelled using the RAFTS software package. There is no information available on the model calibration process. The outputs from the RAFTS model were input to a quasi 2 dimensional MIKE-11 hydraulic model to determine the attenuation through the area downstream of Bacchus Marsh Road. The key outcomes relative to this study are presented in Table 9-6.

Table 9-6 – 1 in 100 AEP Flow Existing Conditions Study Comparison

Location	Model	SKM (1999)		This Study	
		Critical Duration (hr)	Discharge (m^3/s)	Critical Duration (hr)	Discharge (m^3/s)
2 Elcho Drain (Bacchus Marsh Rd)	RAFTS/RORB	3	10.2	9	18.2
Outlet (Forest Road)	RAFTS/RORB	9	61	12	42.1
Outlet (Forest Road)	MIKE-11/Tuflow	12	39	12	29.2

From the above table, it is observed that the comparative peak flows at the Bacchus Marsh Road study boundary are notably different. The flows are also substantially different at the Forest Road South model boundary. It is unclear why there is a difference between the two model runs. It may be due to a different reach type and fraction pervious in the SKM (1999) model or perhaps it is due to a coarser sub catchment pattern with a shorter relative waterway length per unit catchment area.

10. HOVELLS CREEK SUBCATCHMENT C157AX

10.1. Site Description

The Hovells Creek catchment is located in the north eastern portion of the study area. The catchment outlet is located at the proposed culvert crossing at O'Hallorans Road. The total area of the catchment included in the investigation is 9.3 km^2, of which the study area comprises around 40%. There are two separate waterways or drainage lines which discharge flows across the study area boundary at Staceys Road and Bacchus Marsh Road. A map of the sub catchment is included as Figure 10-1.

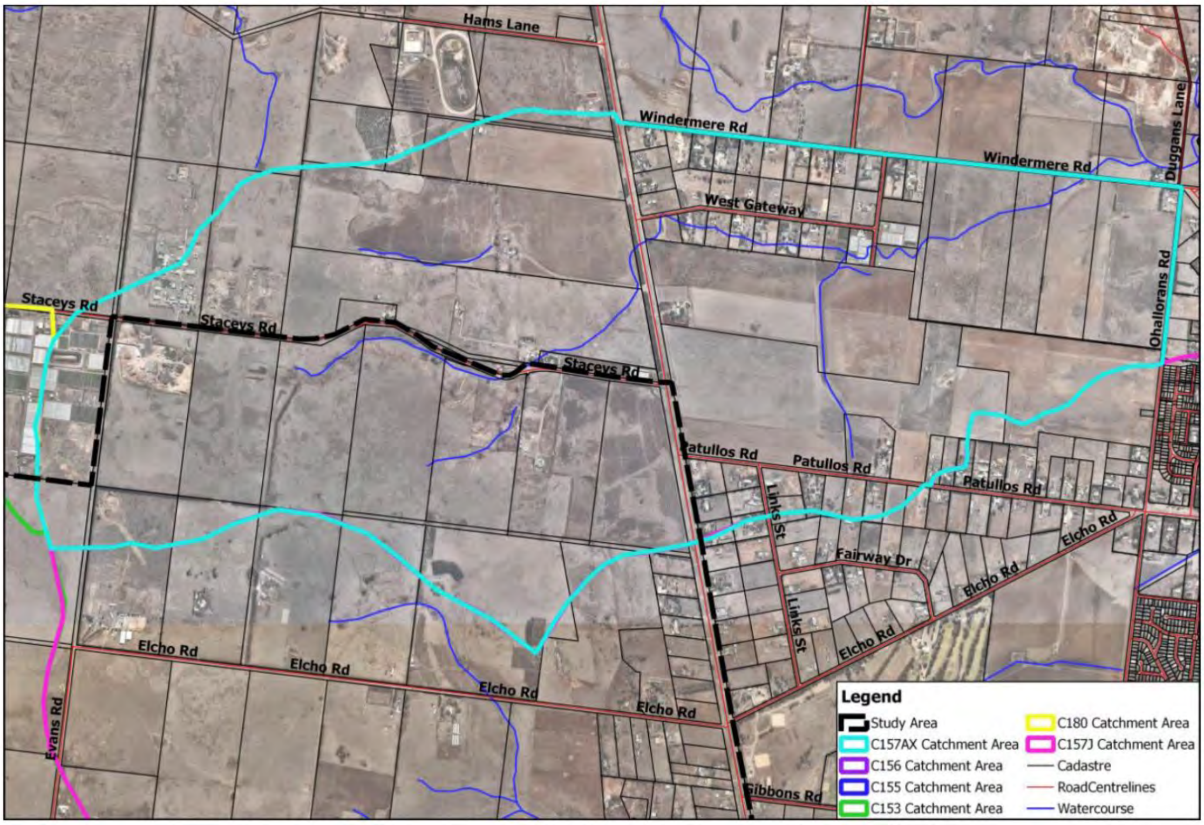

Figure 10-1: Hovells Creek Sub Area

There are three studies describing the investigation area being Craigie (2012), Spiire (2014) and WBM Oceanics (2002).

10.2. Hydrology

10.2.1. RORB Catchment Model

10.2.1.1. General

A RORB model has been developed for the Hovells Creek catchment. As described above, the model was calibrated by first generating the model without any impervious areas included in the catchment file. The model was calibrated to a rational method flow estimate and the impervious areas were subsequently placed into the model for the design runs.

For this catchment, a natural reach type was selected for the undeveloped conditions model and a mixture of natural and excavated, unlined waterway reach types were applied in the existing conditions model. The relative locations of different reach types are displayed in Figure 10-2.

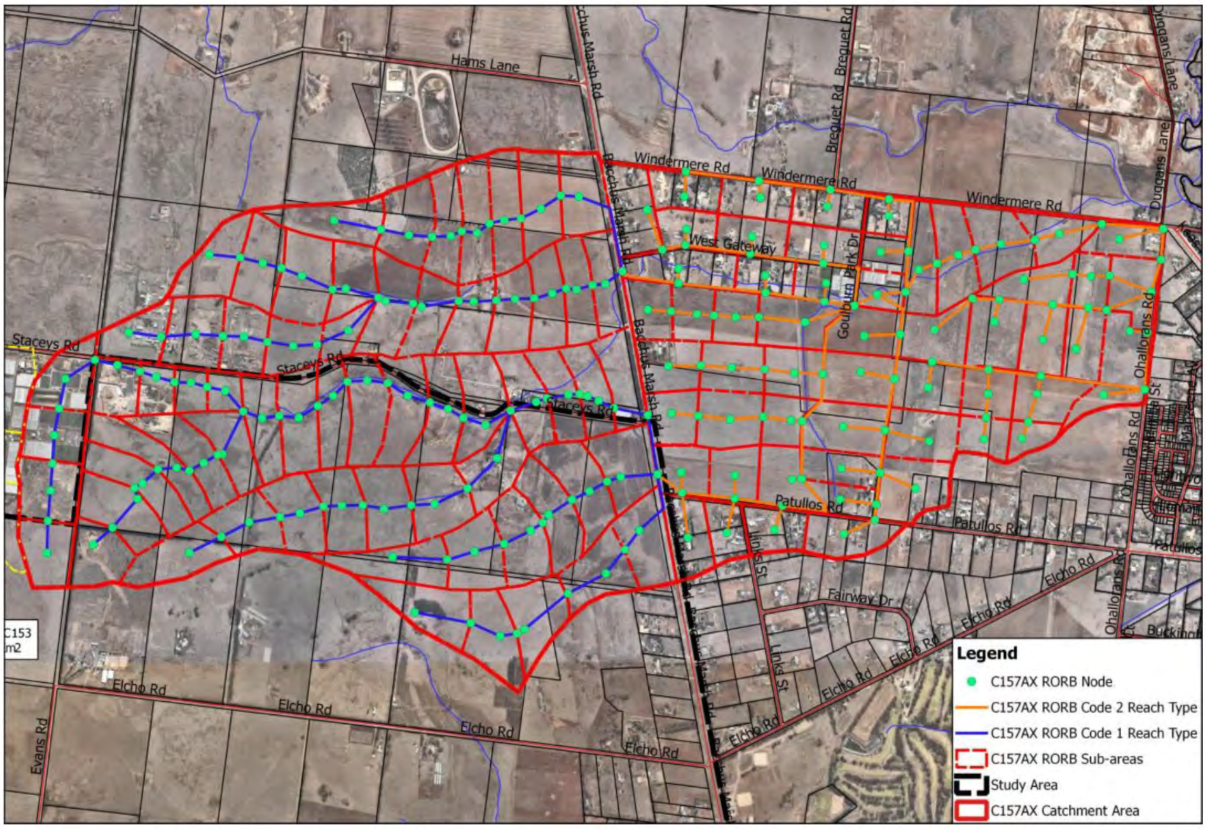

Figure 10-2: RORB Existing Conditions Reach type

10.2.1.2. Sub-Catchment Area

The Hovells Creek sub catchment model has an area of 9.3 km^2, which was delineated into 122 sub-catchment areas ranging in size between 3.2 and 10.8 ha. A schematic of the RORB model is included in Appendix H1.

The fraction impervious value for the undeveloped condition model is zero. The fraction impervious values applied to the existing conditions model were developed using the existing zonings. The basis upon which the RORB model impervious fractions were selected is included as a figure included in Appendix H2.

10.2.1.3. Delay Parameters

<u>K_C Value</u>

The range of k_C values which are plausible for the catchment have been estimated using a variety of methods as detailed in Table 10-1.

Table 10-1 - Estimate k_C parameter equations

ESTIMATE	EQUATION	Kc
Dandenong Creek and Westernport Catchments	$k_c = 1.53 * A^{0.55}$ ▪ A = Area (km^2)	5.22
Yarra and Maribyrnong Catchments	$k_c = 1.19 * A^{0.56}$ ▪ Where A = Area (km^2)	4.15
Vic(MAR>800mm)	$k_c = 2.57 * A^{0.45}$ ▪ A = Area (km^2)	7.0
Vic(MAR<800mm)	$k_c = 0.49 * A^{0.65}$ ▪ A = Area (km^2)	2.1

The results from the analysis indicate the range of kc values that would be expected for the catchment are between 2.1 and 7.0.

m Value

An m value of 0.8 has been adopted as discussed above.

Initial Loss and Runoff Coefficient

Losses of 15 mm IL and 2 mm/hr CL as discussed above have been adopted for use and only varied if necessary to achieve an appropriate match with the target calibration flow.

10.2.2. Flow Estimation Comparison

The RORB model has been calibrated to the VicRoads rational method flows with a 1 in 100 AEP. The rainfall runoff process conceptualisation is different between RORB and the rational method. The result of these differences is that the modelled outputs are not consistent across different probability events, durations and catchment areas.

The approach taken in calibrating the RORB model has been to ensure to the extent possible that the 1 in 100 AEP outflows do not vary by more than 30% form the VicRoads rational method estimates. A range of locations within the catchment have been considered when undertaking the calibration. The details of the modelled outcomes are presented in Appendix H3 and the outcome are displayed graphically for both the 1 in 100 AEP and the 1 in 5 AEP event as Figure 10-3 and Figure 10-4.

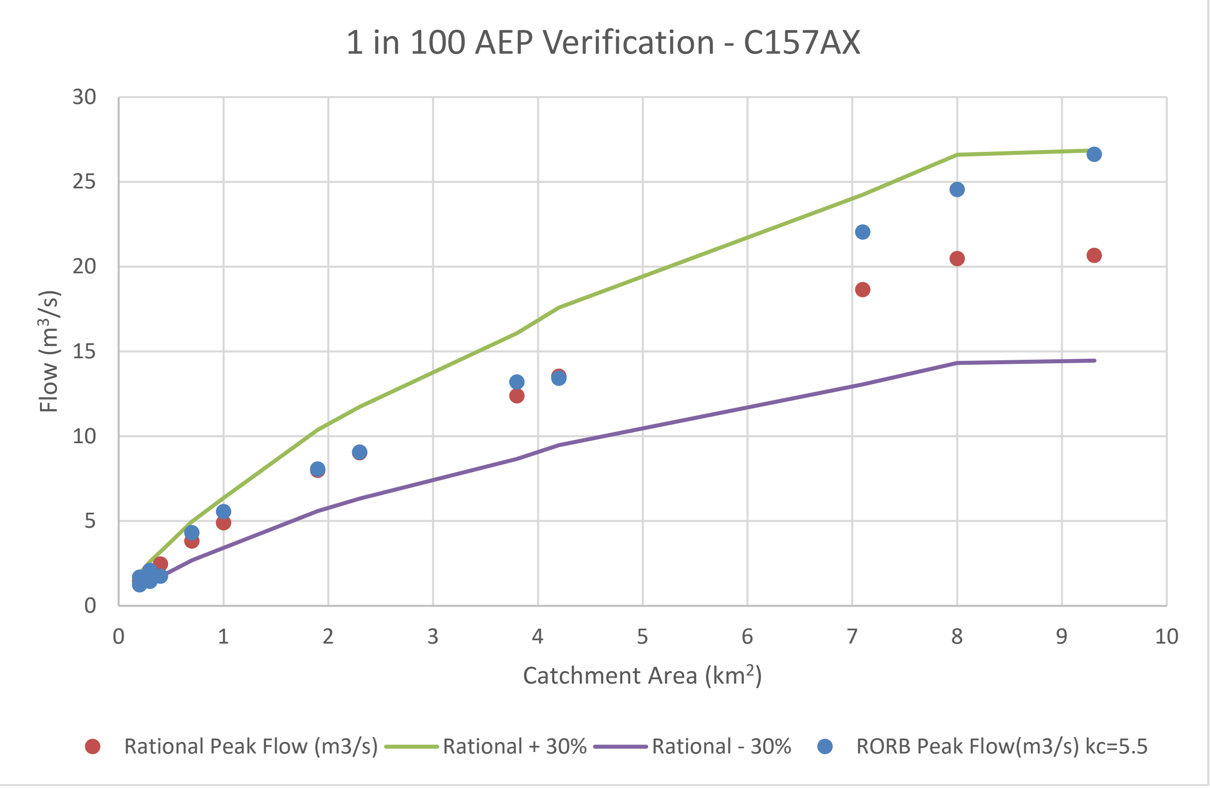

Figure 10-3: 1 in 100 AEP flow comparisons RORB vs VicRoads rational method

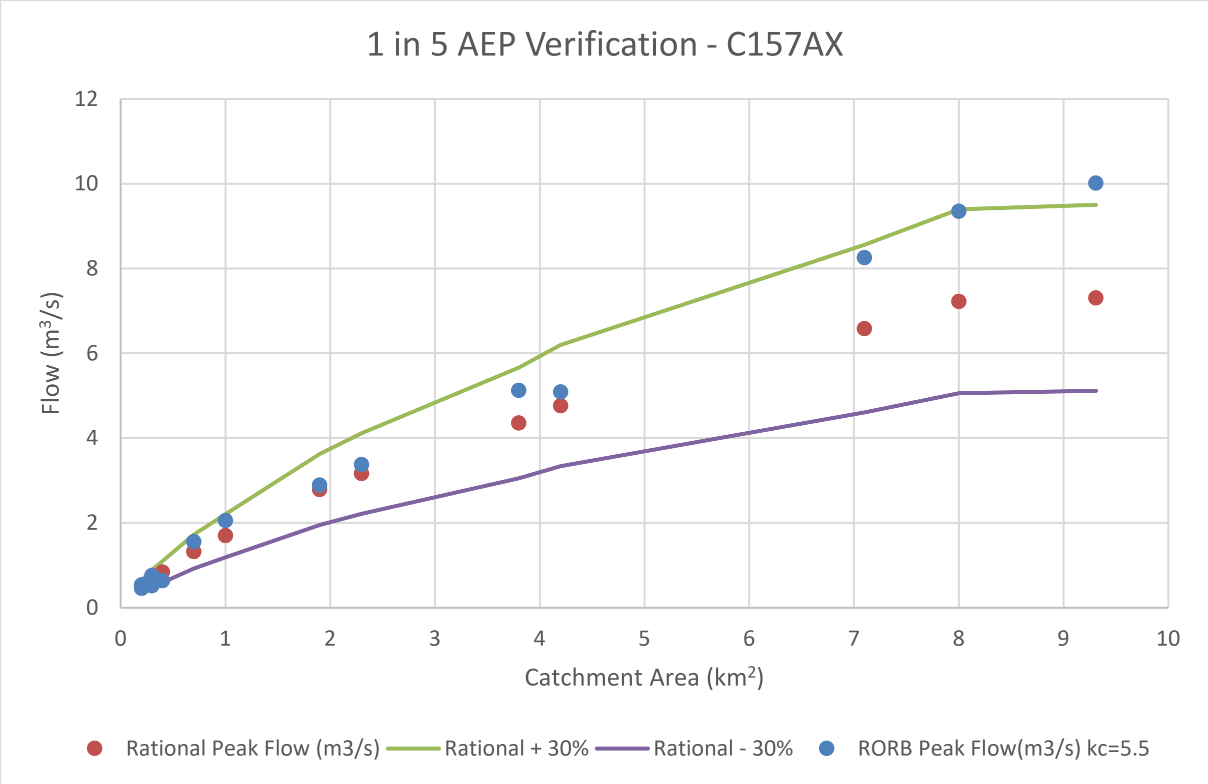

Figure 10-4: 1 in 5 AEP flow comparisons RORB vs VicRoads rational method

It may be observed from the above figure that the gradient of the runoff versus catchment area graph is different for the two modelling procedures. The RORB delay parameters is the predominate variable used for calibration and the parameter set detailed in Table 10-2 was found to offer the best compromise fit to the target flows.

Table 10-2 - Adopted RORB parameters

Parameter	Value
m	0.8
Kc	5.5
K_c/d_{av}	1.46
Initial Loss (mm)	15.0
Continuing Loss (mm/hr)	2.0

The K_c/d_{av} parameter is around 35% lower than the Cowies Creek figure of 2.26, indicating that the Hovells Creek catchment is more efficient.

In addition to the VicRoads rational method and RORB modelled outcomes, both Tuflow and RFFE outcomes have been generated. The locations within the model considered to be of greatest relevance in the study are the overall catchment outlet and also the locations where waterways or flow paths cross the study area boundary. The flow at each of those locations has been reported upon comparatively in Table 10-3.

Table 10-3 – 1 in 100 AEP Flow estimation comparisons, undeveloped models

Location	Catchment Area (km^2)	RORB Critical Duration (hr)	Discharge (m^3/s)			
			VicRoads	RORB	RFFE	Tuflow
1 (Bacchus Marsh Rd)	3.8	12	12.4	13.2	6.9	11.6 (2hr)
2 (Staceys Rd)	2.3	12	9.0	9.1	NA	11.2 (2hr)
Outlet	9.3	12	20.7	26.6	14.5	19.9 (12hr)

The locations numbers referred to in the above table are detailed on the RORB sub area catchment map included as Appendix H1.

10.3. Hydraulics

10.3.1. Model Development

Two separate Tuflow models have been developed as part of the existing conditions modelling, being an undeveloped and an existing conditions model. The main difference between the two models is the proportion of imperviousness, which has been set to zero for undeveloped conditions. The proportion of imperviousness for existing conditions has been based upon the zonings for the various areas as detailed as a figure included as Appendix H2.

The model has been set up with a grid size of 10 m to determine the critical duration and 5 m for the final existing conditions run. The respective time steps for the models are 4 seconds and 2 seconds. A range of 1 dimensional elements have been incorporated into the model as follows:

- 2 No. surveyed culverts under Bacchus Marsh Road
- 1 no. Culvert under O'Hallorans Road

A channel was included in the model, approximating the shape as described in Craigie (2012) for the proposed Lara West development, inclusive of retarding basins. A channel was also modelled from the Manzeene Village development east of O'Hallorans Road. Furthermore, road reserves as presented in Craigie (2012) were lowered by 150mm in the hydraulic model.

A thematic image of the DTM used in Tuflow modelling is included as Appendix H4. Manning's n roughness parameters applied across the model are presented as Appendix H5.

