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Newcomb - Whittington Drainage / Flood Study Final Report

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Newcomb - Whittington Drainage/Flood Study Final Report

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Author(s) :	Michael South and Joel Leister
Synopsis :	This report documents the methodology and results for the Newcomb – Whittington Drainage / Flood Study.

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EXECUTIVE SUMMARY

Study Objective

The Newcomb – Whittington drainage catchments has been identified as having a known problem with drainage related or “stormwater” flooding. The City of Greater Geelong has received numerous reports of flooding problems within the Newcomb – Whittington drainage catchments. The majority of the problems relate to the characteristics of the catchments (flat and low lying topography, development density and age). The old (pre 1980’s) drainage system typically has limited capacity in the underground pipes and a lack of clear overland flowpaths for surcharge and excess flows. This is further impacted by the outlet channels being of limited capacity and subject to coastal inundation from tide and storm surge events. The primary objectives of this study were to characterise existing flooding and to develop an appropriate flood management strategy to mitigate stormwater flooding in the area.

Study Methodology

The study was carried out under the following core elements.

1. *Preliminary Tasks* – These were project initiation, including an inception meeting and initial site inspection, along with a data collation and review exercise. The data collation and review phase included an analysis of previous drainage investigations, council policy, aerial photography of the area, topography, GIS datasets, digital plans and design information.

2. *Digital Terrain Model* – Photogrammetry and LiDAR data of the study area was provided by the City of Greater Geelong (CoGG) and used to assist in the hydrological model development. Additional continuous elevation strings representing features of hydraulic importance (such as retarding basin crests) were sourced for use in the modelling.

3. *Hydrological and Hydraulic Modelling, and Mapping of the Existing Conditions* – The hydrologic and hydraulic modelling was undertaken using the traditional approach of applying flow boundaries from the hydrological model (RORB) to the two-dimensional (2D) hydraulic model (TUFLOW). The existing flood characteristics were identified through hydrologic and hydraulic modelling of the 20%, 10%, 5% and 1% Average Exceedance Probability (AEP) flood events. The flood results were mapped using GIS. An assessment of flood damage was undertaken using the stage-damage curve approach.

4. *Mitigation Option Assessment and Mapping* – A wide range of potential structural and non-structural flood mitigation measures were screened, from which a shortlist of three (3) alternative flood mitigation schemes were selected with two of these schemes tested using the hydraulic model. Flood damage, scheme cost and benefit-cost ratios were determined for each of the schemes tested. A „do nothing’ option was also considered.

5. *Selection and Detailed Mapping of the Preferred Mitigation Scheme* – The mitigation schemes were assessed according to their ability to reduce flood damage. The schemes were ranked according to a range of economic and non-economic factors. A preferred strategy was then selected in consultation with CoGG. The preferred scheme was mapped using GIS, with hardcopy plans of flood extent and flood levels produced.

The key results from the investigation are summarised in the following sections.

Existing Flooding Characteristics

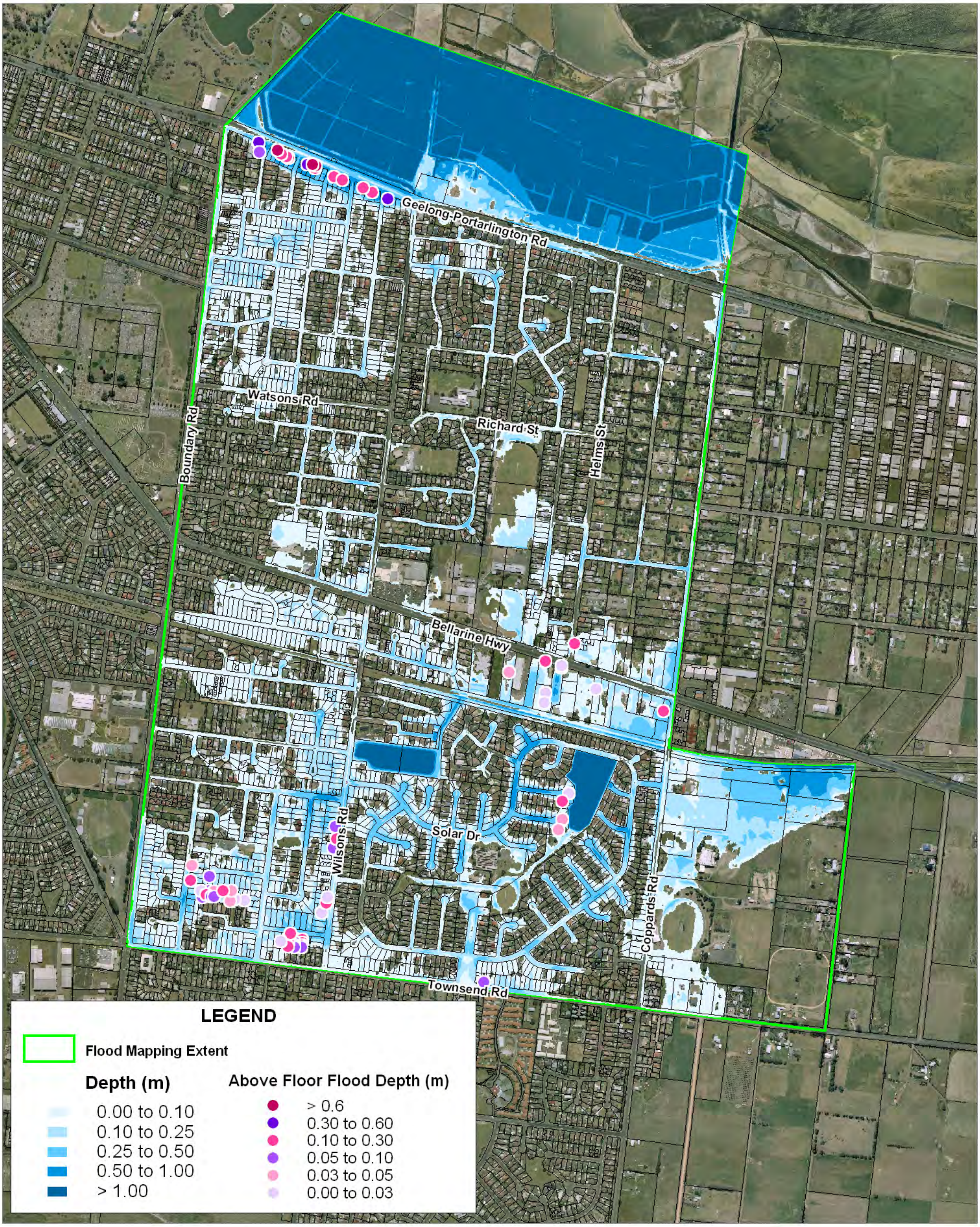
The flood extent of the 1% Annual Exceedance Probability (AEP) flood, i.e. the 100 Year Average Recurrence Interval (ARI) flood, is shown in Figure E-1. The number of flood-affected properties was identified and the average annual flood damage (AAD) was calculated at $178,000. Table E-1 shows the total number of properties that have floor level information available and are inundated to above floor level in the range of flood events analysed. An analysis was also undertaken to determine the number of properties within the study mapping area, ie, not the full catchments, that have flooding within the property boundaries in the 1% AEP event. This information is also detailed in Table E-1.