The RORB model results indicate that the critical duration storm for various sub areas is consistent at 12 hours. The Tuflow model has been run for the 2 hour, 6 hour, 9 hour, 12 hour and 48 hour duration events.

The downstream model boundary has been developed as follows:

- The modelled area has been extended a distance of around 700 m downstream of the investigation area defined in the Scope of Works.
- A downstream hydraulic slope of 0.001 m/m has been adopted based upon the measured grade of the Hovells Creek waterway.

Information available from other studies is as follows:

- Modelling described in Craigie (2012) indicates that at the O'Hallorans Road culvert, a flow 40 m^3/s corresponds to a flood depth of 14 mAHD.
- The same report indicates that at the Bacchus Marsh Road southern culvert, a flow 14.6 m^3/s corresponds to a flood depth of 26.2 mAHD.

10.3.2. Existing Conditions Model

The existing conditions model has been set up and run in both RORB and Tuflow models. The comparative outflows from the two models are presented in Table 10-4.

Table 10-4 – 1 in 100 AEP Existing Conditions Flow Comparisons

Location	Catchment Area (km^2)	RORB		Tuflow (Adopted)	
		Critical Duration (hr)	Discharge (m^3/s)	Critical Duration (hr)	Discharge (m^3/s)
1 (Bacchus Marsh Rd)	3.8	9	13.0	2	12.9
2 (Staceys Rd)	2.3	3	9.2	2	11.7
Outlet	9.3	2	35.6	12	21.3

As per the modelling methodology, the flows from the Tuflow model have been adopted for use in this study.

Inundation maps have been included as Appendix H6 detailing inundation depths.

10.4. Sensitivity Assessment

The impact of varying various hydraulic model parameters was tested as part of the modelling. A variety of alternative runs which were undertaken trialling different grid sizes and roughness parameters and the results are presented in Table 10-5.

Table 10-5 – Tuflow Outflow Sensitivity Assessment

Model Conditions	Manning's n		Grid Size (m)	Duration (hr)	Discharge (m^3/s)
	Waterway	Floodplain			
Undeveloped	0.04	0.04	10	2	42.3
Undeveloped	0.04	0.07	10	2	22.7
Existing	0.04	0.04	10	2	44.3
Existing	0.04	0.07	10	2	21.9
Existing	0.04	0.07	5	2	22.5
Undeveloped	0.04	0.07	10	9	16.0
Existing	0.04	0.07	10	9	16.9
Existing	0.04	0.07	5	9	17.3
Existing	varies (as for Cowies Creek)		5	9	15.0

The sensitivity analysis indicates that the results are sensitive predominately to Manning's n values and that varying the grid size has a minimal effect on estimated flows.

10.5. Other Studies

There are three existing reports which have investigated the Hovells Creek catchment, being Craigie (2012), Spiire (2014) and WBM Oceanics (2002). Of the three reports, WBM Oceanics (2002) is considered to be of greatest relevance since both Craigie (2012) and Spiire (2014) have adopted the flood modelling approach from that study. Spiire (2014) has not been considered further as part of this review as there are no flows documented in that report.

The hydrologic analysis described in WBM Oceanics (2002) includes development of a RORB model. It is reported that the RORB model was calibrated to the rational method. It remains unstated which version of the rational method, however, it is judged likely the VicRoads version was applied. Differences in model parameters between WBM Oceanics (2002) and this study are as follows:

- The kc delay parameter was set at a value of 6.5 which is higher than the figure used for this study.
- A storage exponent (m) of 0.7 was applied in comparison to a value of 0.8 for this study.
- An initial loss of 20 mm was used as compared to a figure of 15 mm for this study.
- A Proportional Loss (PL) of 0.05 was applied. This study has applied a continuing loss rather than a proportional loss.

Craigie (2012) reports in greatest detail on the modelled flows. Further, Craigie incorporates additional inflows to the catchment which are estimated to cross the northern catchment boundary in large flood events. It is understood that the additional inflow hydrograph from the northern catchment was provided to the Craigie (2012) analysis by BMTWBM via CoGG, however the details of the hydrograph were never documented in a report.

Only a few of the various modelling reports of the Hovells Creek catchment, are directly comparable with the analysis we have completed. A range are presented below, noting that all reported flows are for undeveloped conditions.

10.5.1. Comparison 1

The simplest direct comparison is between the RORB outcomes from WBM Oceanics (2002) and the undeveloped conditions outcomes reported in this study. The relevant flows and RORB characteristics are detailed in Table 10-6:

Table 10-6 – Comparative Characteristics No.1

Characteristic (1 in 100 AEP event)	WBM Oceanics (2002)	SMEC (2018)
Peak RORB Outflow (m^3/s)	30	26.6
RORB k_c parameter	6.5	5.5
RORB m parameter	0.7	0.8
RORB IL Parameter (mm)	20	15
RORB CL Parameter (mm/hr)	0.5	2

The runoff from the two models is similar. The continuing loss in the SMEC model is greater than that in WBM Oceanics (2002), which leads to some reduction in outflows. Further, the difference in the m parameter leads to some variation in peak flow. A conversion factor described in Laurenson et.al. (2010) can be applied to establish the equivalent kc for a given peak flow and m value as follows:

Conversion factor = $(Q_p/2)^{m-m'}$

where

Q_p = peak flow

m = existing value

m'= new value.

The equivalent RORB K_c parameter required to achieve a peak outflow of 30m^3/s with an m value of 0.8 is 5.0. In applying the RORB governing equation ($S = K_c \times Q^m$), the increase of Kc from 5 to 5.5 in SMEC (2018) accounts for the difference between the two models.

10.5.2. Comparison 2

A less direct comparison involves reported outputs from Craigie (2012) and WBM Oceanics (2002). Craigie (2012) amends parameters from WBM Oceanics and also introduces an inflow from a catchment to the north. The relevant outcomes are presented in Table 10-7.

Table 10-7 – Comparative Characteristics No.2

Characteristic (1 in 100 AEP event)	WBM Oceanics (2002)	SMEC (2018)	Craigie (2012)
Bacchus Marsh Rd RORB Flow (m^3/s)	NA	13.2	14.6
Peak RORB Outflow (m^3/s)	30	26.6	40
RORB k_c parameter	6.5	5.5	6.5
RORB m parameter	0.7	0.8	0.7
RORB IL Parameter (mm)	20	15	20
RORB CL Parameter (mm/hr)	0.5	2	NA
RORB RoC Parameter	NA	NA	0.95

The outcomes at Bacchus Marsh Road suggest that differences between the models in terms of delay and loss parameters is around 10% which accounts for a difference of around 4 m^3/s at the catchment outlet. This suggests that the additional flow from the northern catchment accounts for the remaining difference, being 9.4 m^3/s.

10.5.3. Comparison 3

The least direct comparison is that between Tuflow model outcomes from modelling undertaken by WBM Oceanics in 2012 and reported on indirectly in Craigie (2012). The relevant outcomes are presented in Table 10-8.

Table 10-8 – Comparative Characteristics No.3

Characteristic (1 in 100 AEP event)	WBM Oceanics 2012 as presented in Craigie (2012)	SMEC (2018)
Bacchus Marsh Rd Tuflow Flow (m^3/s)	14.4	11.6
Peak Tuflow Outflow (m^3/s)	39.2	19.9

The outcomes at Bacchus Marsh Road vary between the two models, however, the inputs at that location are estimated using distinctly different processes. The Tuflow model domain developed by WBM Oceanics covers the region downstream of Bacchus Marsh Road only with inflows provided from a RORB model, while the SMEC (2018) model includes the entire upstream catchment. The hydrograph at Bacchus Marsh Road is generated within the Tuflow model in SMEC (2018) and would be different in a range of respects to that used in the WBM Oceanics model. The peak outflow varies between the two models, with the outflow from SMEC (2018) being around half of that generated in WBM Oceanics. The additional inflow from the north which is estimated to be around 9.6 m^3/s in Comparison no.2 would increase flows reported in SMEC (2018) to 29.5 m^3/s. Ultimately, the outcomes from the two models are different with the SMEC outcomes being around 75% of the WBM Oceanics outcomes from 2012.

The comparisons presented above indicate the following:

- The RORB model parameters are different between the SMEC (2018) and the WBM Oceanics (2002)/Craigie (2012) analyses. The SMEC (2018) model produces lower flows which are predominately due to adoption of a slightly higher delay parameter (equivalent K_c of 5.0 in WBM Oceanics (2002) vs k_c of 5.5 in SMEC (2018)).
- The comparison of RORB model outcomes in the above point is independent of additional inflows from the catchment to the north which are estimated to add around 10 m^3/s to the peak catchment flow.
- The Tuflow modelled outcomes are different between the WBM Oceanics model and the SMEC (2018) model with the equivalent modelled outflows being lower in SMEC (2018). It is not considered that a straightforward comparison between the two models is possible due to the different schematisations. In particular, the outflows from the WBM Oceanics model are generated from a composite RORB model and Tuflow model routing process over the catchment. By comparison, the outflows from the model described in SMEC (2018) are generated from a Tuflow model of the entire catchment.

In addition to the above, the sensitivity analysis demonstrates that a wide range of outcomes are possible from the Tuflow model by varying the Manning's n parameter across the catchment. The Manning's n values applied in the modelling have been adopted in consultation with CoGG with the aim of providing consistency in parameter values across the various modelled sub-catchments in the NGGA.

Without offering any comment on the absolute accuracy of the current or earlier flood estimates, it is considered that the approach in this study is more consistent across all catchments than the amalgam of previous studies for the investigation area. Further, it is considered to offer a reasonable basis, consistent with current standards on which to size flood related infrastructure.

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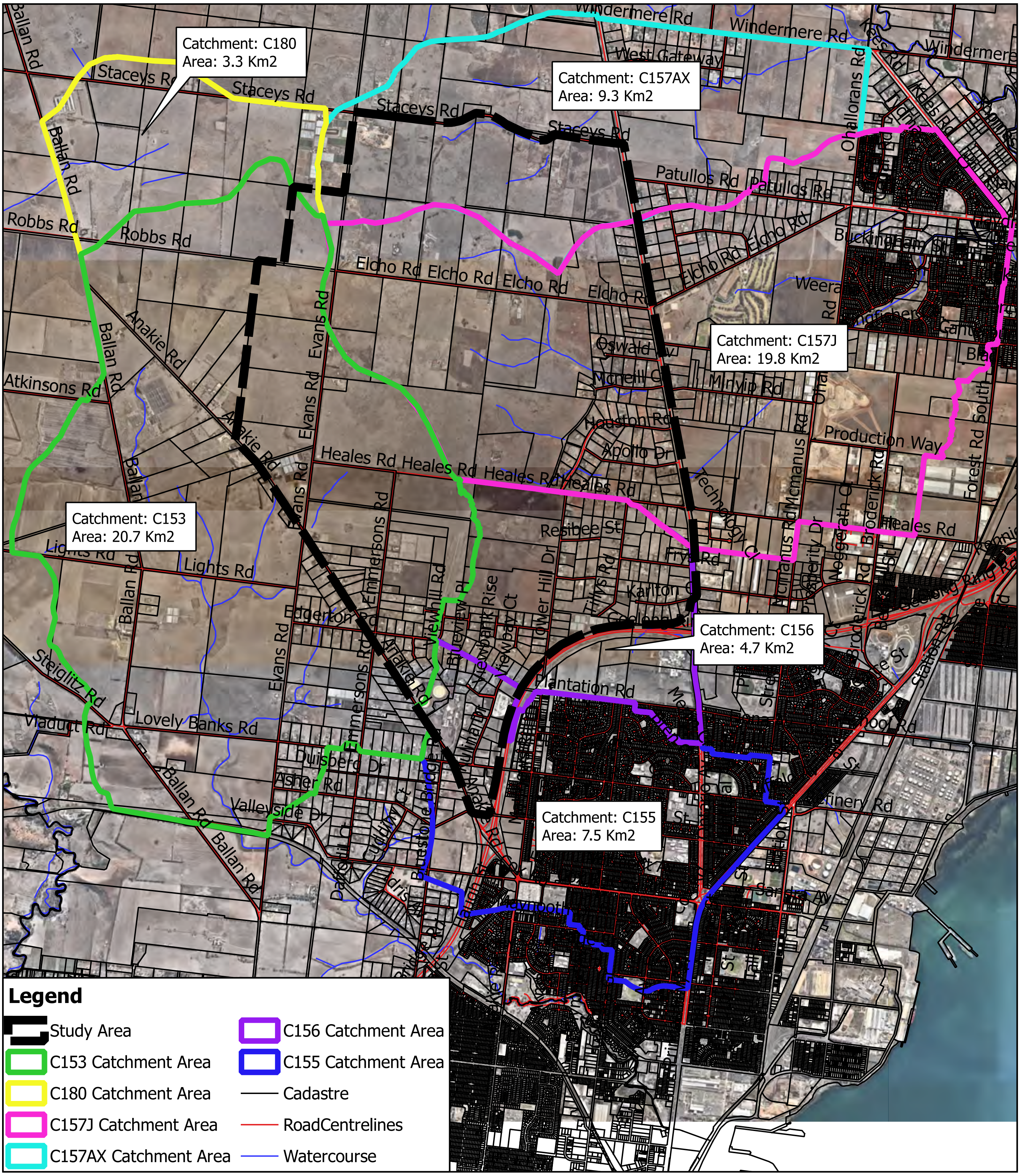
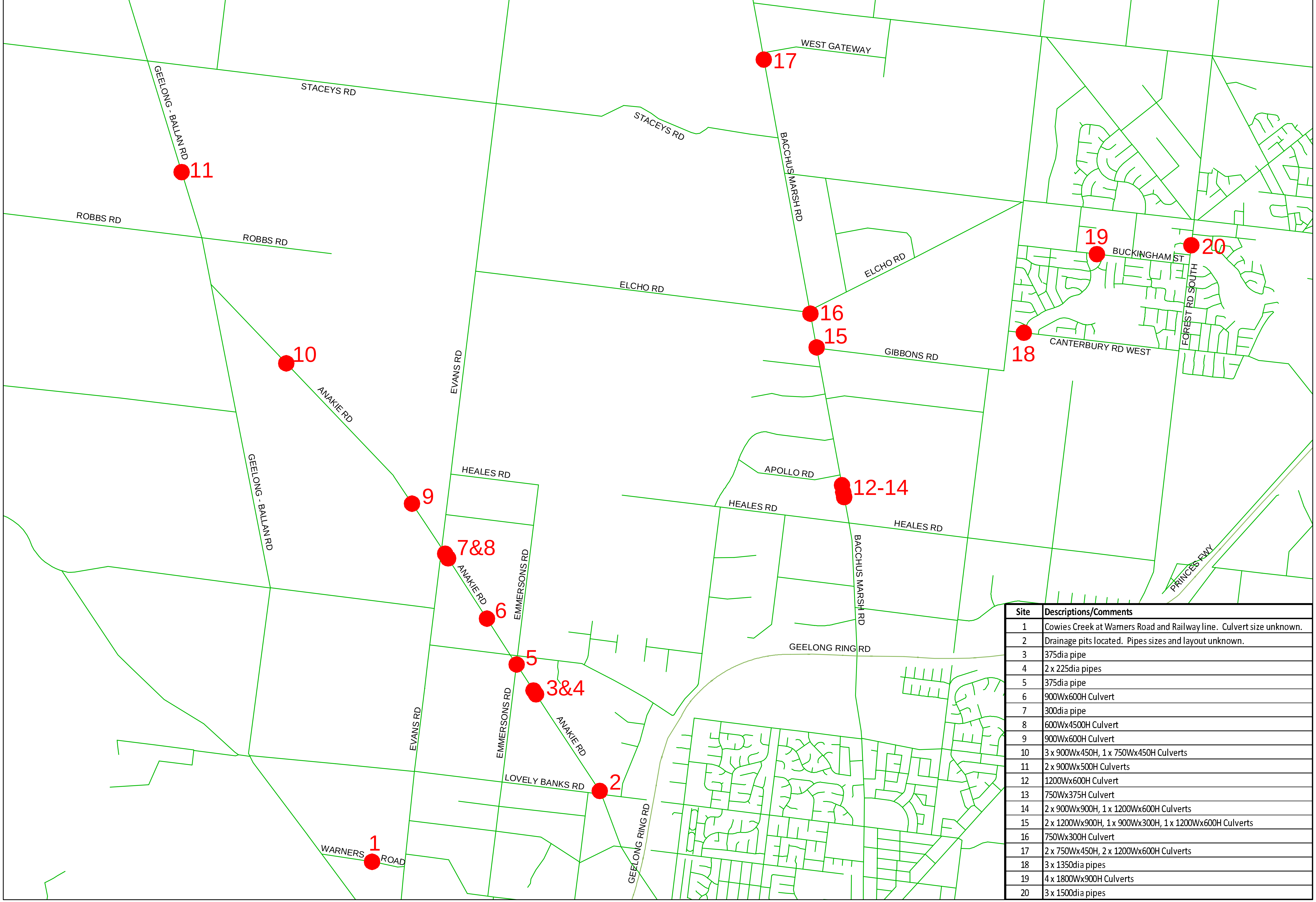
APPENDIX A INVESTIGATION AREA