Table E-1 Number of Flooded Properties

AEP	Number of Flooded Properties - Existing Conditions	
	Within Property	Above Floor*
20%	1011	19
10%	1656	33
5%	2162	44
1%	2696	65

* Results based on properties surveyed by CoGG.

Hazard mapping was undertaken using the methodology prescribed in the Melbourne Water document *Guidelines for Development in Flood-prone Areas* (Melbourne Water 2008). The analysis is designed to determine if it is safe for people to move about on a property during a flood event. Safety is defined in terms of the depth, velocity and velocity-depth product. The existing 1% AEP hazard mapping for the study area is shown in Figure E-2. As expected, the majority of the main overland flow through the catchments is classified as safe in a 1% AEP event. This is due to the relatively flat nature of the catchments resulting in flows of low velocity. Properties located on Geelong - Portarlington Road west of Wilsons Road have been deemed unsafe. This is a result of the adopted Tail Water Level being higher than the elevation of many of these low lying properties, thus leaving these properties inundated in the modelling. Elsewhere within the residential area, unsafe flooding is contained within the road reserves. In addition to these main overland flow paths, Wilsons Road and Solar Drive Retarding Basins and the storage created by the Bellarine Highway east of Coppards Road, are classified as unsafe in a 1% AEP flood.



Title:

Existing Conditions 1% AEP Peak Flood Depth

Figure: **E-1**

Rev: **E**

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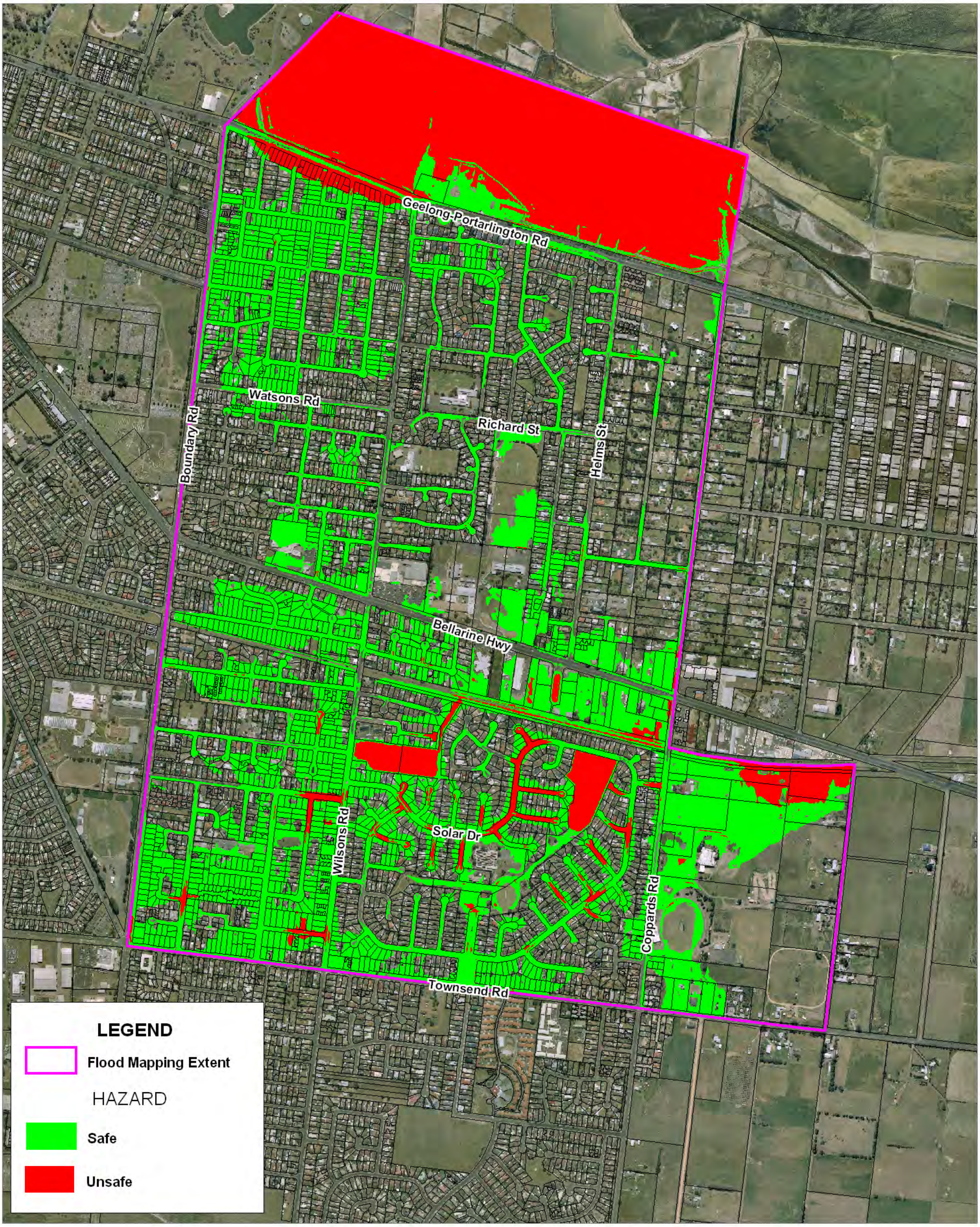