APPENDIX A - INVESTIGATION AREA



APPENDIX B SURVEYED CULVERT LOCATIONS

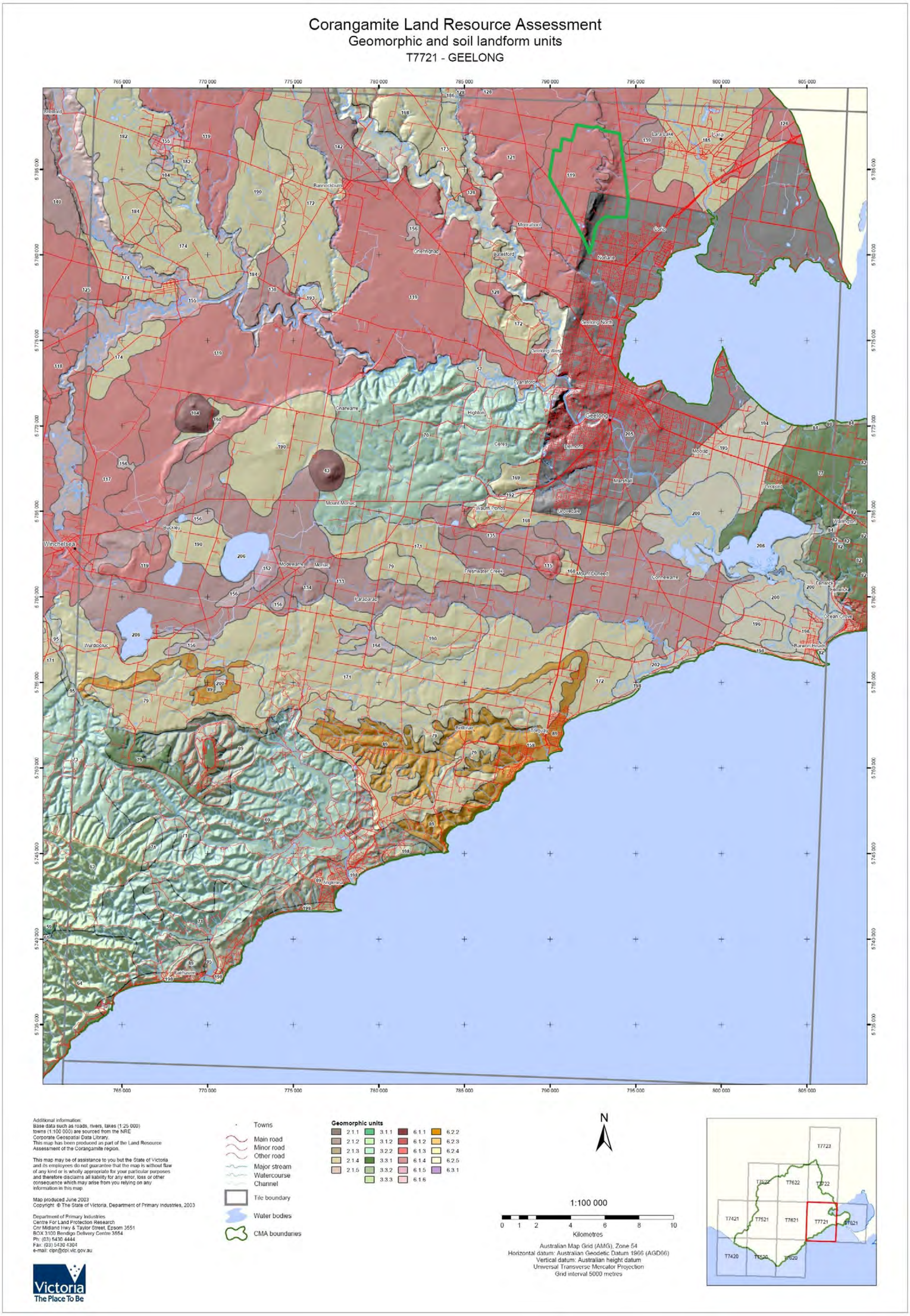
APPENDIX B - SURVEYED CULVERT LOCATIONS



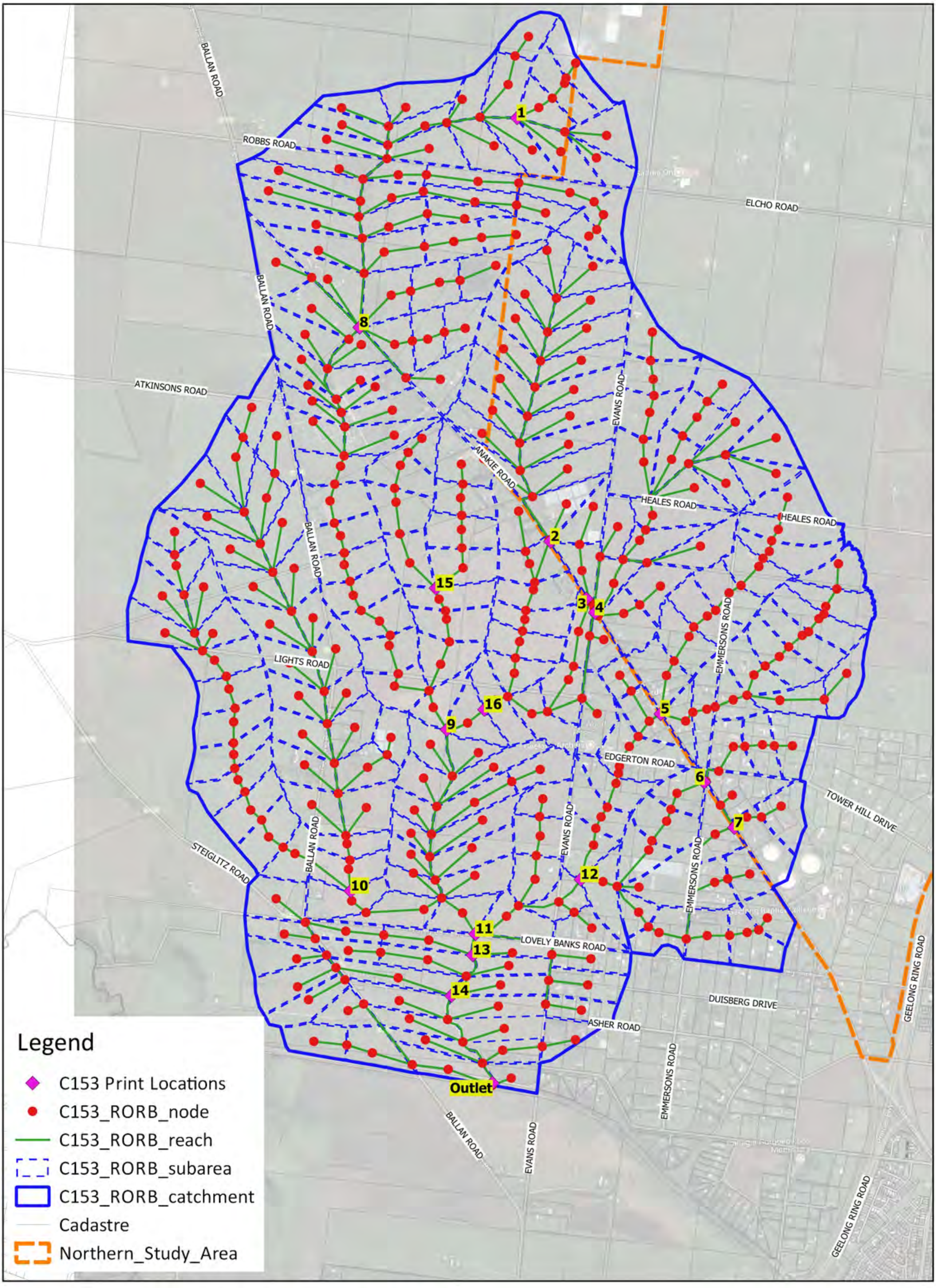
Site	Descriptions/Comments
1	Cowies Creek at Warners Road and Railway line. Culvert size unknown.
2	Drainage pits located. Pipes sizes and layout unknown.
3	375dia pipe
4	2 x 225dia pipes
5	375dia pipe
6	900Wx600H Culvert
7	300dia pipe
8	600Wx4500H Culvert
9	900Wx600H Culvert
10	3 x 900Wx450H, 1 x 750Wx450H Culverts
11	2 x 900Wx500H Culverts
12	1200Wx600H Culvert
13	750Wx375H Culvert
14	2 x 900Wx900H, 1 x 1200Wx600H Culverts
15	2 x 1200Wx900H, 1 x 900Wx300H, 1 x 1200Wx600H Culverts
16	750Wx300H Culvert
17	2 x 750Wx450H, 2 x 1200Wx600H Culverts
18	3 x 1350dia pipes
19	4 x 1800Wx900H Culverts
20	3 x 1500dia pipes

APPENDIX C GEELONG SOIL MAP

APPENDIX C - GEELONG SOIL MAP

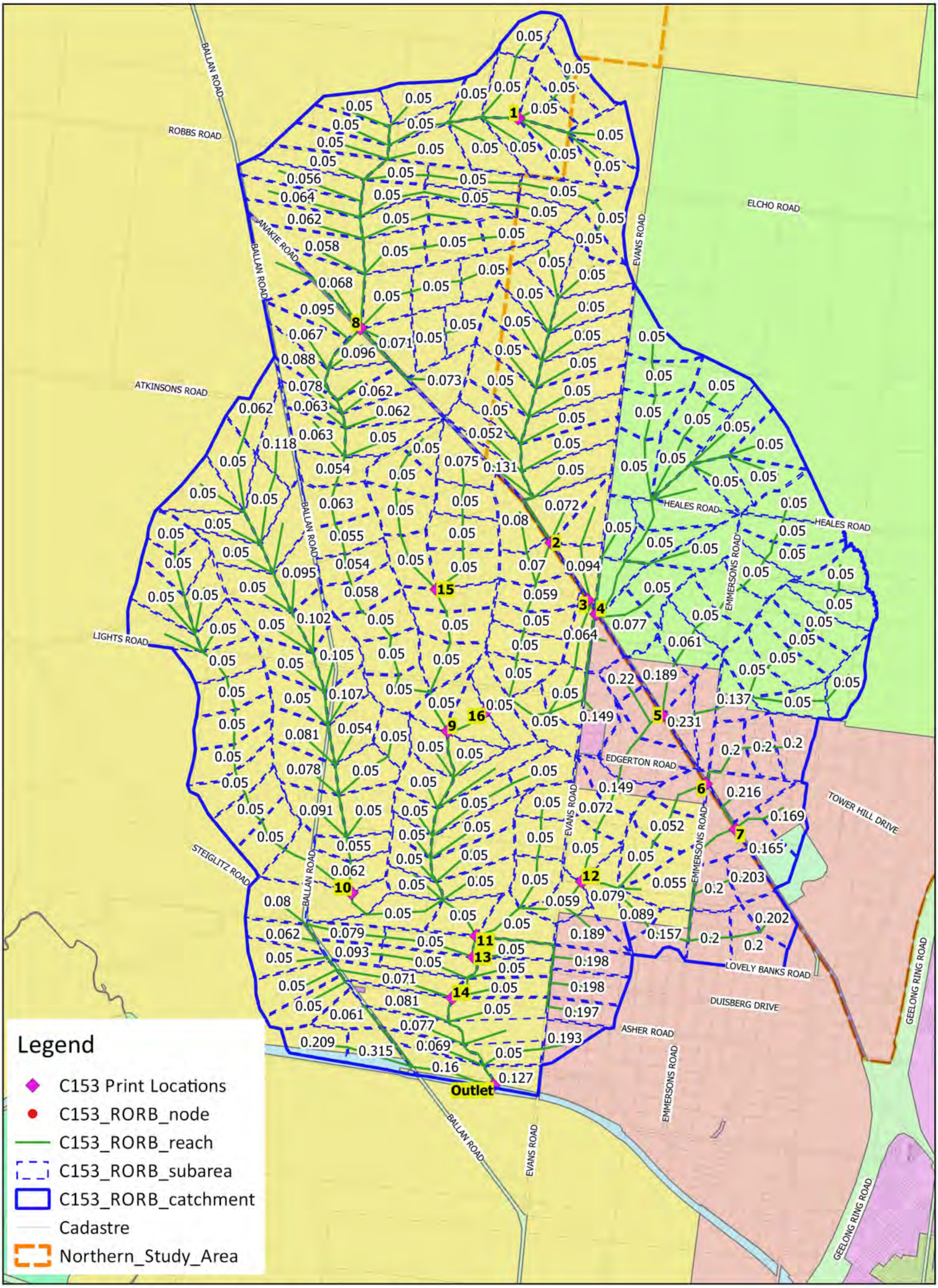


APPENDIX D COWIES CREEK FIGURES AND MAPS



APPENDIX D1 **COWIES CREEK | C153**

RORB MODEL



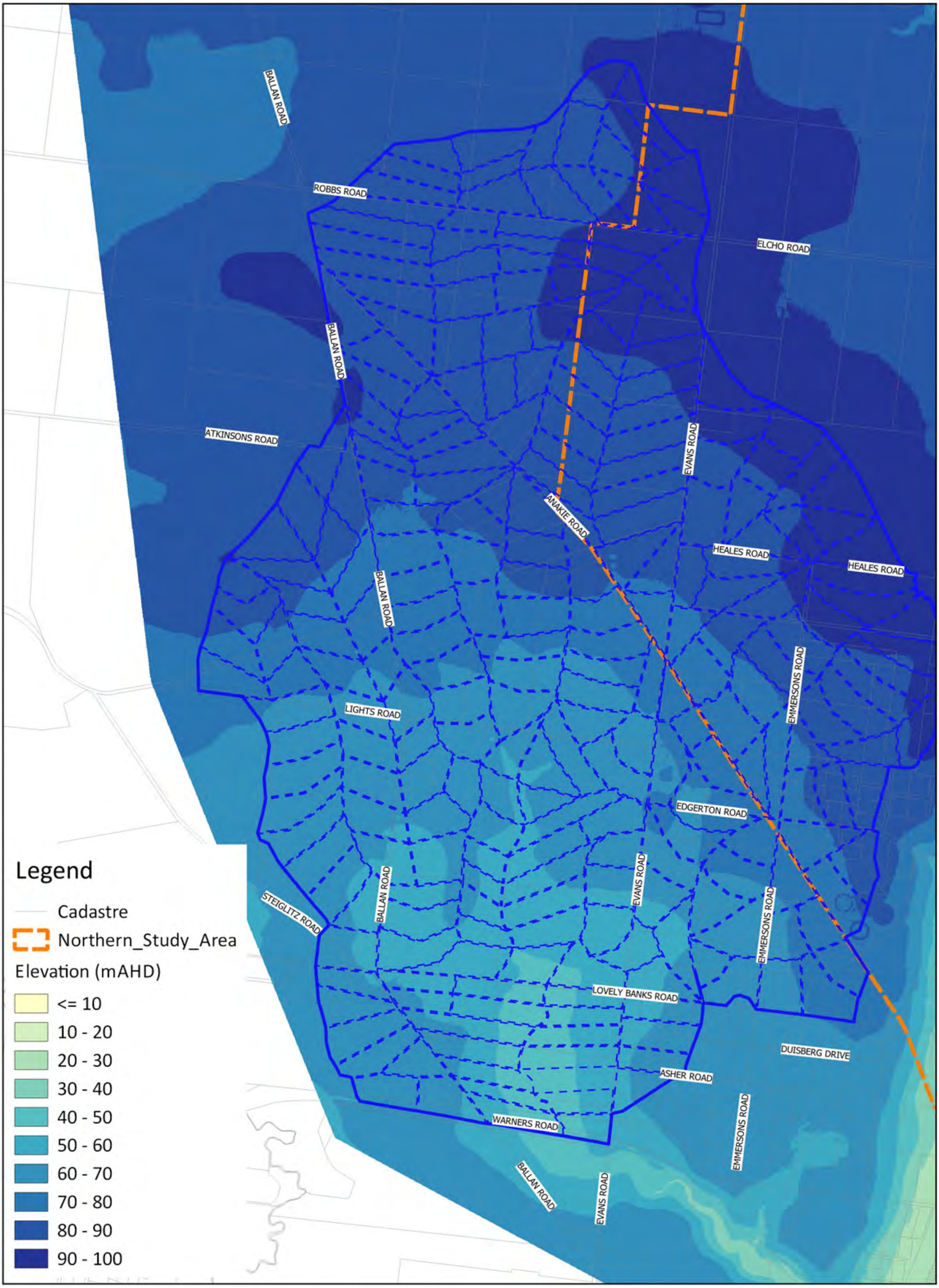
APPENDIX D2 **COWIES CREEK | C153**

RORB SUBAREA | EXISTING CONDITIONS FRACTION IMPERVIOUS

APPENDIX D3 - Vicroads Rational Method vs RORB Undeveloped Model Flow Comparison

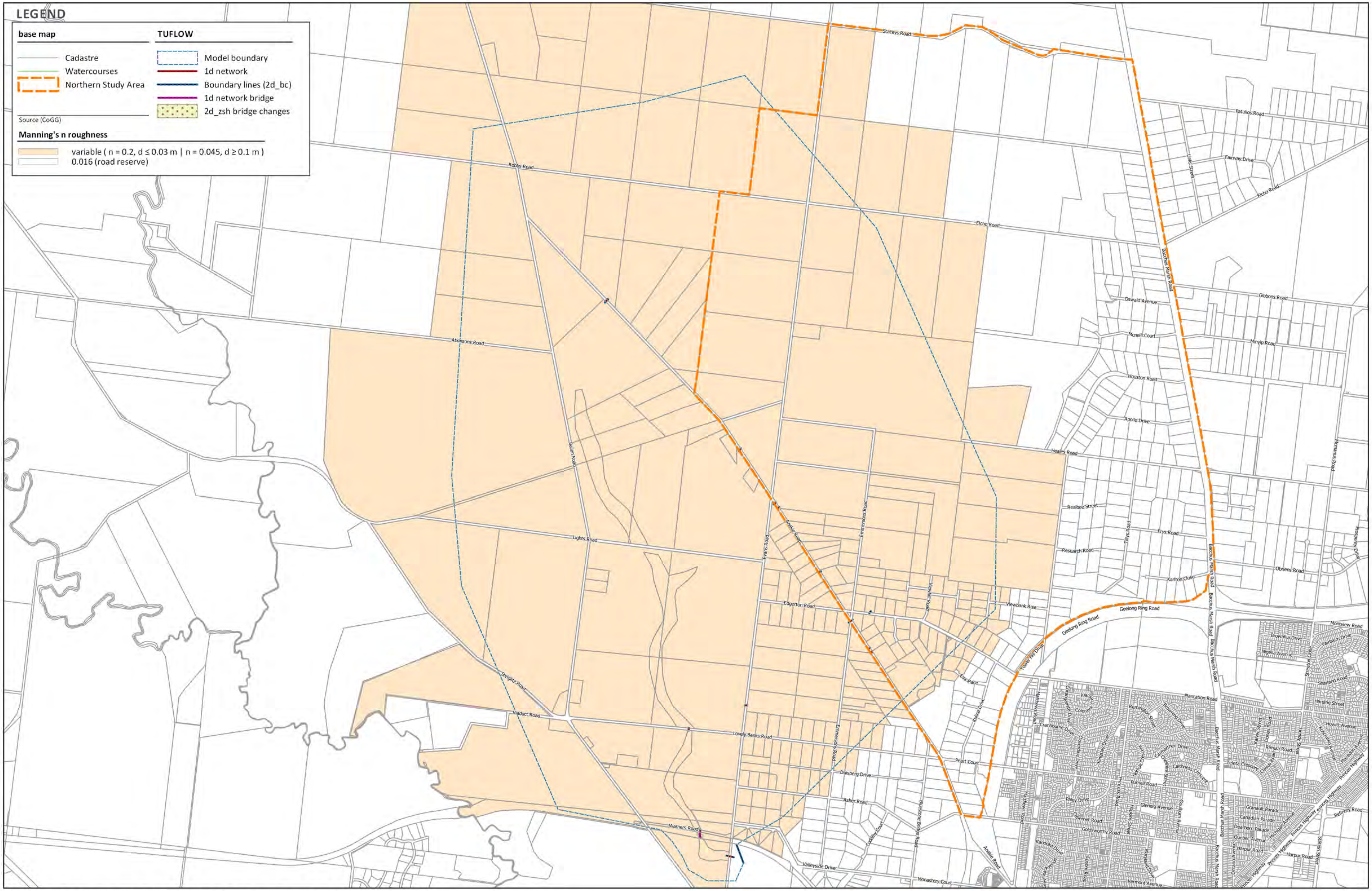
COWIES CREEK C153 - 1 IN 100 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hrs)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	0.5	0.6	0.12	1.3	2.08	65.0	2	2.5		2.9	-15
2	1.5	0.9	0.12	1.3	2.03	50.0	9	4.7		6.6	-29
3	0.1	0.3	0.12	1.3	2.10	92.0	2	0.6		0.8	-23
4	1.6	0.9	0.12	1.3	2.02	50.0	9	5.0		7.0	-28
5	1.6	0.9	0.12	1.3	2.02	50.0	3	6.0		7.0	-14
6	0.2	0.4	0.12	1.3	2.09	80.0	2	1.3		1.5	-12
7	0.3	0.5	0.12	1.3	2.09	73.0	1	2.1		2.0	7
8	3.5	1.2	0.12	1.3	1.93	44.6	9	9.6		13.0	-26
9	10.1	1.8	0.12	1.3	1.60	30.3	12	22.4		21.2	6
10	3.2	1.2	0.12	1.3	1.94	44.6	12	8.5		12.0	-29
11	18.9	2.3	0.12	1.3	1.47	25.4	12	39.8		30.5	30
12	3.7	1.2	0.12	1.3	1.92	44.6	12	11.1		13.7	-19
13	19.3	2.3	0.12	1.3	1.46	25.4	12	40.3		31.1	30
14	19.9	2.4	0.12	1.3	1.45	25.4	12	41.0		31.8	29
15	0.7	0.7	0.12	1.3	2.07	60.0	2	2.9		3.8	-23
16	4.1	1.2	0.12	1.3	1.90	44.6	12	11.0		15.0	-27
Outlet	20.66	2.4	0.12	1.3	1.44	25.4	12	41.4	29.0	32.8	26

COWIES CREEK C153 - 1 IN 5 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	0.5	0.6	0.12	0.9	2.08	32.4	9	0.9		1.0	-20
2	1.5	0.9	0.12	0.9	2.03	25.7	12	1.7		2.3	-26
3	0.1	0.3	0.12	0.9	2.10	44.8	9	0.2		0.3	-22
4	1.6	0.9	0.12	0.9	2.02	25.7	12	1.9		2.5	-37
5	1.6	0.9	0.12	0.9	2.02	25.7	12	2.2		2.5	-16
6	0.2	0.41	0.12	0.9	2.09	39.5	9	0.5		0.5	-13
7	0.3	0.48	0.12	0.9	2.09	36.2	9	0.7		0.7	-4
8	3.5	1.22	0.12	0.9	1.93	22.9	12	3.7		4.6	-30
9	10.1	1.83	0.12	0.9	1.60	15.8	12	7.6		7.7	-15
10	3.2	1.18	0.12	0.9	1.94	22.9	12	3.2		4.3	-36
11	18.9	2.32	0.12	0.9	1.47	13.3	12	13.3		11.1	3
12	3.7	1.25	0.12	0.9	1.92	22.9	12	4.2		4.9	-25
13	19.3	2.34	0.12	0.9	1.46	13.3	12	13.5		11.3	2
14	19.9	2.37	0.12	0.9	1.45	13.3	12	13.7		11.5	1
15	0.7	0.66	0.12	0.9	2.07	30.3	9	1.1		1.3	-27
16	4.1	1.20	0.12	0.9	1.90	22.9	12	3.9		5.3	-37
Outlet	20.66	2.40	0.12	0.9	1.44	13.3	12	13.7	9.6	11.9	-1



APPENDIX D4 **COWIES CREEK | C153**

ELEVATION



APPENDIX D5 COWIES CREEK | C153

EXISTING CONDITIONS | ROUGHNESS PARAMETER

Northern Geelong Growth Area | Flood Impact Assessment

client: CITY OF GREATER GEELONG | Lovely Banks

consultant: SMEC — Member of the Surbana Jurong Group

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CREATED BY Velasco, Karl

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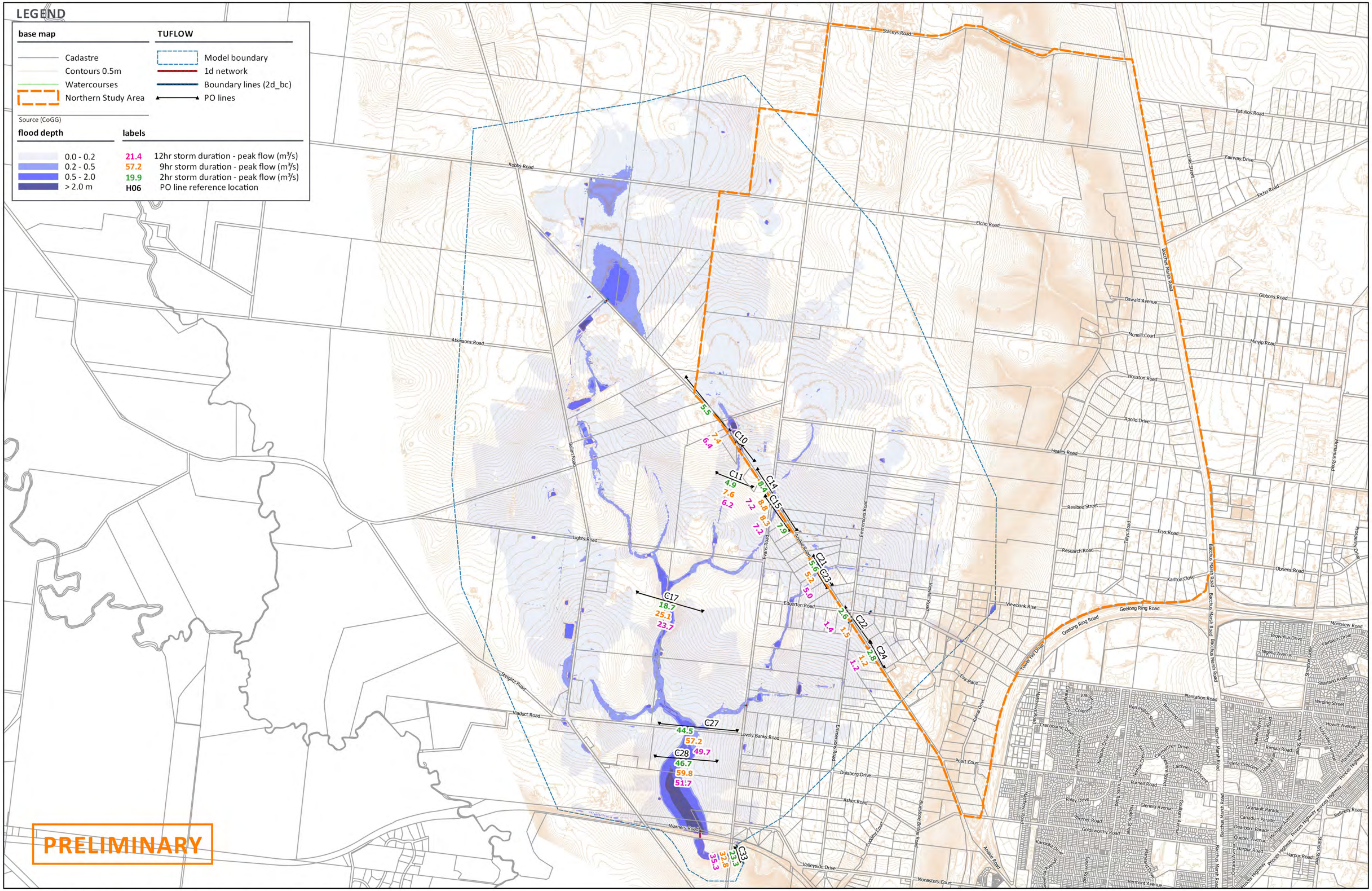

APPENDIX D6 COWIES CREEK | C153

EXISTING CONDITIONS | INDICATIVE 1 IN 100 AEP FLOOD EXTENT

Northern Geelong Growth Area | Flood Impact Assessment

client: CITY OF GREATER GEELONG | Lovely Banks

consultant: SMEC — Member of the Surbana Jurong Group

PROJECT NO 30042177W
DATE 21 Nov 2017
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CREATED BY Velasco, Karl

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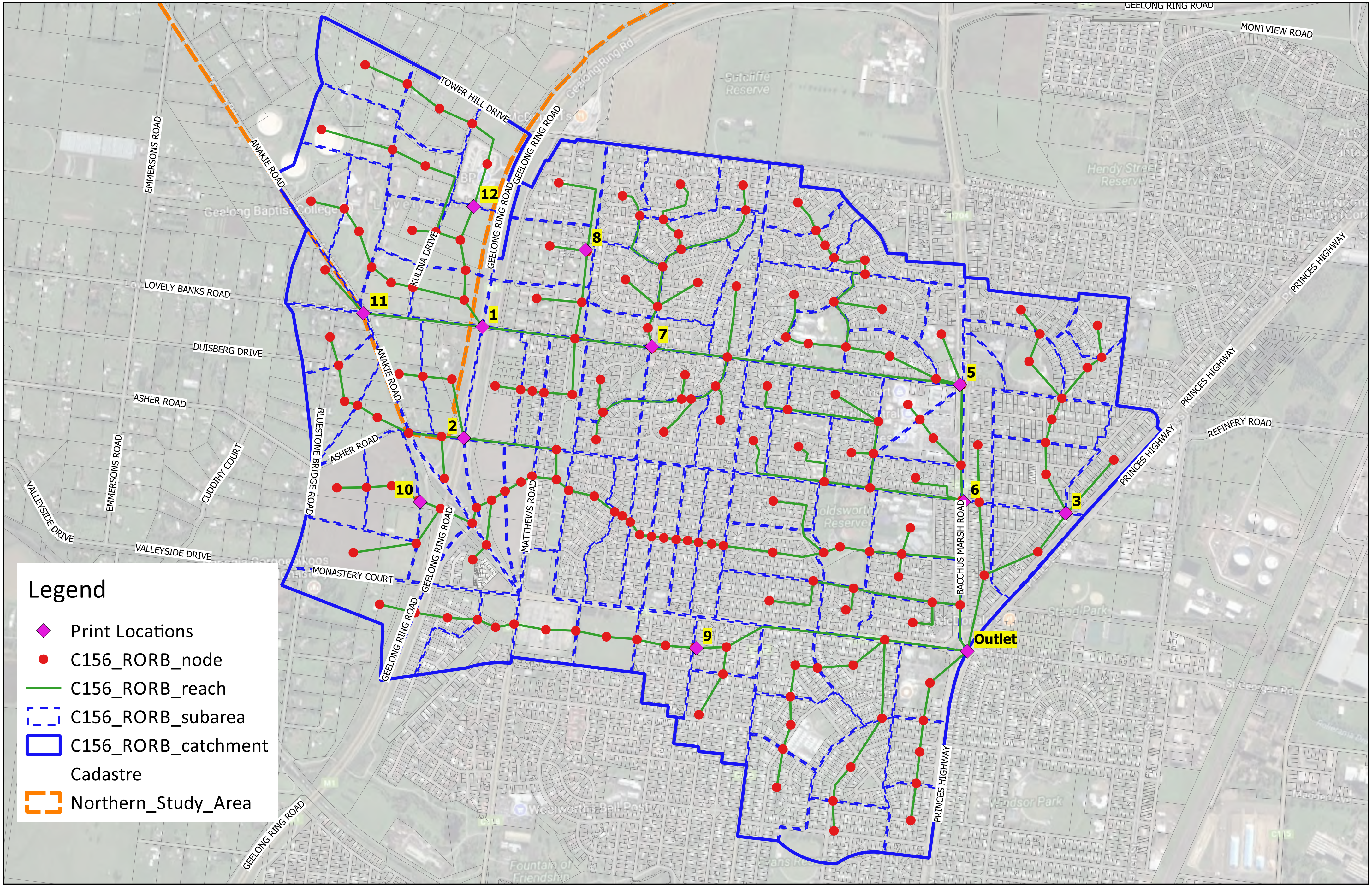
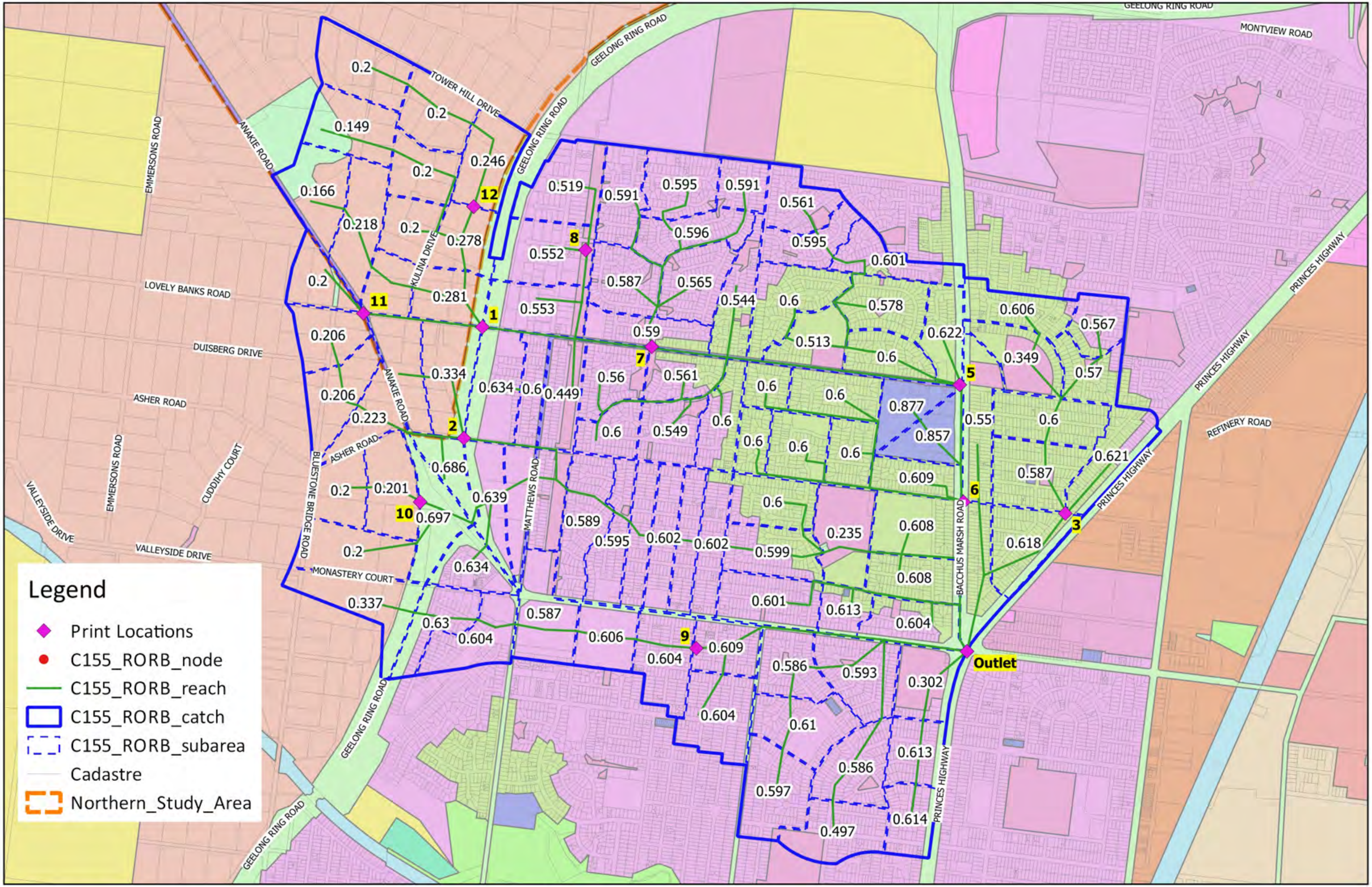
APPENDIX E ST GEORGES DRAIN FIGURES AND MAPS



APPENDIX E1 **ST GEORGES MAIN DRAIN | C155**

RORB MODEL



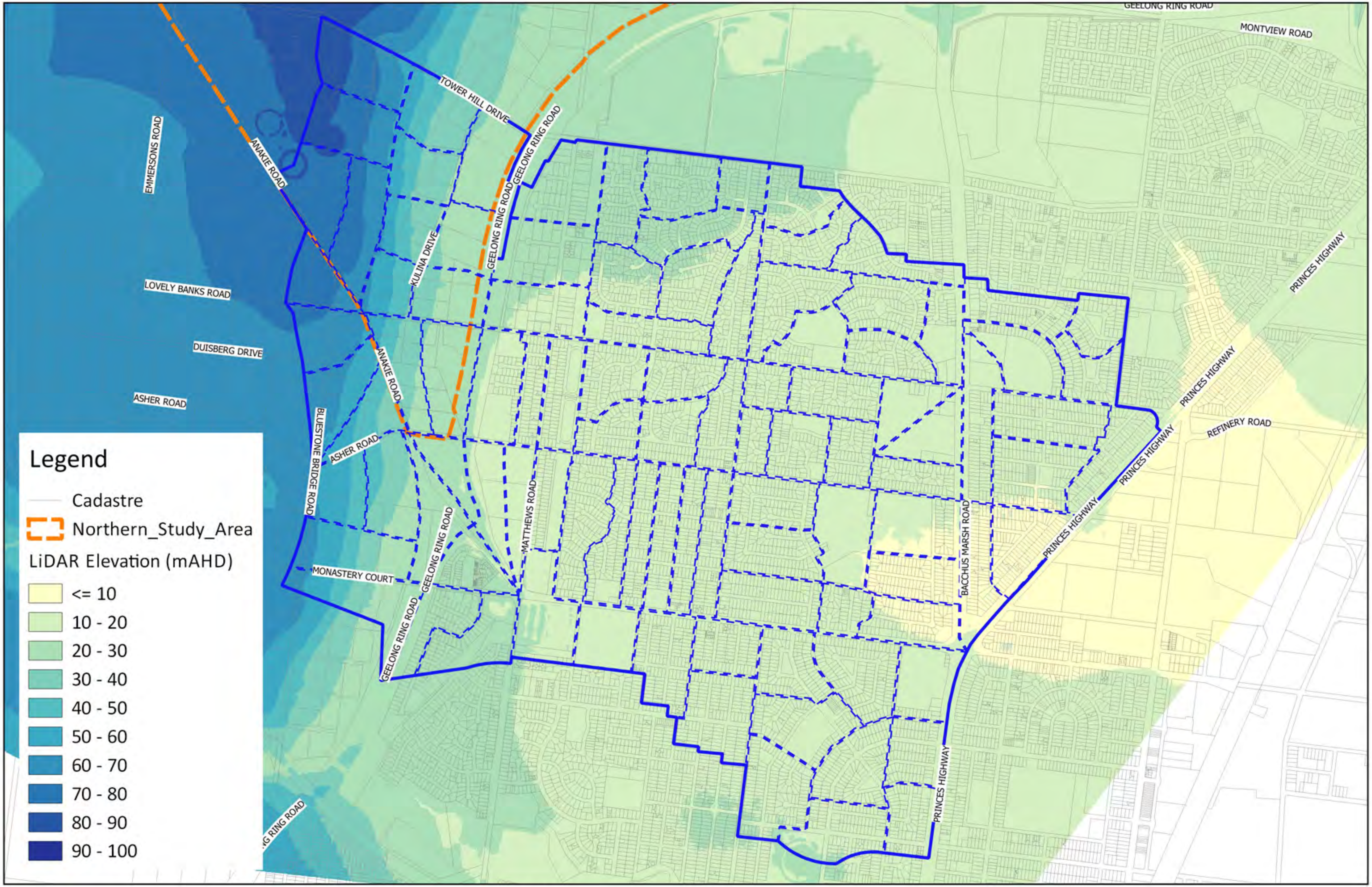
APPENDIX E2 **ST GEORGES MAIN DRAIN | C155**

RORB SUBAREA | EXISTING CONDITIONS FRACTION IMPERVIOUS

APPENDIX E3 - Vicroads Rational Method vs RORB Undeveloped Model Flow Comparison

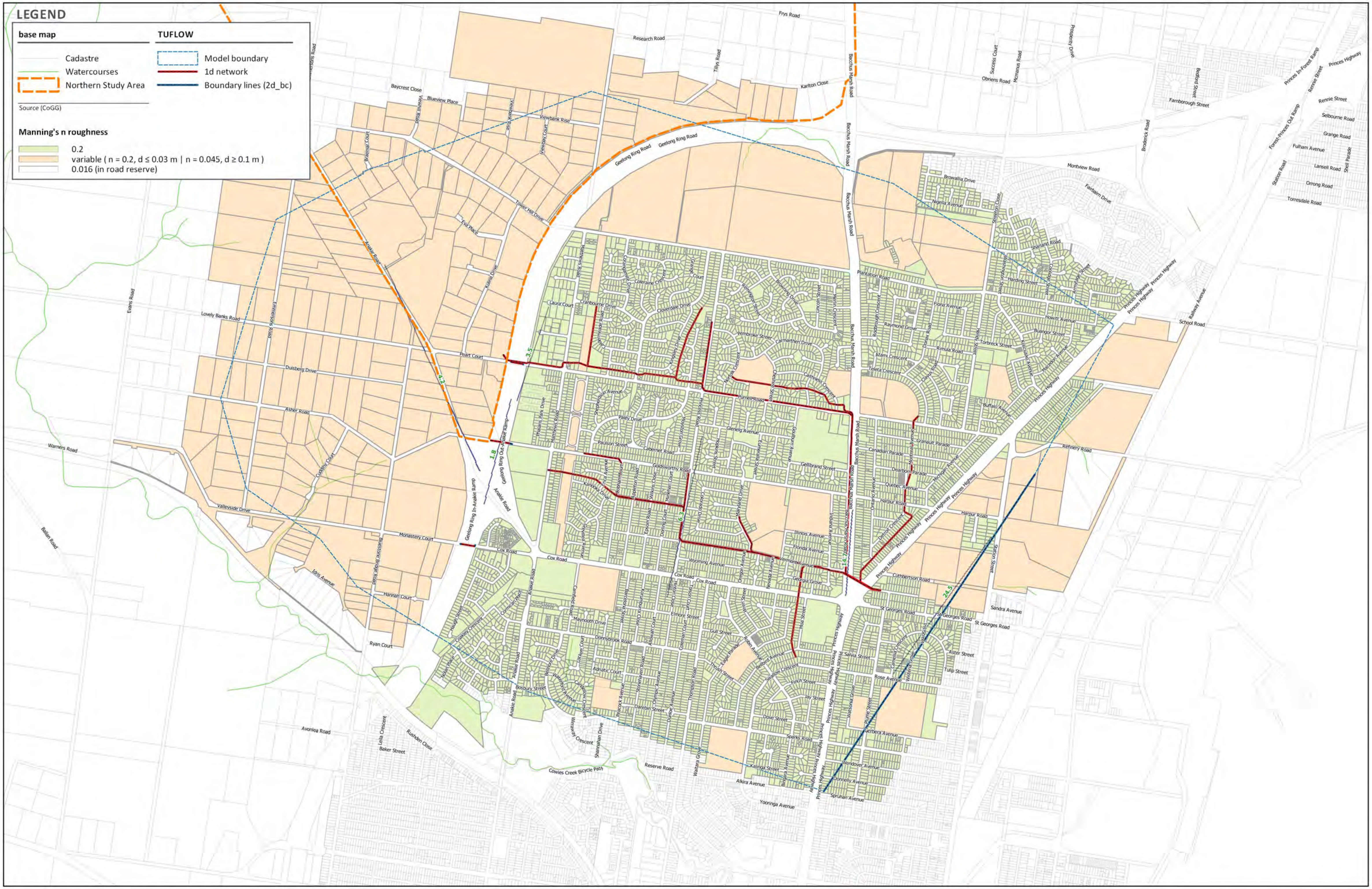
ST GEORGES ROAD DRAIN C155 - 1 IN 100 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	0.9	0.7	0.12	1.3	2.06	58	2	3.7	2.2	4.7	-20
2	0.4	0.5	0.12	1.3	2.08	66	2	2.6	1.1	2.4	8
3	0.5	0.6	0.12	1.3	2.08	64	2	2.7		2.9	-7
4	5.3	1.4	0.12	1.3	1.84	37.8	12	14.6		15.9	-9
5	2.9	1.1	0.12	1.3	1.96	44	9	8.4		10.8	-22
6	3.3	1.2	0.12	1.3	1.94	44	12	9.1		12.2	-25
7	1.9	1.0	0.12	1.3	2.01	46.6	9	7.0		7.7	-9
8	0.2	0.4	0.12	1.3	2.09	79	2	1.7		1.4	17
9	0.5	0.6	0.12	1.3	2.08	64	3	2.1		2.9	-26
10	0.2	0.4	0.12	1.3	2.09	79	1	1.3		1.4	-11
11	0.1	0.3	0.12	1.3	2.10	91	1	0.4		0.8	-47
12	0.3	0.5	0.12	1.3	2.09	72	2	1.2		2.0	-38
Outlet	7.5	1.6	0.12	1.3	1.72	33.4	12	20.3	12.5	18.8	8