Title:

Existing Conditions 1% AEP Peak Flood Hazard

Figure: E-2

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Mitigation Option Assessment

A full range of structural and non-structural flood mitigation elements were considered when developing the three mitigation schemes. The elements considered ranged from upgraded underground pipe systems through to planning scheme amendments and education and awareness programs. These elements were screened to provide a list of elements that were considered suitable for use in the Newcomb – Whittington drainage catchments. Through discussion with Council officers, the elements were combined to form the mitigation schemes for detailed modelling and assessment. The „do nothing' strategy, i.e., the existing flood conditions, was also considered.

Schemes One and Two were assessed using the hydraulic model for each flood event, however no hydraulic assessment was undertaken for Scheme Three as it was deemed unfeasible. Table E-2 shows the number of flooded properties under each scenario assessed. Table E-3 outlines the benefit (as a result of reduced flooding), the capital and on-going costs and Benefit to Cost Ratio (BCR) of Schemes One and Two.

Table E-2 Flood Affected Properties

Option	Flood Affected Property Floors *			
	20% AEP	10% AEP	5% AEP	1% AEP
Existing Conditions	**19**	**33**	**44**	**65**
Scheme One	16	17	18	25
Scheme Two	16	19	24	45
Scheme Three (Not Hydraulically Assessed)				
Scheme Four (Do Nothing)	**19**	**33**	**44**	**65**

* Flood Affected Property Floors are defined as those with flood levels above the surveyed floor level.

Table E-3 Mitigation Option Economic Summary

Options	Annual Damages	Average Annual Benefit	Total Benefit (NPV)*	Capital Cost	Ongoing Costs over 30 Years (PA)	Ongoing Costs over 30 Years (NPV)*	Total Option Cost	BCR
Scheme One	$134,000	$44,000	$546,000	$3,392,000	$81,000	$1,005,000	$4,397,000	0.12
Scheme Two	$144,000	$34,000	$422,000	$44,846,000	$1,076,000	$13,352,000	$58,198,000	0.01
Scheme Four	$178,000							

* NPV – Net Present Value discounted at 7% over 30 years

Preferred Mitigation Scheme

Scheme One is the preferred mitigation scheme for the Newcomb - Whittington drainage catchments. Details of the scheme are shown in Figure E-3. Through consultation with the CoGG, the preferred scheme was selected as it focused on mitigation of overland flows and flood depths to reduce flood hazard within the urban areas of the study area, in particular the area between the intersection of Wilsons Road and Hickey Street, and the southwest corner of the study area. The impact of the preferred scheme on 1% AEP flood levels is shown in Figure E-4, and the hazard mapping for the preferred scheme is shown in Figure E-5. A comparison with the existing case hazard map (Figure E-2) shows a reduction in the extent of the areas classified as unsafe.

Also considered in the selection of the preferred scheme were key indicators such as environmental, social, feasibility and performance. This analysis in regard to the preferred scheme, when compared to the other schemes, is summarised as follows:

- its potential environmental impact is minimal and similar to the other scheme;
- the social impacts arising from Scheme One are very limited and similar to the other scheme; and
- the feasibility and performance are far greater than achieved in Scheme Two.