ST GEORGES ROAD DRAIN C155 - 1 IN 5 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	0.9	0.7	0.12	0.9	2.06	29.4	12	1.2	0.7	1.6	-29
2	0.4	0.5	0.12	0.9	2.08	33.5	9	1.0	0.4	0.8	14
3	0.5	0.6	0.12	0.9	2.08	32.4	9	1.0		1.0	0
4	5.3	1.4	0.12	0.9	1.84	19.8	12	5.6		5.8	-3
5	2.9	1.1	0.12	0.9	1.96	22.9	12	3.1		3.9	-21
6	3.3	1.2	0.12	0.9	1.94	22.9	12	3.4		4.4	-23
7	1.9	1.0	0.12	0.9	2.01	24.2	12	2.6		2.8	-6
8	0.2	0.4	0.12	0.9	2.09	39.5	9	0.5		0.5	5
9	0.5	0.6	0.12	0.9	2.08	32.4	12	0.8		1.0	-21
10	0.2	0.4	0.12	0.9	2.09	39.5	9	0.4		0.5	-15
11	0.1	0.3	0.12	0.9	2.10	44.9	9	0.2		0.3	-47
12	0.3	0.5	0.12	0.9	2.09	36.2	12	0.5		0.7	-32
Outlet	7.5	1.6	0.12	0.9	1.72	17.5	12	7.8	4.2	6.8	15



APPENDIX E4 **ST GEORGES MAIN DRAIN | C155**

ELEVATION



APPENDIX E5 ST GEORGES MAIN DRAIN | C155

EXISTING CONDITIONS | ROUGHNESS PARAMETER

Northern Geelong Growth Areas | Flood Impact Assessment

client: CITY OF GREATER GEELONG; Lovely Banks

consultant: SMEC — Member of the Surbana Jurong Group

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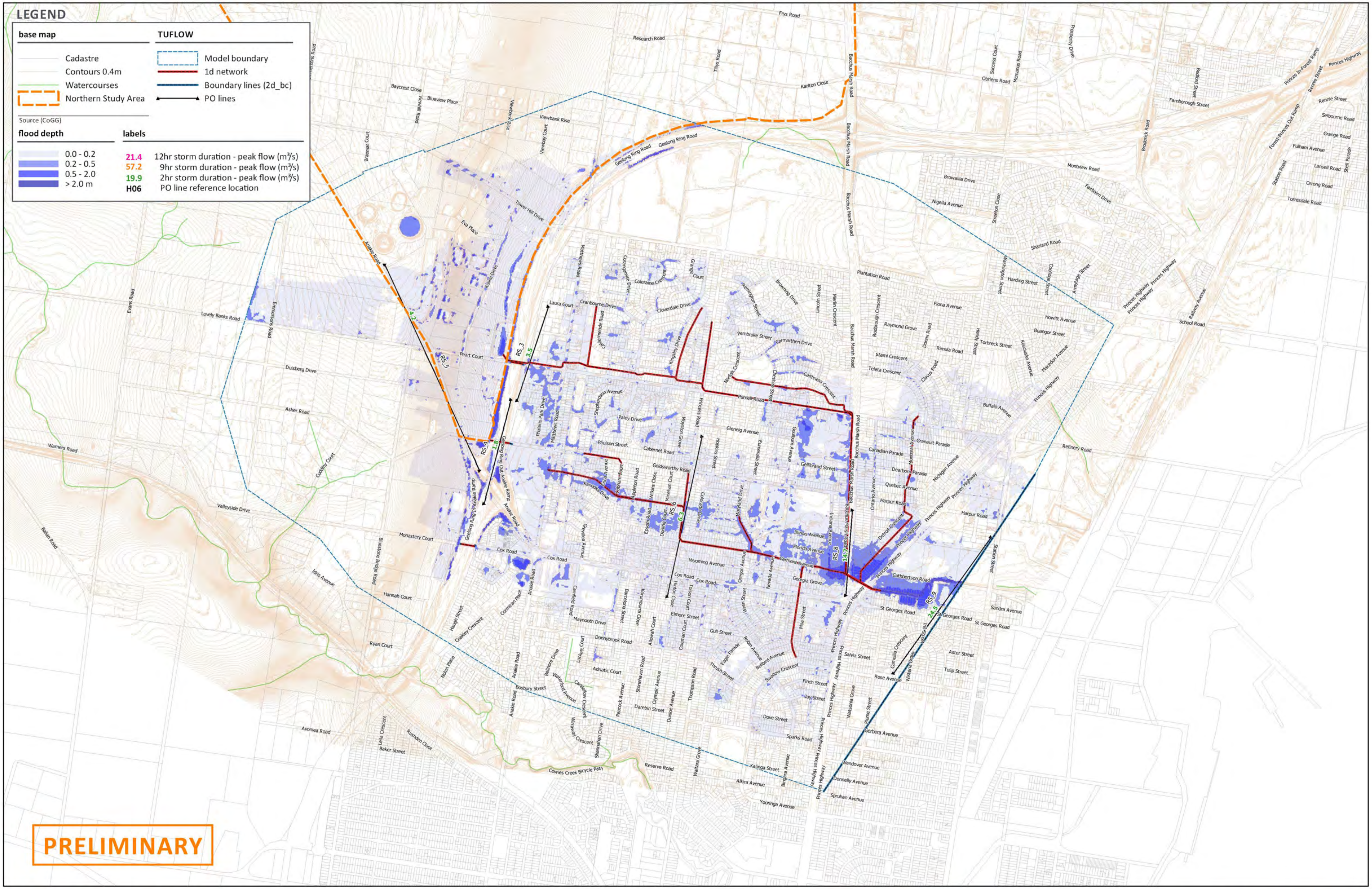

APPENDIX E6 ST GEORGES MAIN DRAIN | C155

EXISTING CONDITIONS | INDICATIVE 1 IN 100 AEP FLOOD EXTENT

Northern Geelong Growth Areas | Flood Impact Assessment

client: CITY OF GREATER GEELONG; Lovely Banks

consultant: SMEC — Member of the Surbana Jurong Group

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PROJECTION MGA Zone 55

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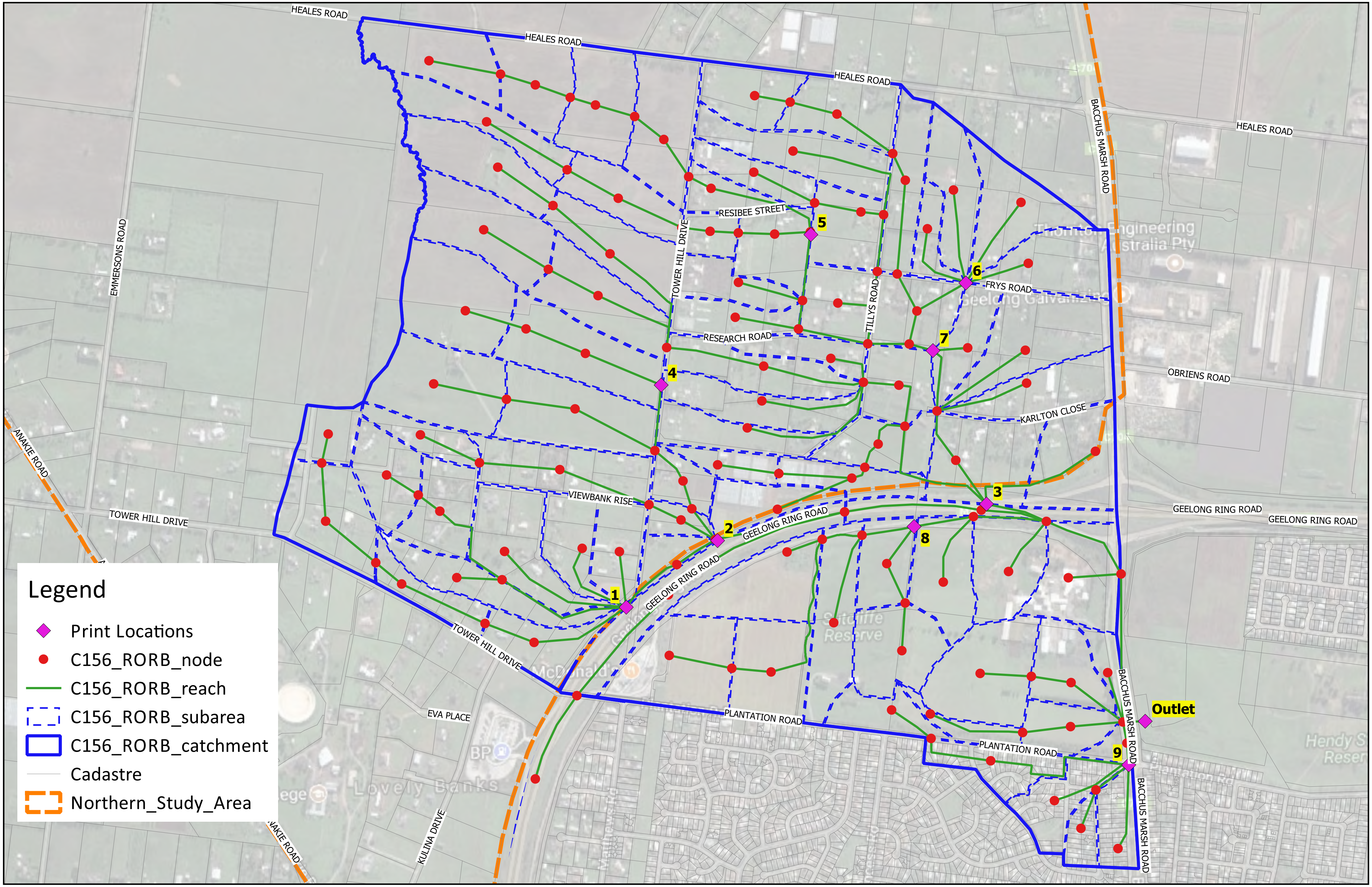
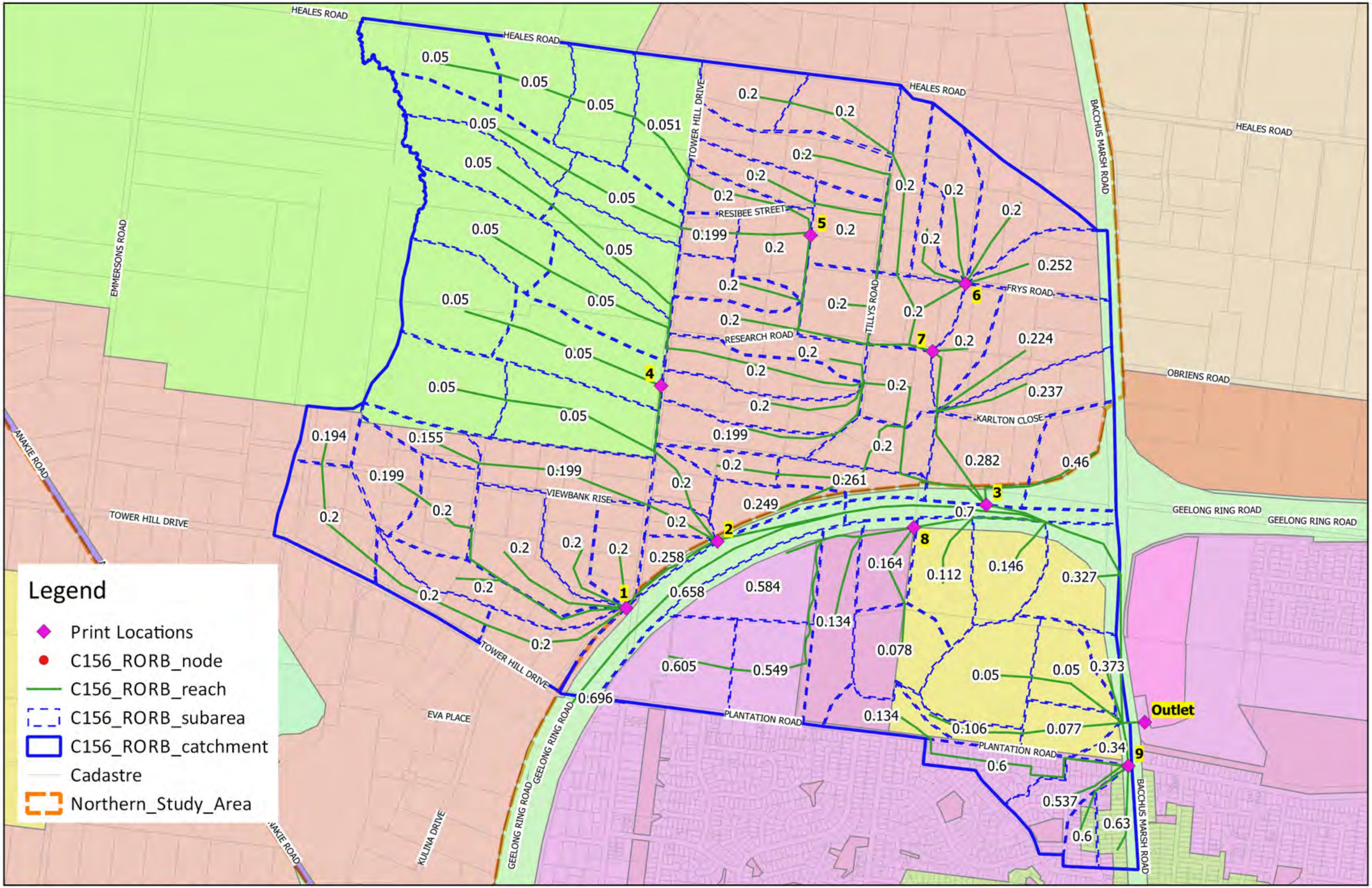
APPENDIX F WHARF RD DRAIN FIGURES AND MAPS



APPENDIX F1 **WHARF ROAD DRAIN | C156**

RORB MODEL



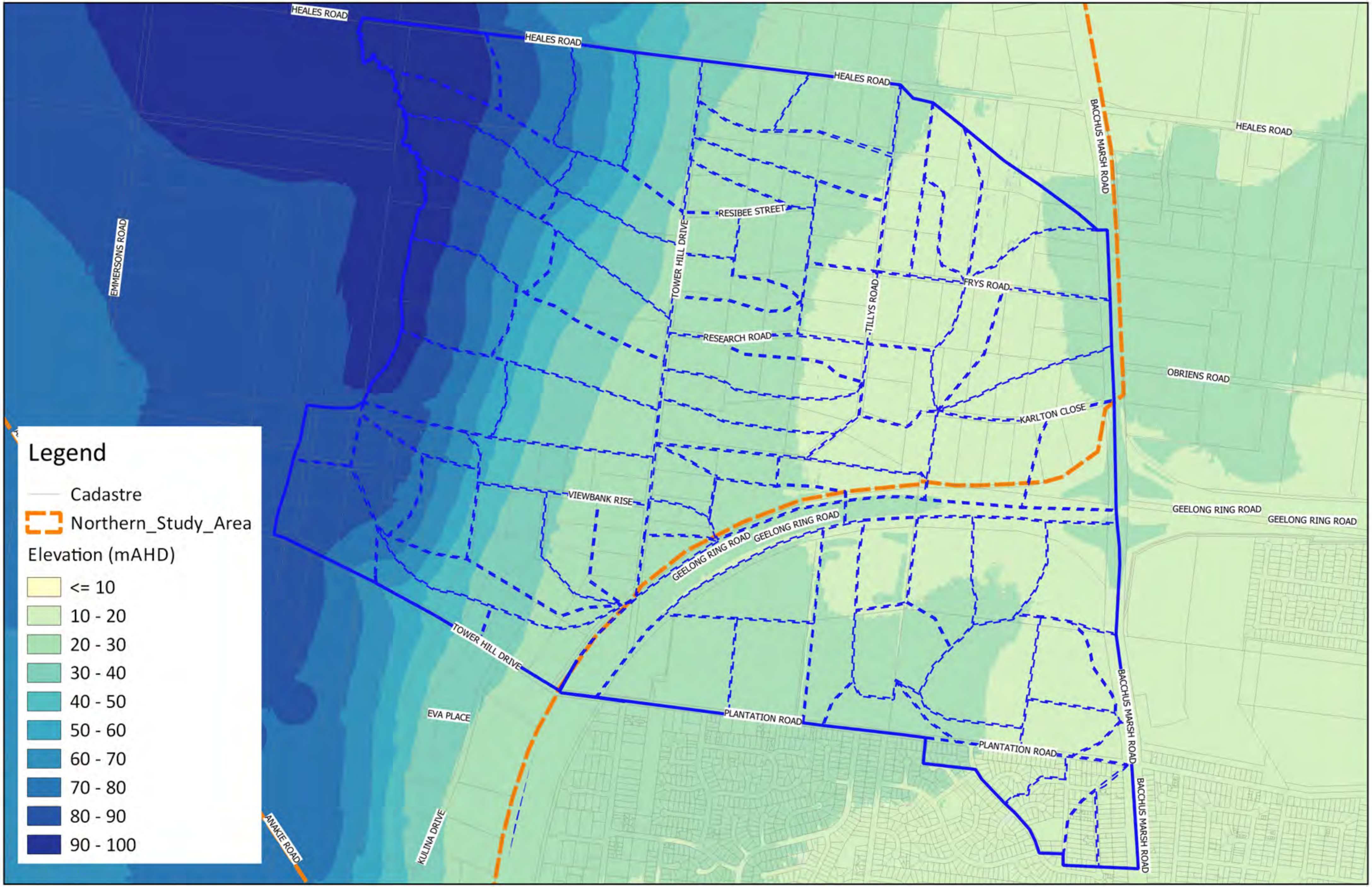
APPENDIX F2 **WHARF ROAD DRAIN | C156**

RORB SUBAREA | EXISTING CONDITIONS FRACTION IMPERVIOUS

APPENDIX F3 - Vicroads Rational Method vs RORB Undeveloped Model Flow Comparison

WHARF ROAD C156 - 1 IN 100 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	0.50	0.6	0.12	1.3	2.08	64	2	2.4	1.4	2.9	-25
2	1.10	0.8	0.12	1.3	2.05	54	2	4.9	2.5	5.3	-16
3	2.40	1.1	0.12	1.3	1.98	44.3	9	8.9	4.7	9.1	-12
4	0.20	0.4	0.12	1.3	2.09	80	2	1.2		1.5	-28
5	0.50	0.6	0.12	1.3	2.08	64	2	2.3		2.9	-28
6	0.20	0.4	0.12	1.3	2.09	80	1	1.9		1.5	17
7	1.30	0.8	0.12	1.3	2.04	50	2	5.3		5.7	-13
8	0.40	0.5	0.12	1.3	2.08	67	2	2.5		2.4	-7
9	0.20	0.4	0.12	1.3	2.09	80	1	1.1		1.5	-34
Outlet	4.74	1.4	0.12	1.3	1.86	38.2	9	15.3	8.8	14.6	-5