The overall flood management strategy recommended in the Newcomb – Whittington Drainage/Flood Study comprises the following:

- Construction of Scheme 1.
- Further investigation into the feasibility of localised flood mitigation measures, such as lifting footpaths and/or underground drainage augmentation, in order to reduce the flood risk for properties that would not benefit from Scheme 1 but have been the subject of previous drainage/flooding requests for service that have identified capacity deficiencies rather than blockages. These investigations would be undertaken in accordance with the priority ranking established by applying the City's prioritisation method for drainage investigation/design work.
- Further investigation into the feasibility of property-specific measures to manage risk from residual flooding with Scheme 1 in place. Potential measures include flood-proofing of individual or groups of buildings/properties by landowners, and property buy-back with on-sell following modifications (where feasible) with conditions known to purchaser.
- Education and awareness program to inform landowners how to minimise the magnitude of damage in a flood event.
- Development controls via designation of areas as liable to flooding in accordance with Building Regulations 2006 and use of flood zones/overlays within the Greater Geelong Planning Scheme.
- Recognition that further development within the catchments has the potential to increase flood risk to people and property. Assessment of rezoning proposals to include application of principle of zero adverse flood impact on adjacent, upstream and downstream areas. Assessment of development and subdivision applications to include application of best practice guidelines for development within or upstream of flood-prone areas.
- Best practice environmental management for stormwater runoff to be encouraged as part of development and subdivision applications in order to reduce runoff and improve water quality, where not a statutory requirement.

Title:

Preferred Scheme (Mitigation Scheme One) Proposed Mitigation Works

Figure: E-3

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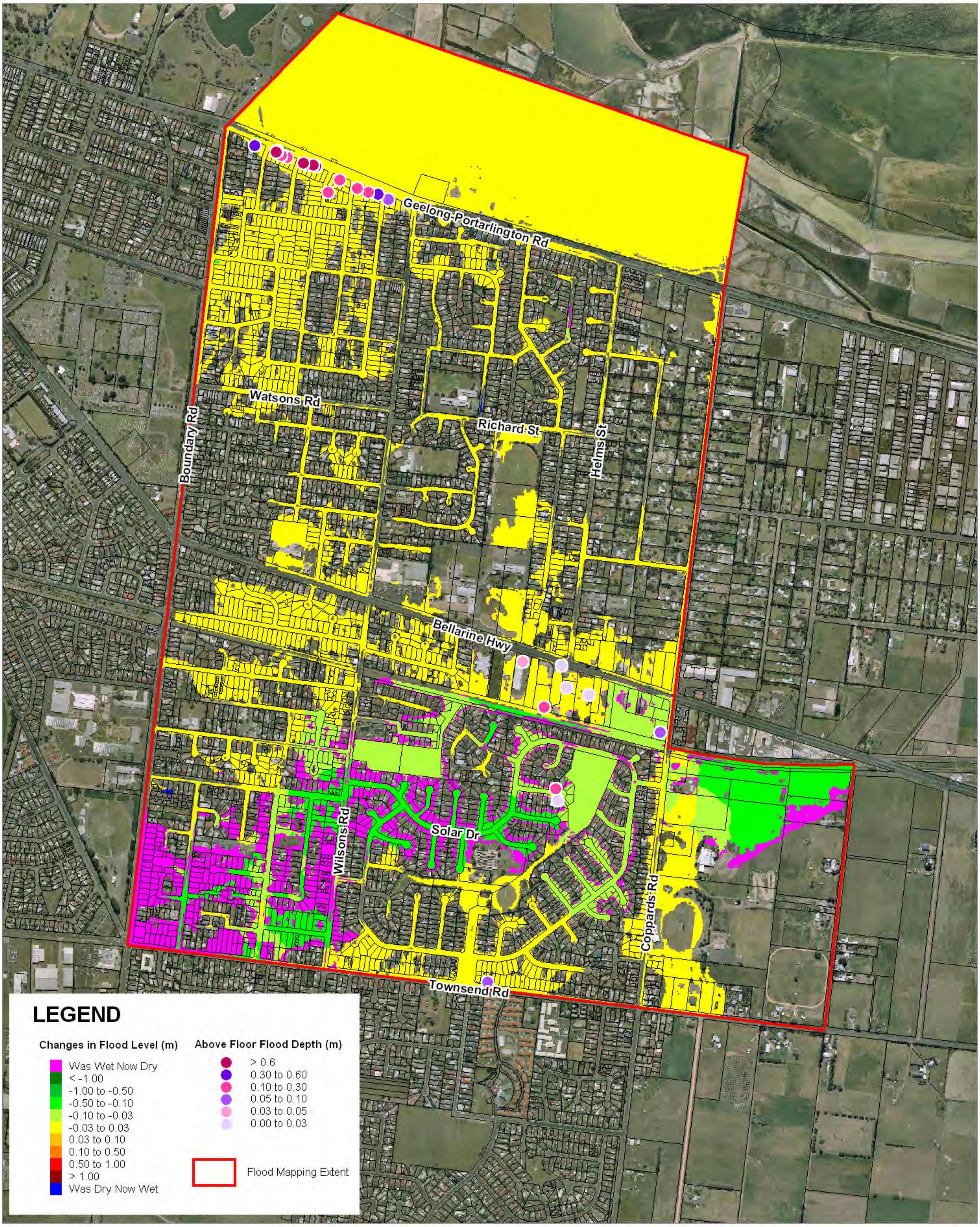

Title:

Preferred Scheme (Mitigation Scheme One) 1% AEP Peak Flood Impact

Figure: E-4

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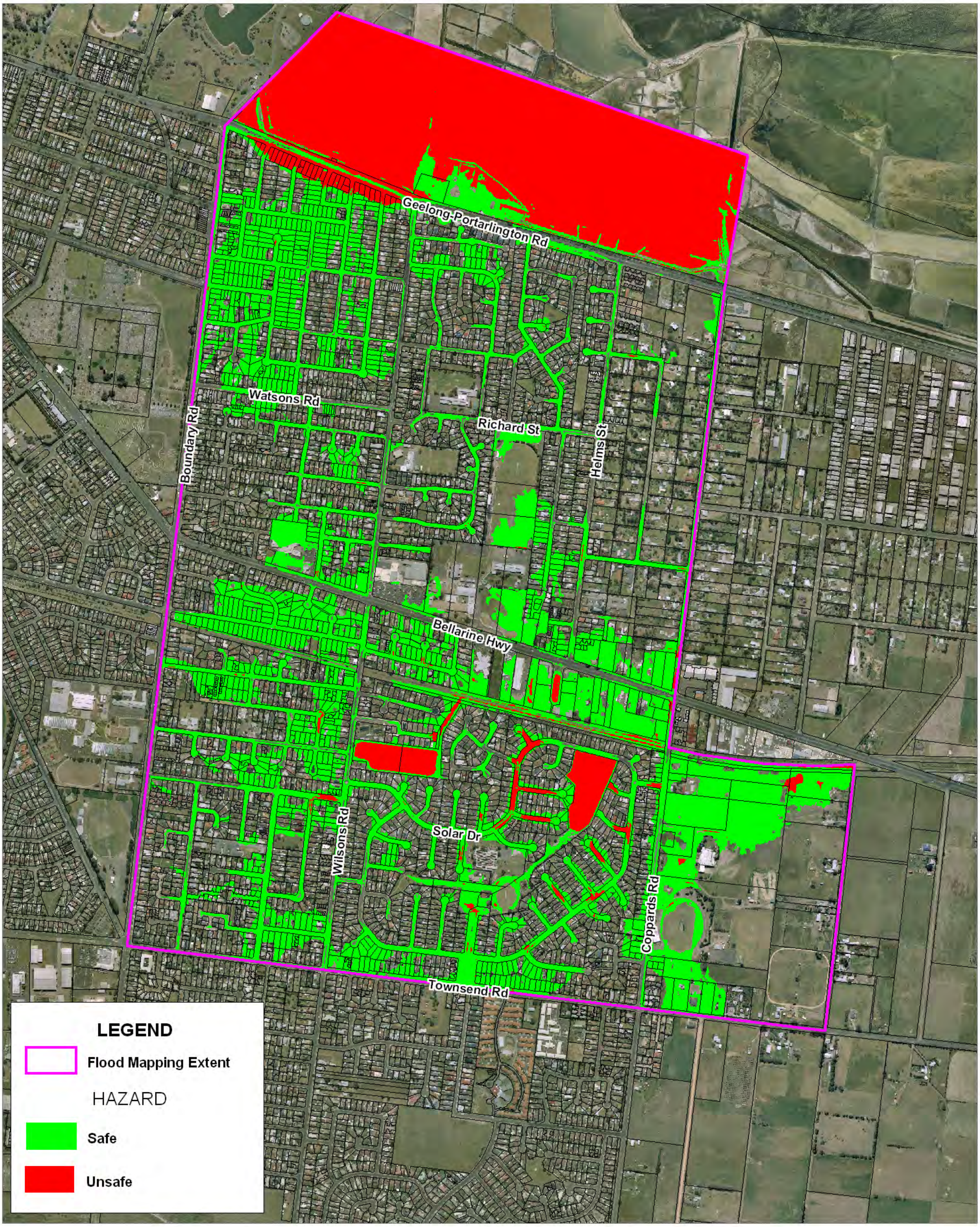

Title:

Preferred Scheme (Mitigation Scheme One)
1% AEP Peak Flood Hazard

Figure: **E-5**

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1 INTRODUCTION

1.1 Background

The City of Greater Geelong (CoGG) has engaged BMT WBM Pty Ltd (WBM) to undertake an assessment of stormwater flooding within the Newcomb - Whittington drainage catchments (C20 to C25, C31 and a portion of C33), and to investigate flood management options to manage and minimise the effects of flooding on the community.

1.2 Catchment Description

The Newcomb - Whittington drainage catchments are located approximately 3km east of Geelong's Central Activities Area and on the south side of the Geelong – Portarlington Road (Figure 1-1). The catchments drain approximately 480 hectares of predominantly urbanised land into Stingaree Bay via constructed channels through the Cheetham Salt Works. Parts of the catchments have not been fully urbanised and remain Low Density Residential Zones due the drainage constraints of the catchments and history of flooding in the region.

The relatively flat grades present in the catchments have resulted in floodwaters having a tendency to remain in the catchments for extended periods after the rainfall event. This is further exacerbated by the limited hydraulic capacity of the outfall/outlet drainage system through the Cheetham Salt Works.

Throughout the catchments, many of the natural flow paths have been rendered ineffective due to the positioning of the streets in a grid style layout. Consequently, floodwaters have been noted to build-up behind road and rail embankments. Parts of the Newcomb – Whittington drainage catchments were subdivided at a time when servicing of infrastructure was not required as part of the sub-division process. Infrastructure was implemented through „user pays' schemes and hence, there are still areas within the catchments where drainage and street construction are yet to be implemented.

A feature of the Newcomb – Whittington drainage catchments is the two types of catchments that exist; 'minor system' catchments defined by areas commanded by the underground or minor drainage system and 'major system' catchments defined by areas generating runoff and overland flows or major system flows in excess of the capacity of the underground system. These two types of catchments are not necessarily coincidental throughout the study area, e.g. for Whittington, the minor system catchments drain to the north whilst the partially overlapping major system catchments drain to the east in accordance with natural drainage paths.