WHARF ROAD C156 - 1 IN 5 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	0.50	0.6	0.12	0.9	2.08	32.4	9	0.9	0.5	1.0	-18
2	1.10	0.8	0.12	0.9	2.05	27.4	9	1.9	0.8	1.9	-8
3	2.40	1.1	0.12	0.9	1.98	23	9	3.3	1.6	3.3	-11
4	0.20	0.4	0.12	0.9	2.09	39.6	9	0.4		0.5	-22
5	0.50	0.6	0.12	0.9	2.08	32.4	9	0.9		1.0	-22
6	0.20	0.4	0.12	0.9	2.09	39.6	9	0.6		0.5	15
7	1.30	0.8	0.12	0.9	2.04	25.7	9	2.1		2.0	-9
8	0.40	0.5	0.12	0.9	2.08	33.6	9	0.9		0.8	1
9	0.20	0.4	0.12	0.9	2.09	39.6	9	0.4		0.5	-32
Outlet	4.7	1.4	0.12	0.9	1.86	19.9	12	5.7	3.0	5.3	-1



APPENDIX F4 **WHARF ROAD MAIN DRAIN | C156**

ELEVATION

APPENDIX F5 WHARF ROAD DRAIN | C156

EXISTING CONDITIONS | ROUGHNESS PARAMETER

Northern Geelong Growth Areas | Flood Impact Assessment

client CITY OF GREATER GEELONG

Lovely Banks

consultant SMEC Member of the Surbana Jurong Group

PROJECT NO 30042177W
DATE 27 Jul 2017
REVISION C
CREATED BY Velasco, Karl

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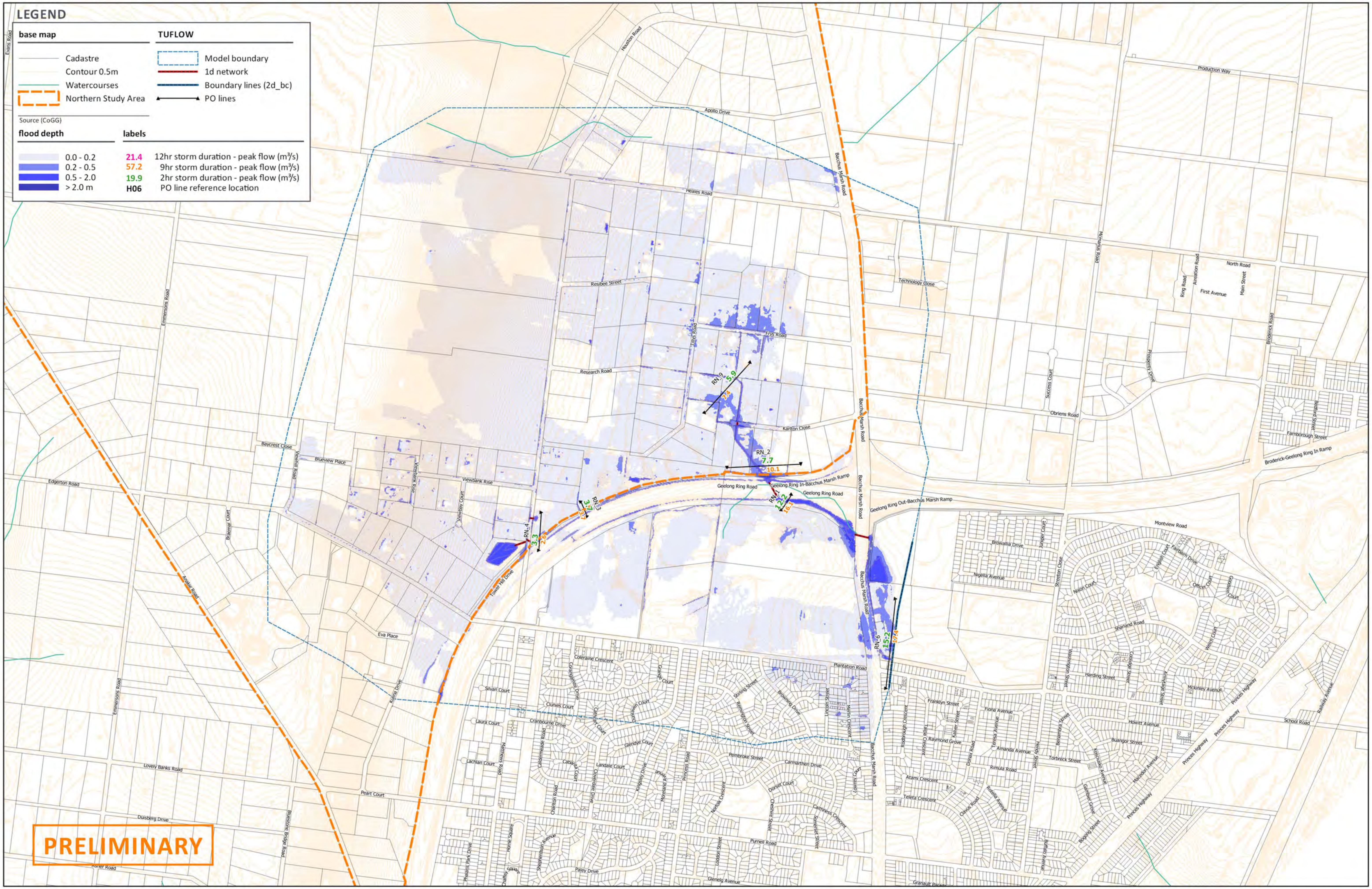
PROJECTION MGA Zone 55

Level 10 71 Queens Road Melbourne VIC 3004 Australia



APPENDIX F6 WHARF ROAD DRAIN | C156

EXISTING CONDITIONS | INDICATIVE 1 IN 100 AEP FLOOD

Northern Geelong Growth Areas | Flood Impact Assessment

client: CITY OF GREATER GEELONG | Lovely Banks

consultant: SMEC — Member of the Surbana Jurong Group

PROJECT NO	30042177W
DATE	21 Nov 2017
REVISION	C
CREATED BY	Velasco, Karl

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MAP SCALE	1:15,000
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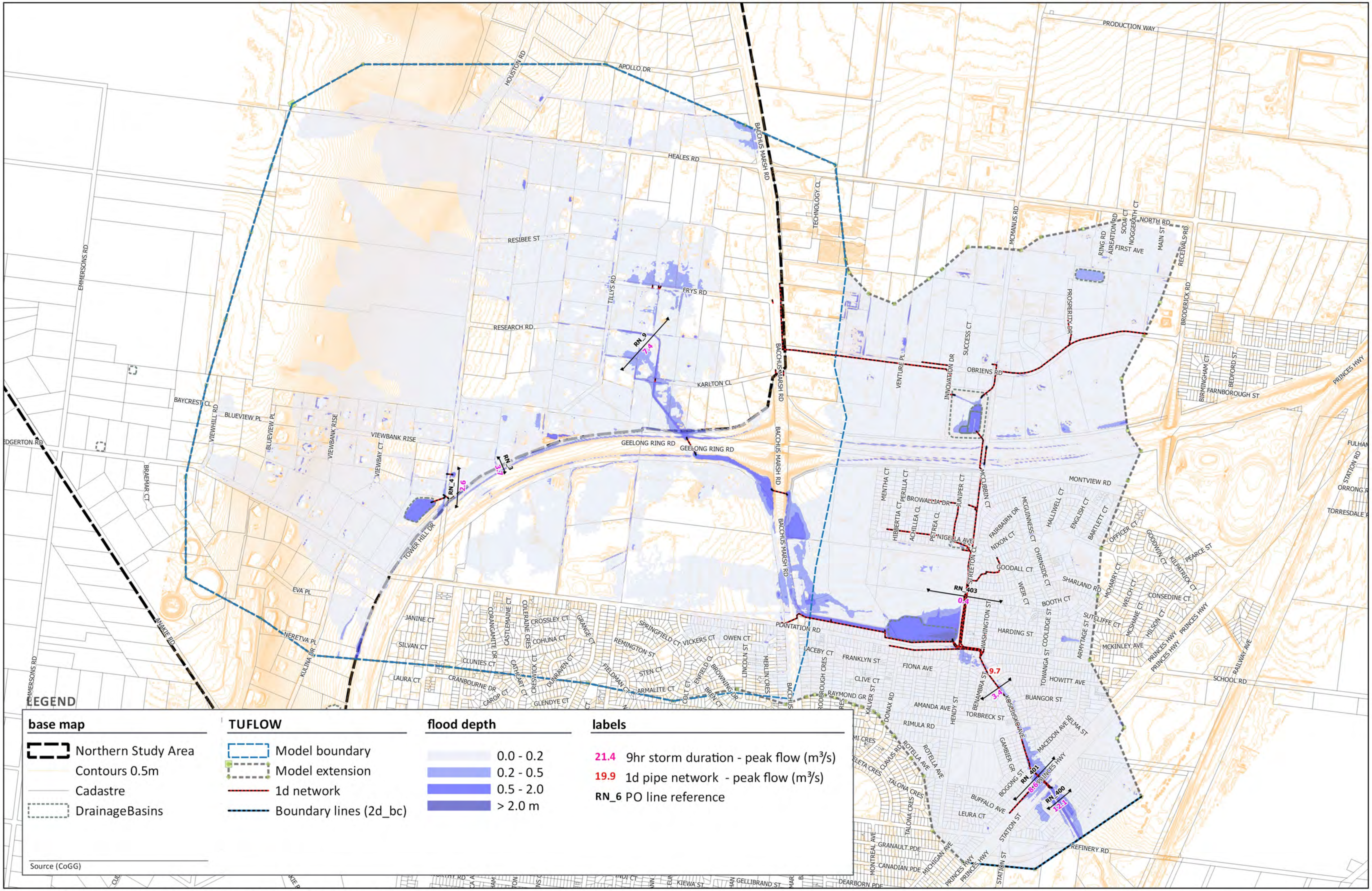
DATUM	GDA94
PROJECTION	MGA Zone 55

Level 10 71 Queens Road Melbourne VIC 3004 Australia



APPENDIX F7 WHARF ROAD DRAIN EXTENSION | C156

EXISTING CONDITIONS | 1 IN 100 AEP FLOOD EXTENT (9HR)

Northern Geelong Growth Areas | Flood Impact Assessment

client: CITY OF GREATER GEELONG — Lovely Banks

consultant: SMEC — Member of the Surbana Jurong Group

PROJECT NO 30042177W
DATE 10 May 2018
REVISION A
CREATED BY KV

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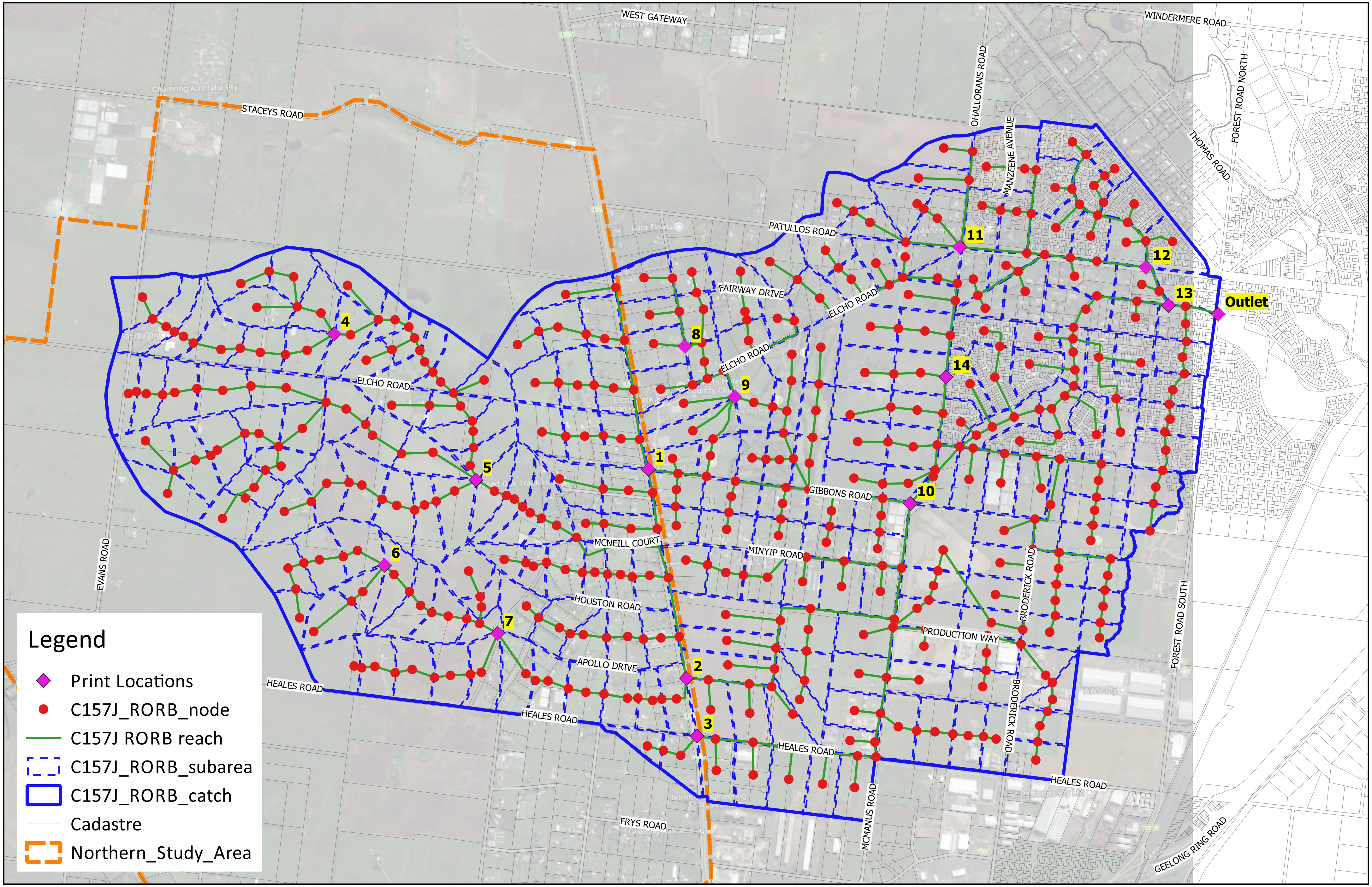
MAP SCALE 1:15,000
PAGE SIZE A3
DATUM GDA94
PROJECTION MGA Zone 55

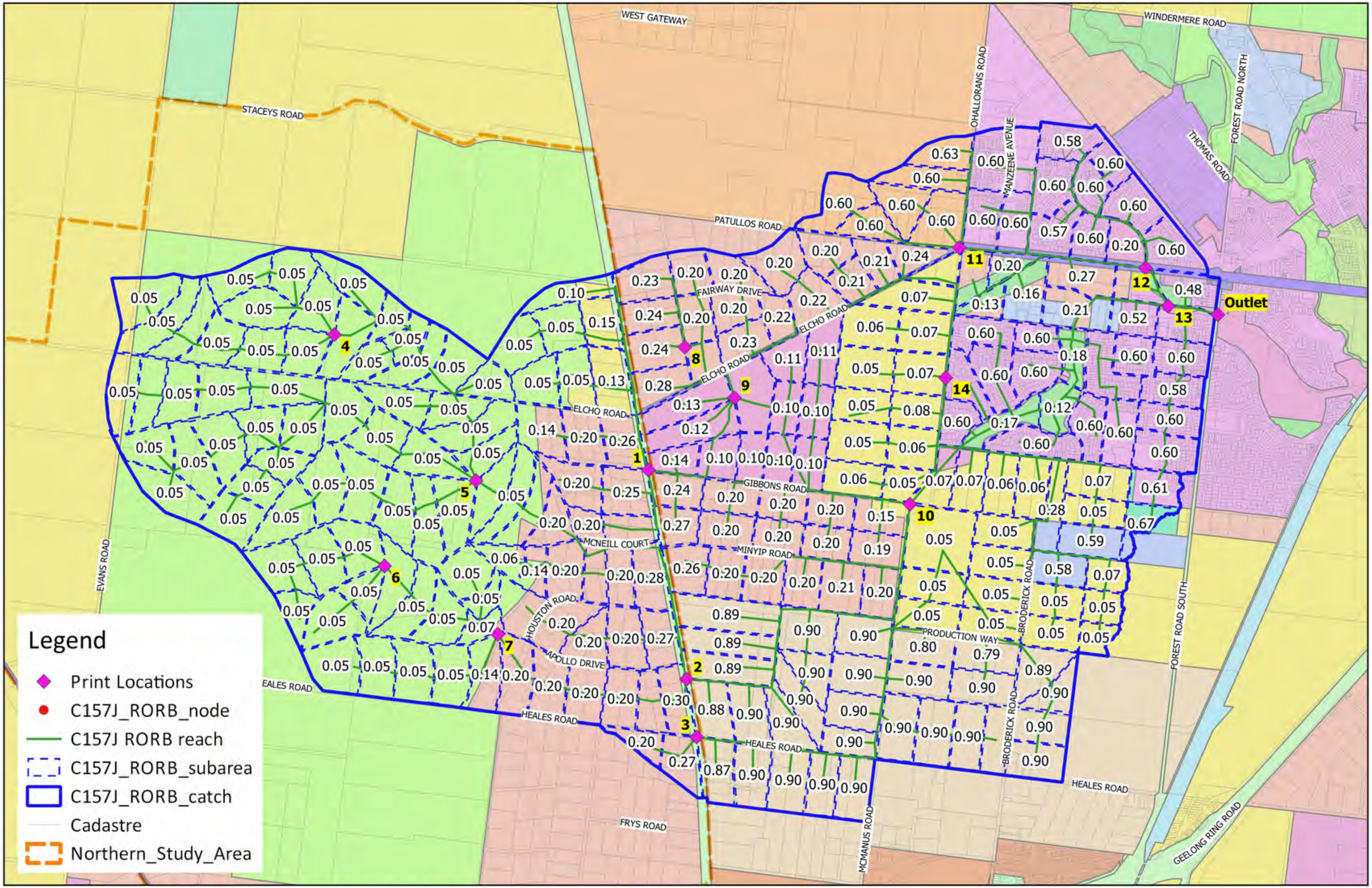
Level 10 71 Queens Road Melbourne VIC 3004 Australia

APPENDIX G ELCHO DRAIN FIGURES AND MAPS



APPENDIX G1 **ELCHO DRAIN | C157J**

RORB MODEL



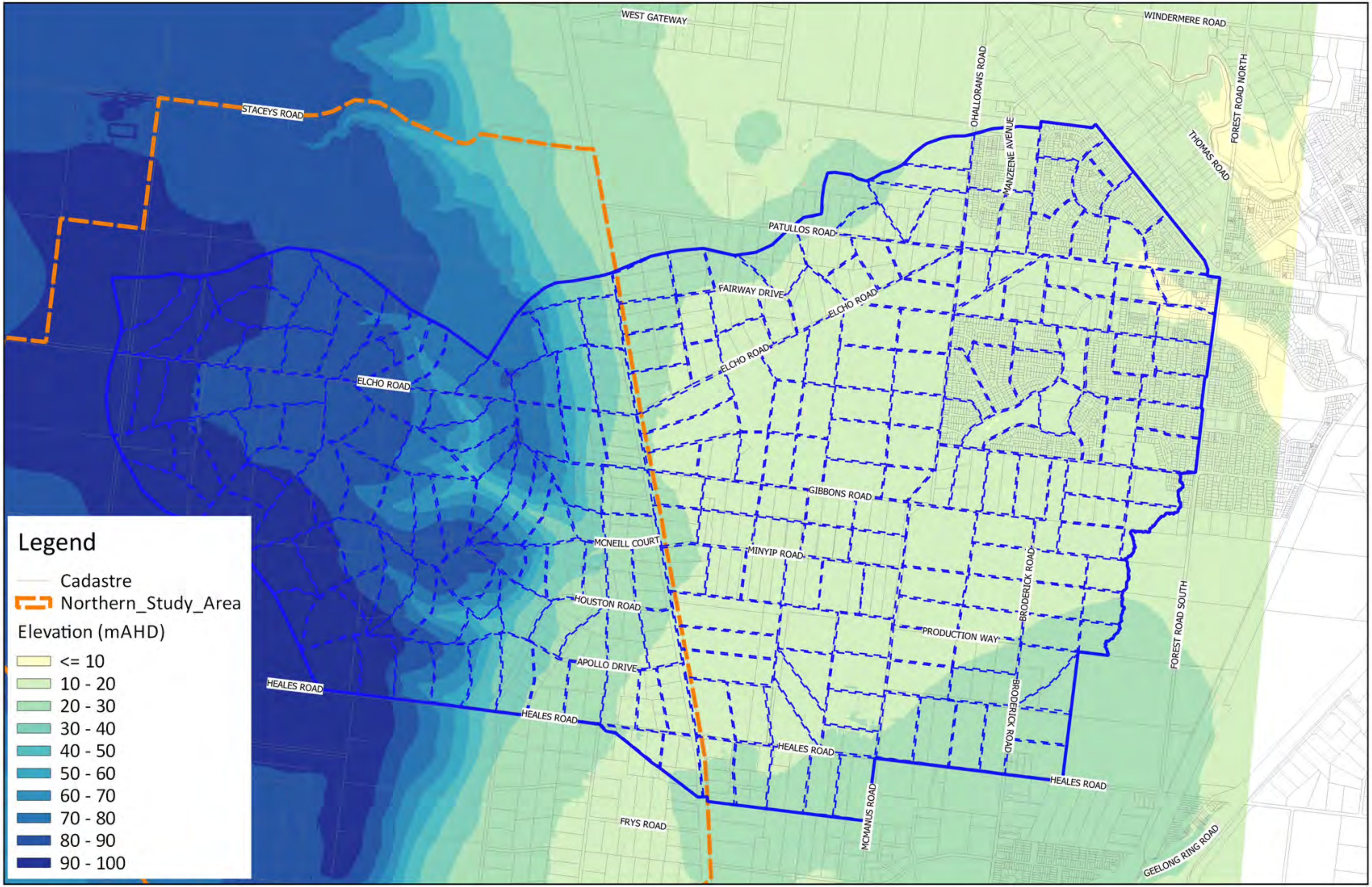
APPENDIX G2 **ELCHO DRAIN | C157J**

RORB SUBAREA | EXISTING CONDITIONS FRACTION IMPERVIOUS

APPENDIX G3 - Vicroads Rational Method vs RORB Undeveloped Model Flow Comparison

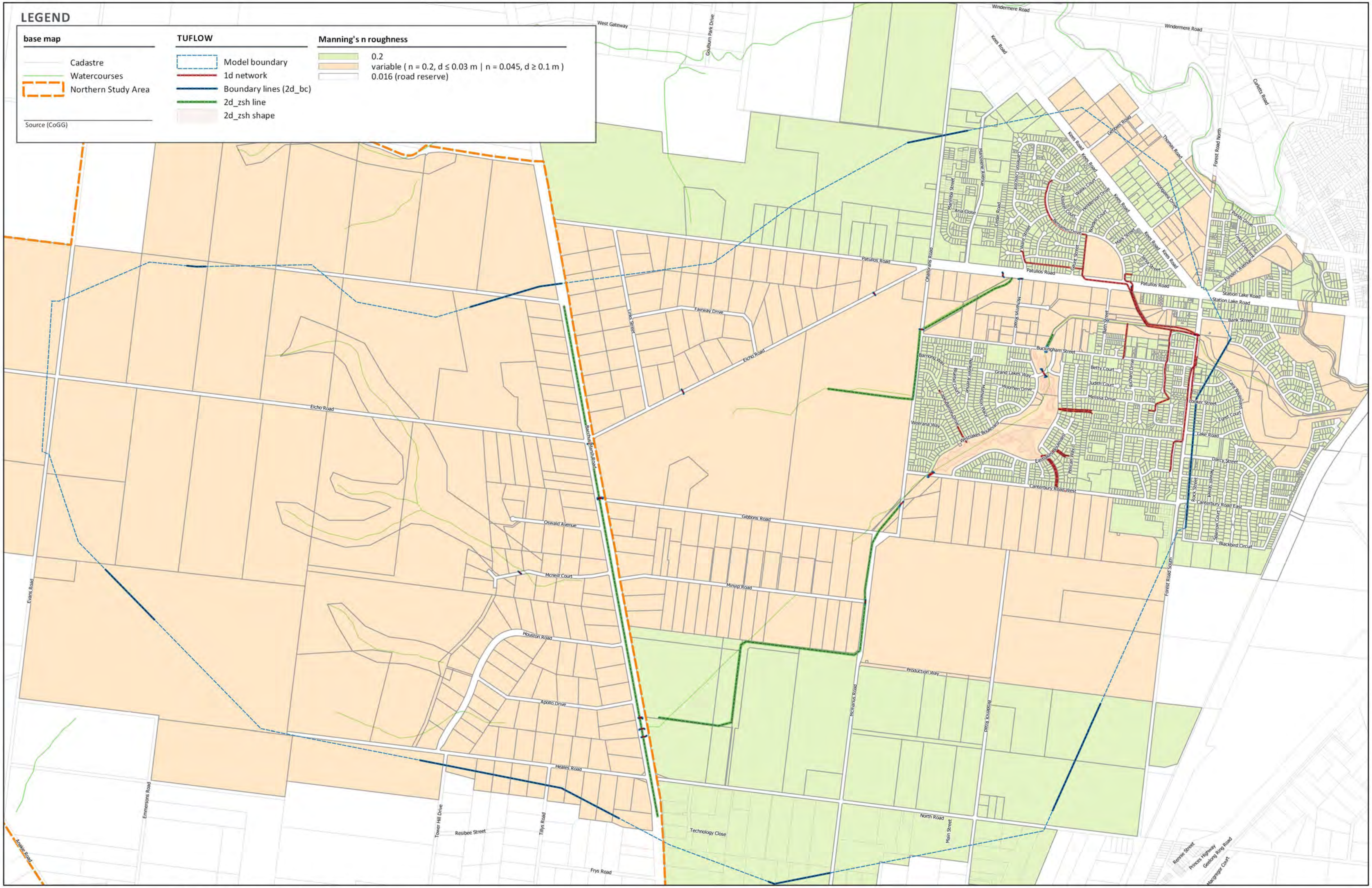
ELCHO DRAIN C157J - 1 IN 100 AEP FLOWS										
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	1.18	0.8	0.12	1.3	2.04	54	2	4.8	5.6	126
2	6.60	1.6	0.12	1.3	1.77	34.2	12	18.2	17.3	-56
3	0.10	0.3	0.12	1.3	2.10	92	1	1.0	0.8	-2
4	0.80	0.7	0.12	1.3	2.06	59	3	3.7	4.2	-25
5	3.80	1.3	0.12	1.3	1.91	38.7	9	12.5	12.2	-10
6	0.50	0.6	0.12	1.3	2.08	65	2	2.7	2.9	-22
7	1.40	0.9	0.12	1.3	2.03	51	3	5.7	6.3	-21
8	0.30	0.5	0.12	1.3	2.09	73	2	1.5	2.0	-36
9	2.32	1.0	0.12	1.3	1.98	44.9	9	8.8	9.0	55
10	11.40	1.9	0.12	1.3	1.58	30.8	12	26.3	24.0	-6
11	0.50	0.6	0.12	1.3	2.08	65	2	2.2	2.9	-34
12	1.80	1.0	0.12	1.3	2.01	47.6	9	6.8	7.5	-20
13	19.20	2.3	0.12	1.3	1.46	25.9	12	39.2	31.5	2
14	0.90	0.7	0.12	1.3	2.06	59	9	3.4	4.7	-37
Outlet	19.77	2.4	0.12	1.3	1.45	25.9	12	39.5	32.3	0

ELCHO DRAIN C157J - 1 IN 5 AEP FLOWS										
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	1.18	0.8	0.12	0.9	2.04	27.5	9	1.8	2.0	119
2	6.60	1.6	0.12	0.9	1.77	17.7	12	6.6	6.2	-55
3	0.10	0.3	0.12	0.9	2.10	45	9	0.3	0.3	2
4	0.80	0.7	0.12	0.9	2.06	29.5	9	1.4	1.5	-19
5	3.80	1.3	0.12	0.9	1.91	19.9	12	5.0	4.3	-6
6	0.50	0.6	0.12	0.9	2.08	32.5	9	1.0	1.0	-16
7	1.40	0.9	0.12	0.9	2.03	25.8	9	2.1	2.2	-17
8	0.30	0.5	0.12	0.9	2.09	36.4	9	0.5	0.7	-30
9	2.32	1.0	0.12	0.9	1.98	23	9	3.2	3.2	42
10	11.40	1.9	0.12	0.9	1.58	15.9	12	8.7	8.6	-16
11	0.50	0.6	0.12	0.9	2.08	32.5	9	0.9	1.0	-30
12	1.80	1.0	0.12	0.9	2.01	24.3	9	2.5	2.6	-19
13	19.20	2.3	0.12	0.9	1.46	13.5	12	12.8	11.4	-11
14	0.90	0.7	0.12	0.9	2.06	29.5	12	1.2	1.6	-30
Outlet	19.77	2.4	0.12	0.9	1.45	13.5	12	12.8	11.6	-13



APPENDIX G4 **ELCHO DRAIN | C157J**

ELEVATION



APPENDIX G5 ELCHO DRAIN | C157J

EXISTING CONDITIONS | ROUGHNESS PARAMETER

Northern Geelong Growth Areas | Flood Impact Assessment

client: CITY OF GREATER GEELONG; Lovely Banks

consultant: SMEC — Member of the Surbana Jurong Group

PROJECT NO 30042177W
DATE 27 Jul 2017
REVISION C
CREATED BY Velasco, Karl

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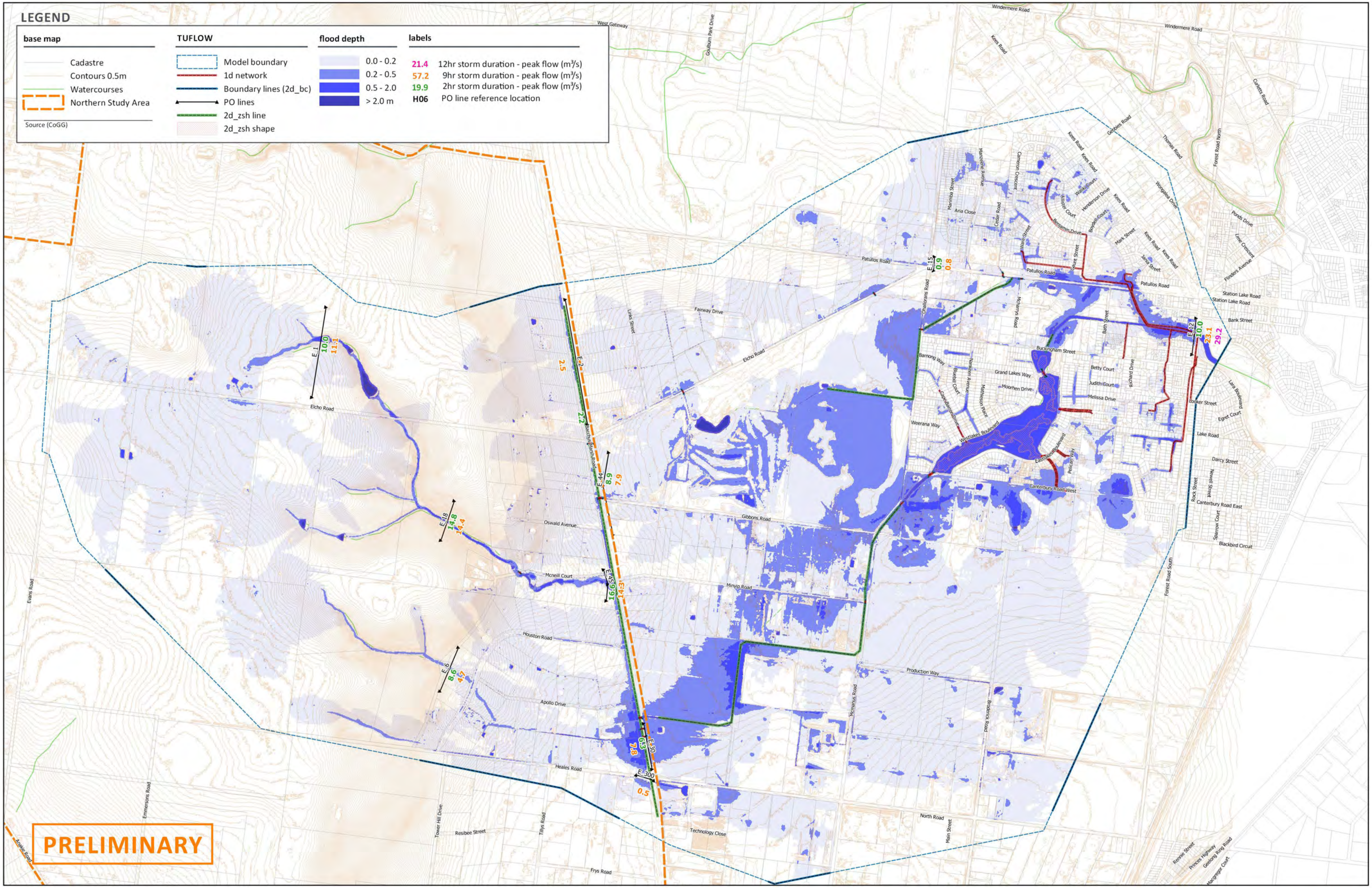
MAP SCALE 1:20,000
PAGE SIZE A3
DATUM GDA94
PROJECTION MGA Zone 55

Level 10 71 Queens Road Melbourne VIC 3004 Australia



APPENDIX G6 **ELCHO DRAIN | C157J**

EXISTING CONDITIONS | INDICATIVE 1 IN 100 AEP FLOOD EXTENT

Northern Geelong Growth Areas | Flood Impact Assessment

client: City of Greater Geelong; Lovely Banks

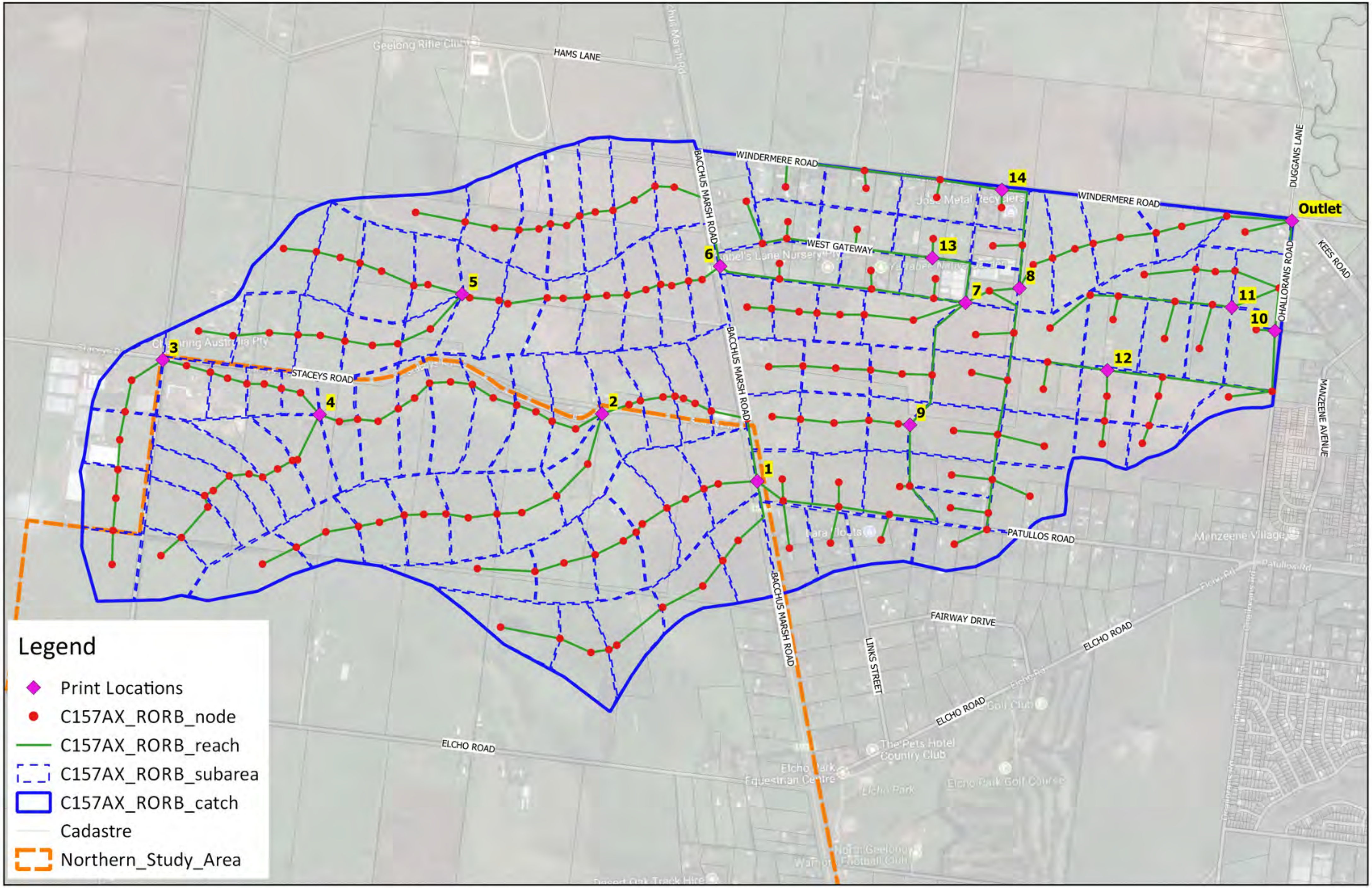
consultant: SMEC — Member of the Surbana Jurong Group