The existing drainage network is generally undersized and therefore contributes to localised „stormwater' flooding. Like many drainage systems in areas of a similar age, the Newcomb-Whittington's drainage network has a limited capacity for the catchments it is draining. The Newcomb-Whittington catchments are drained by a series of underground pipes, open channels and the natural overland flow paths. Two retarding basins have also been constructed within the catchments to help alleviate local flooding issues.

1.3 History of Flooding

Council has received numerous reports of flooding problems within the Newcomb – Whittington drainage catchments. The majority of the problems relate to the characteristics of the catchments (flat and low lying topography, development density and age). The old (pre 1980's) drainage system typically has limited capacity in the underground pipes and a lack of clear overland flowpaths for surcharge and excess flows. This is further impacted by the outlet channels being of limited capacity and subject to coastal inundation from tide and storm surge events.

Recent flooding has been recorded in the Newcomb – Whittington drainage catchments in April 2001 and February 2005 as a result of significant, long duration rainfall events.

1.4 Study Area

The Newcomb - Whittington study area is detailed in Figure 1-2. The study area is defined as the outer limit to the contributing area to the Newcomb – Whittington drainage catchments. The study area is also limited to Stingaree Bay, the discharge point for these catchments. The study area is modelled in detail using both hydrologic and complex two-dimensional hydraulic models to simulate the flood behaviour within the catchments. The area that is to be flood mapped is also shown in Figure 1-2. The area mapped within the Whittington, SE Newcomb and Coppards Road can be considered to be an extension of the Moolap Industrial Area Flood Management Plan (2004), which examined the downstream area.

1.5 Key Objectives

The key objectives of this study are to:

1. develop a Digital Terrain Model (DTM) of the study catchments from digital data captured by Qasco and supplied by the City of Greater Geelong;

2. determine the flood extents, depths and associated hazard of the critical 20%, 10%, 5% and 1% annual exceedance probability (AEP) flood events through the use of hydrologic and hydraulic models for existing conditions;

3. identify and assess potential mitigation strategies to reduce damages associated with flooding;

4. determine the flood extents, depths and associated hazard of the critical 20%, 10%,5% and 1% annual exceedance probability (AEP) flood events through the use of hydrologic and hydraulic models for the preferred mitigation strategy; and

5. produce a report and flood maps detailing the methodology and results from the above four tasks.

Title:

Locality Map

Figure: 1-1

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Approx. Scale

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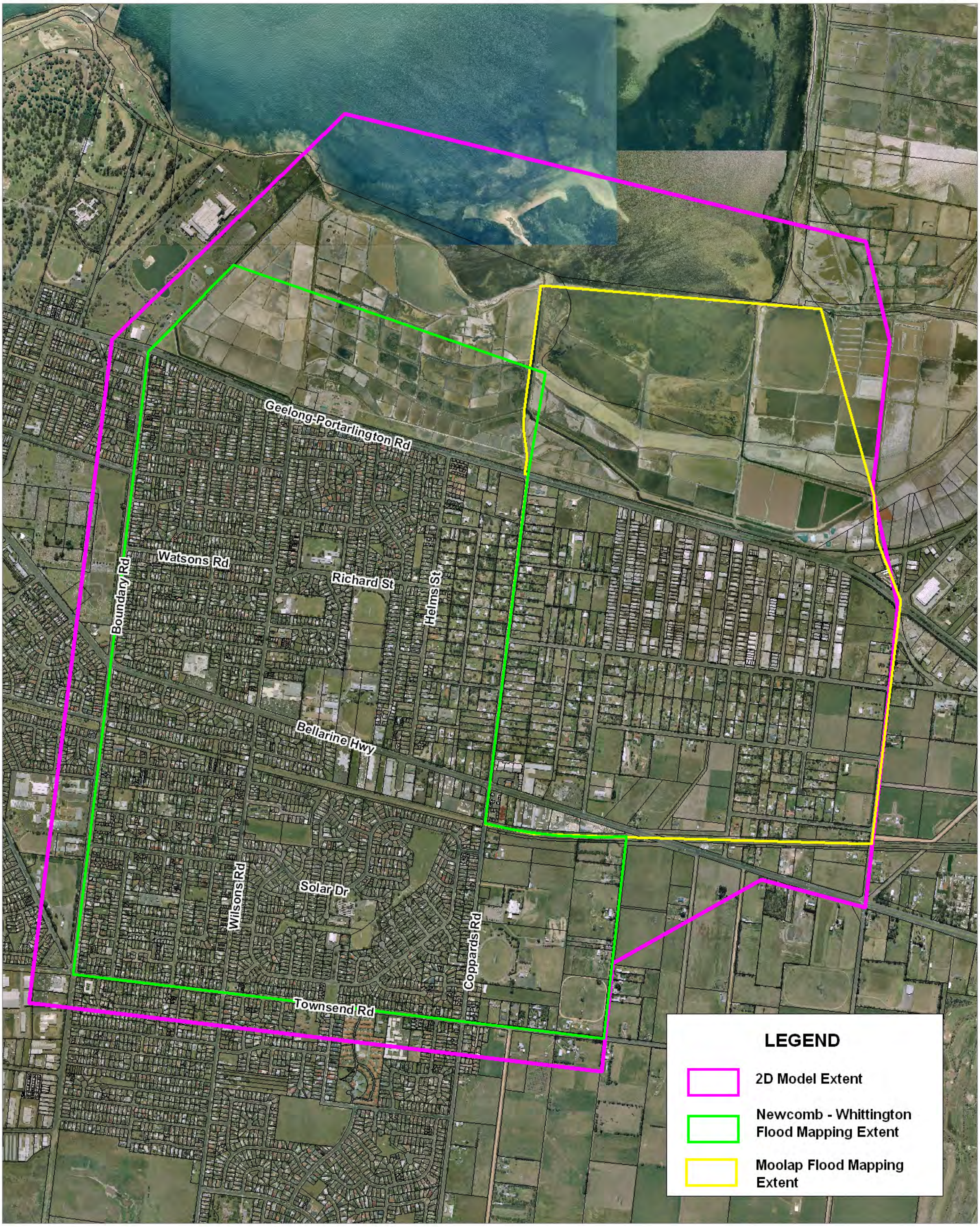