PROJECT NO	30042177W
DATE	21 Nov 2017
REVISION	C
CREATED BY	Velasco, Karl

MAP SCALE 1:20,000
PAGE SIZE A3
DATUM GDA94
PROJECTION MGA Zone 55

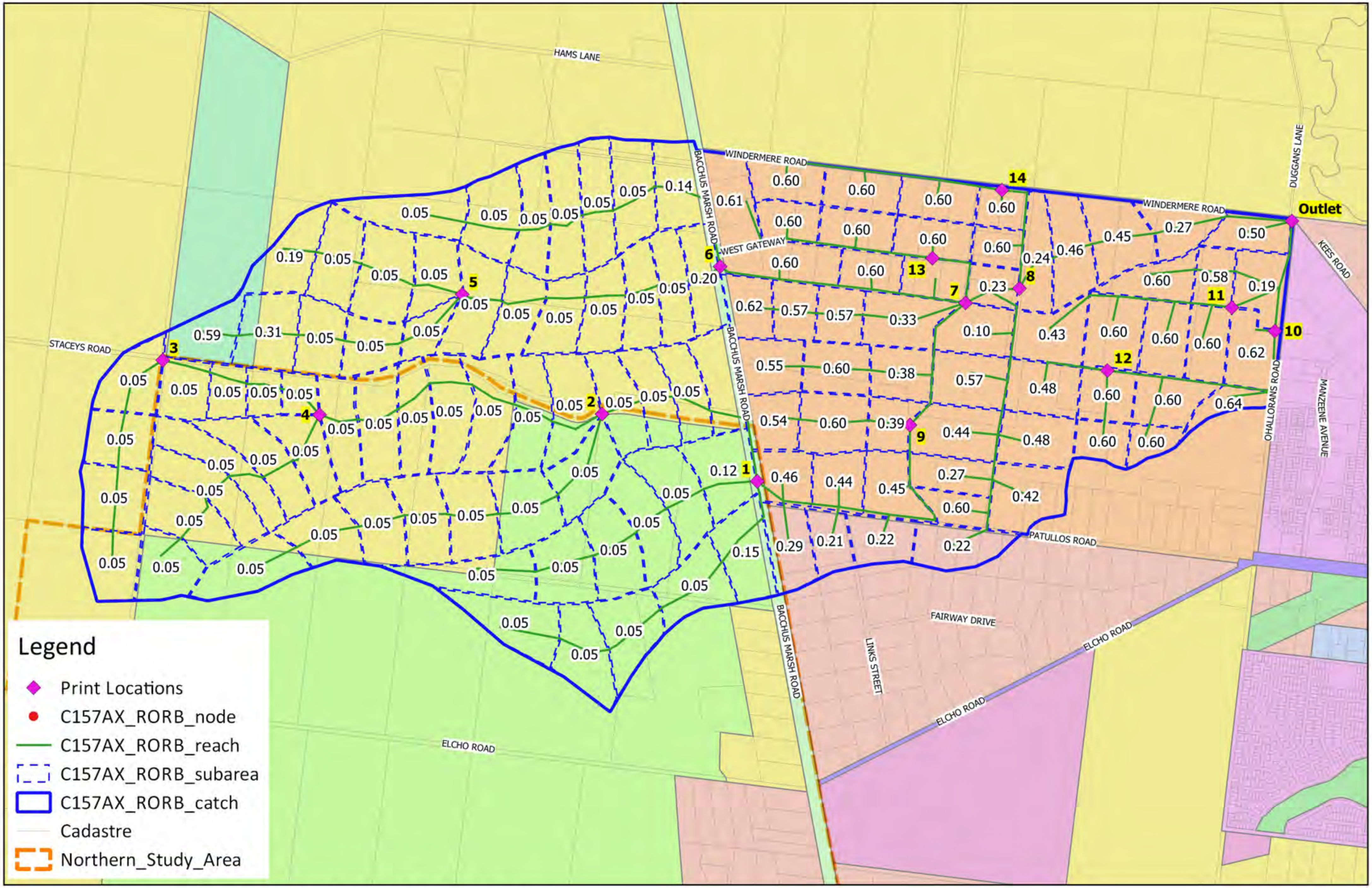
Level 10 71 Queens Road Melbourne VIC 3004 Australia

APPENDIX H HOVELLS CREEK FIGURES AND MAPS



APPENDIX H1 **HOVELLS CREEK | C157AX**

RORB MODEL



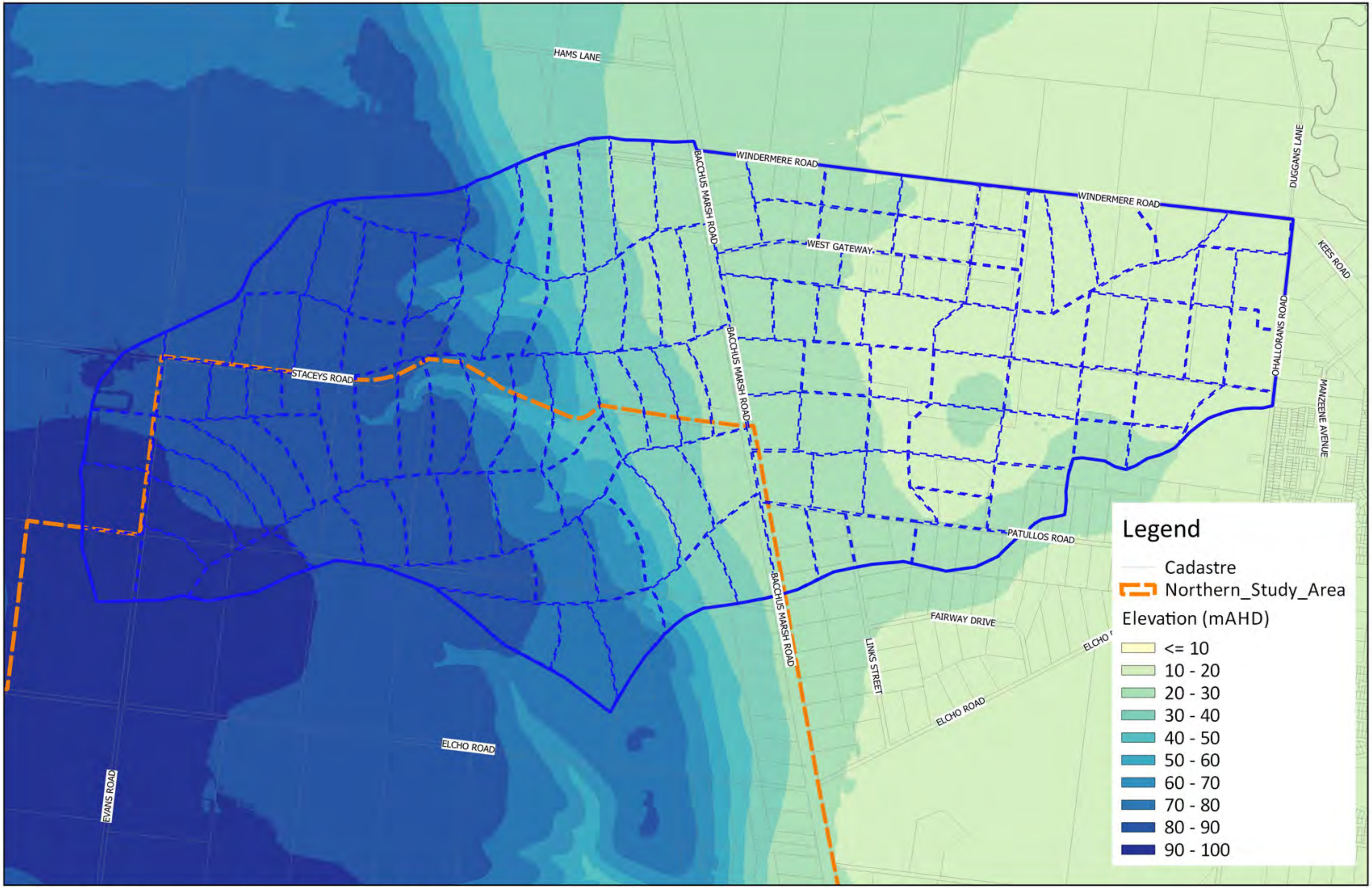
APPENDIX H2 **HOVELLS CREEK | C157AX**

RORB SUBAREA | EXISTING CONDITIONS FRACTION IMPERVIOUS

APPENDIX H3 - Vicroads Rational Method vs RORB Undeveloped Model Flow Comparison

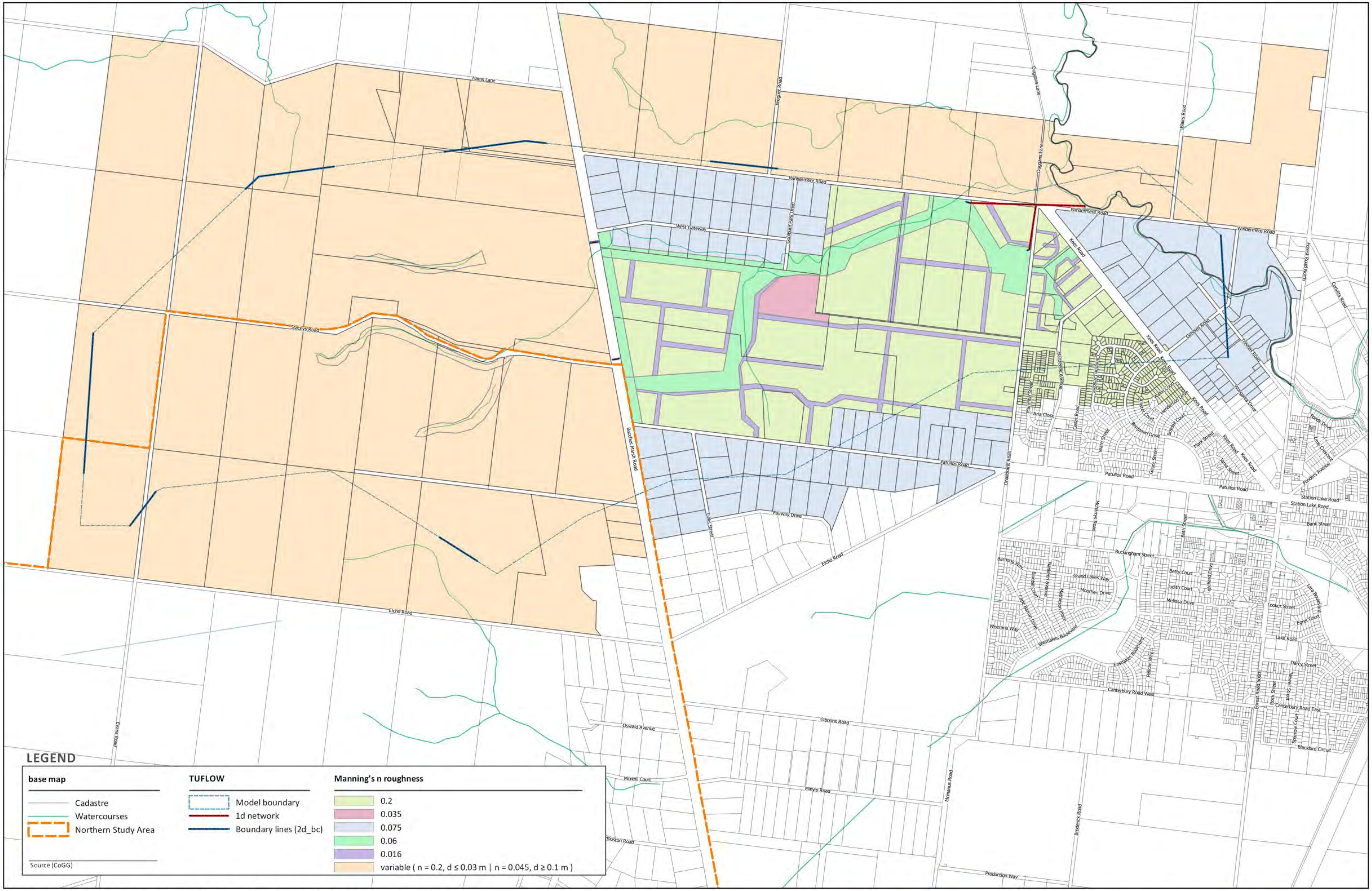
HOVELLS CREEK C157AX - 1 IN 100 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	3.8	1.3	0.12	1.3	1.91	39.3	9	13.2	6.9	12.4	-11
2	2.3	1.0	0.12	1.3	1.99	45.6	3	9.1		9.0	-19
3	0.3	0.5	0.12	1.3	2.09	74	2	2.0		2.0	-25
4	1.0	0.8	0.12	1.3	2.05	55	2	5.6		4.9	-11
5	0.7	0.7	0.12	1.3	2.07	61	2	4.3		3.8	-13
6	1.9	1.0	0.12	1.3	2.01	48.3	3	8.1	3.8	8.0	-19
7	7.1	1.6	0.12	1.3	1.75	34.7	9	22.0		18.6	-2
8	8.0	1.7	0.12	1.3	1.70	34.7	9	24.5		20.5	1
9	4.2	1.3	0.12	1.3	1.89	39.3	9	13.4		13.5	-17
10	0.4	0.5	0.12	1.3	2.08	68	3	1.7		2.5	-44
11	0.3	0.5	0.12	1.3	2.09	74	2	2.1		2.0	-17
12	0.2	0.4	0.12	1.3	2.09	81	1	1.7		1.5	-10
13	0.3	0.5	0.12	1.3	2.09	74	2	1.4		2.0	-41
14	0.2	0.4	0.12	1.3	2.09	81	2	1.2		1.5	-32
Outlet	9.3	1.8	0.12	1.3	1.63	31.3	9	26.6	14.5	20.7	7

HOVELLS CREEK C157AX - 1 IN 5 AEP FLOWS											
Catchment Location	Catchment Area (Km^2)	Time of Concentration (hr)	C10	FY	FA	Intensity (mm/hr)	RORB Critical Duration (hr)	RORB Peak Flow (m^3/s)	RFFE (m^3/s)	Rational Peak Flow (m^3/s)	Difference (%)
1	3.8	1.3	0.12	0.9	1.91	20	12	5.1	2.3	4.4	-7
2	2.3	1.0	0.12	0.9	1.99	23.1	12	3.4		3.2	-8
3	0.3	0.5	0.12	0.9	2.09	36.5	9	0.7		0.7	-18
4	1.0	0.8	0.12	0.9	2.05	27.6	9	2.1		1.7	-5
5	0.7	0.7	0.12	0.9	2.07	30.5	9	1.6		1.3	-7
6	1.9	1.0	0.12	0.9	2.01	24.4	9	2.9	1.3	2.8	-10
7	7.1	1.6	0.12	0.9	1.75	17.7	12	8.3		6.6	-2
8	8.0	1.7	0.12	0.9	1.70	17.7	12	9.3		7.2	0
9	4.2	1.3	0.12	0.9	1.89	20	12	5.1		4.8	-17
10	0.4	0.5	0.12	0.9	2.08	33.8	9	0.6		0.8	-34
11	0.3	0.5	0.12	0.9	2.09	36.5	9	0.8		0.7	-11
12	0.2	0.4	0.12	0.9	2.09	39.7	9	0.5		0.5	-8
13	0.3	0.5	0.12	0.9	2.09	36.5	9	0.5		0.7	-40
14	0.2	0.4	0.12	0.9	2.09	39.7	9	0.5		0.5	-26
Outlet	9.3	1.8	0.12	0.9	1.63	16	12	10.0	4.9	7.3	4



APPENDIX H4 **HOVELLS CREEK | C157AX**

ELEVATION



APPENDIX H5 **HOVELLS CREEK | C157AX**

EXISTING CONDITIONS | ROUGHNESS PARAMETER

Northern Geelong Growth Area | Flood Impact Assessment

client CITY OF GREATER GEELONG Lovely Banks

consultant SMEC Member of the Surbana Jurong Group

PROJECT NO 30042177W
DATE 27 Jul 2017
REVISION C
CREATED BY Velasco, Karl

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DATUM GDA94
PROJECTION MGA Zone 55

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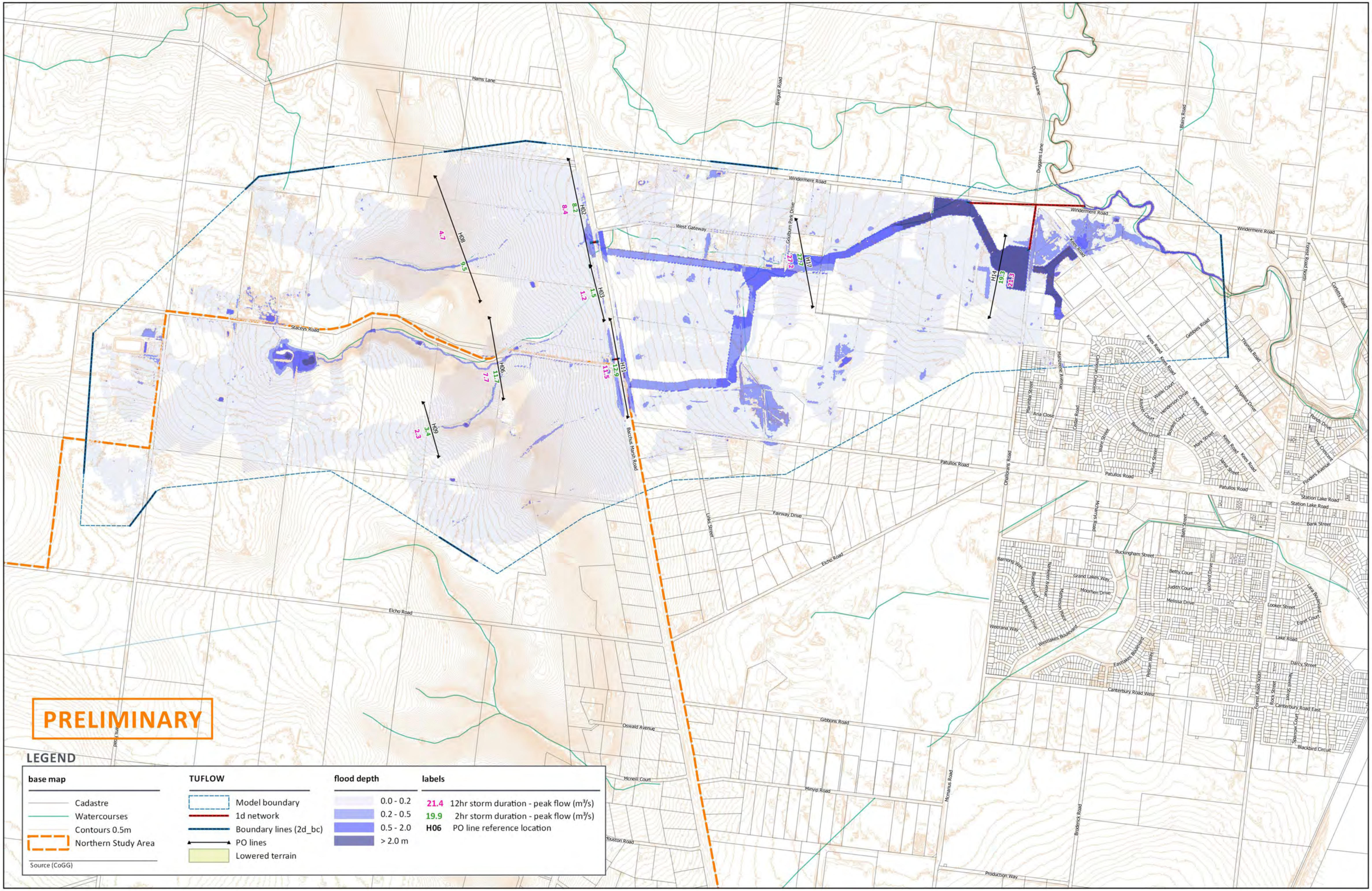

APPENDIX H6 HOVELLS CREEK | C157AX

EXISTING CONDITIONS | INDICATIVE 1 IN 100 AEP FLOOD EXTENT

Northern Geelong Growth Area | Flood Impact Assessment

client: CITY OF GREATER GEELONG; Lovely Banks

consultant: SMEC, Member of the Surbana Jurong Group

PROJECT NO 30042177W
DATE 21 Nov 2017
REVISION C
CREATED BY Velasco, Karl

MAP SCALE 1:20,000
PAGE SIZE A3
DATUM GDA94
PROJECTION MGA Zone 55

Level 10 71 Queens Road Melbourne VIC 3004 Australia

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Project Name:	NGGA Flood Impact Assessment and Stormwater Management Strategy
Project Number:	30042177W
Revision Number:	Final Rev4

Revision History

Revision #	Date	Prepared by	Reviewed by	Approved for Issue by
Draft	2/11/16	T.Rhodes		
Final	11/1/17	T.Rhodes	S.van Hall	T.Rhodes
Revision 1	28/7/17	T.Rhodes	S.van Hall	T.Rhodes
Revision 2	21/11/17	T.Rhodes	S.van Hall	T.Rhodes
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Revision 4	18/09/18	T.Rhodes / K.Velasco	S.van Hall	T.Rhodes

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SMEC Project File	18/09/18	1 Electronic

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