Title:

Study Area and Flood Mapping Limit

Figure: 1-2

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2 STUDY APPROACH

There were six key stages in the study as follows:

- Data collection;
- Flood model development;
- Flood mapping;
- Flood damages assessment;
- Mitigation options assessment; and
- Reporting.

The remainder of Section 2 outlines the adopted approach for each of these stages. A detailed description of the key stages is given in subsequent sections of the report.

2.1 Data Collation

2.1.1 Study Inception and Site Visit

Following commissioning of the study, an inception meeting and site visit were held between representatives from CoGG and BMT WBM. The principal objectives were to confirm the project approach, obtain key and relevant data and to discuss known flooding issues within the catchments.

2.1.2 Drainage, Topographic and GIS Data Sets

All relevant data for the drainage systems was obtained from the CoGG. The data was comprehensively reviewed to identify any significant data gaps and to gain a complete understanding of issues in the study area. Where required, field survey was commissioned to address gaps in available data.

2.2 Flood Model Development

The flood model was developed using the traditional approach of utilising hydrologic and hydraulic computer models. The hydrologic model determines the runoff hydrographs that occur following a particular rainfall event. The hydrographs describe the quantity, rate and timing of the runoff that results from rainfall events. These hydrographs then become a key input into the hydraulic model. The hydraulic model simulates the movement of floodwaters through overland flow paths, storage areas, and hydraulic structures. The hydraulic model calculates flood levels and flow patterns and also models the complex interactions between overland flow paths and underground drainage.

The hydrologic modelling of the catchments was undertaken using RORB. A new RORB hydrological model of the catchments was developed for this flood study. As discussed further in Section 4.1.1.7, although two retarding basins exist within the Newcomb - Whittington drainage catchments, they are included in the RORB model for completeness only. The hydraulic model is used to simulate the behaviour of the retarding basins during a flood event. No calibration data was available for the hydrological model, so for most parameters, typical values appropriate for the catchments characteristics were adopted and the model was then verified against the Rational Method and previous studies within the catchments. The adopted loss model was an initial loss/volumetric runoff coefficient model.

Hydraulic modelling of the catchments was undertaken using the 2D/1D dynamic hydraulic modelling package TUFLOW. The model incorporated both the overland flow paths and the underground trunk drainage system. No data was available for calibration so typical parameter values based on experience and the data collected during the site inspections were applied. TUFLOW was run as an unsteady flow model to ensure reliable representation of the storage within the system and the complex timing and interaction of flows in the drainage network.

2.3 Flood Mapping

Flood maps showing flood extent, depth and height were produced for each design flood analysed. Design floods are hypothetical floods used for planning and floodplain management investigations. A design flood is defined by its probability of occurrence. It represents a flood that has a particular probability of being exceeded in any one year. For example, the 1% Annual Exceedance Probability (AEP) or 100 year Average Recurrence Interval (ARI) flood is a best estimate of a flood magnitude which has 1 chance in 100 of being exceeded in any one year. It should be noted that planning for the 1% AEP flood does not guarantee protection for the next 100 years. Design flood levels were determined for the 20%, 10%, 5% and 1% AEP floods.

2.4 Flood Damage Assessment

The design floods were used to make an assessment of the financial losses to residential properties and public infrastructure. These financial losses were then used as a basis to do an economic assessment of potential mitigation options.

2.5 Mitigation Options Assessment

A range of options designed to mitigate the existing flood impact and associated damages were considered and analysed for effectiveness. The economic impacts of each scenario, as well as a range of non-economic factors, were compared in order to ascertain the most suitable outcome.

2.6 Reporting

Several meetings were held with CoGG during the course of the study to present findings before proceeding to subsequent stages. The findings of the study are presented in this Flood Study Report.

3 DATA COLLATION

3.1 Site Inspection

Following commissioning of the study, an inception meeting was held between CoGG representatives and BMT WBM project staff. During this meeting background data was supplied, project documentation was exchanged and the scope of works was discussed and approved. This meeting was followed by a site inspection with Council's representative, where flooding issues throughout the study area were outlined and viewed.

3.2 GIS Data

All relevant data for the drainage systems was obtained from the CoGG. The data was comprehensively reviewed to identify any significant data gaps and to gain a complete understanding of issues in the study area.

Additional project related GIS data was sourced from Council's GIS system. In particular, the following data were supplied:

- cadastral information over the study area;
- planning scheme zones over the catchments; and
- aerial photography.

3.3 Drainage Data

CoGG supplied drainage network data for the catchments in digital formats (as GIS datasets and pdf documents). Data included pipe networks (location and size), open channel locations and pit locations. Pipe data gaps were identified and in-filled where possible by interpolation of inverts from upstream and downstream. A survey brief was issued to assist in filling in the remaining missing data.

3.4 Topographic Data

CoGG provided WBM with new aerial photogrammetry for the area to be flood mapped. The quoted vertical accuracy of this data is +/- 100mm and is considered suitable for the hydraulic modelling of the catchments.

The data was supplied in AutoCAD (DWG) format, and was subsequently converted to a Triangulated Integrated Network (TIN), which was imported into MapInfo Professional. The resulting DEM is shown in Figure 3-1.

Within the flood mapping area the sampling resolution for the DEM is 0.5m. Based on our past experience, we have found that this level of detail is well suited to simulating the topography of urbanised environments for hydraulic modelling.

CoGG also provided WBM with topographic data from LiDAR flown over the entire municipality's developed areas. The vertical accuracy of the LiDAR data was +/- 150mm, and was not considered as suitable as the photogrammetry for the purposes of the hydraulic modelling. However, this data was used in the development of the hydrologic model due to the wider coverage of the catchments.

The LiDAR data was supplied in csv format, and was subsequently converted to a Triangulated Integrated Network (TIN) using Vertical Mapper, with the resultant file imported into MapInfo Professional. The resulting DEM is shown in Figure 3-2.

Outside the flood mapping area, the sampling resolution for the DEM is 1.0m. Based on our past experience, we have found that this level of detail is well suited to the development of the hydrologic model.

3.5 Survey Data

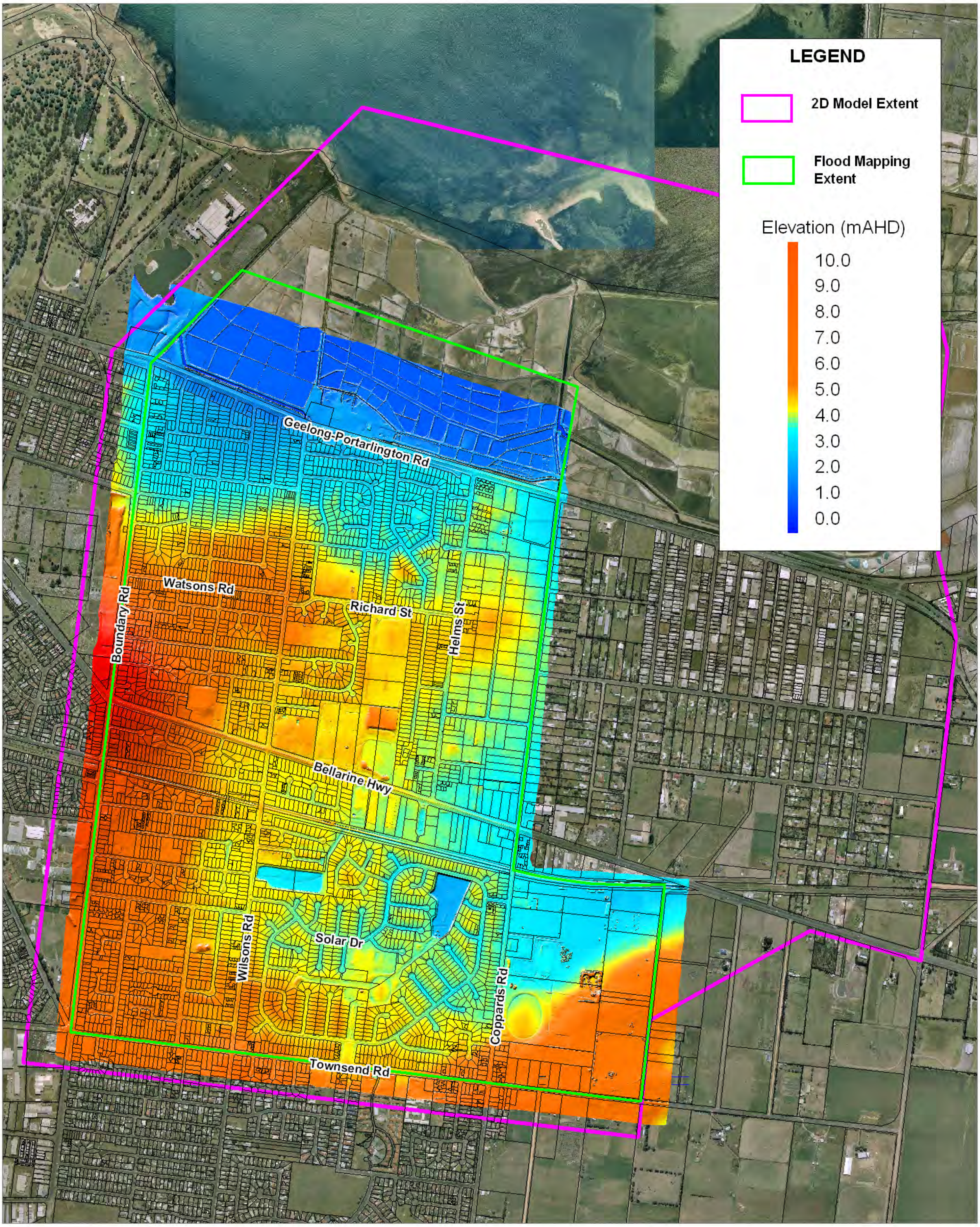
Following a review of the data provided by the CoGG it was determined that additional field data would be required to fill missing pipe data, confirm the validity of the photogrammetry and capture additional topographic data not present in the DEM. TGM Group Pty Ltd were commissioned on the 16th March 2010 to carry out the required field survey.

TGM Group Pty Ltd supplied the requested field survey data on the 21st of April, 2010 in the form of an AutoCAD drawing with an accompanying comma separated value (CSV) text file and site photos.

Pipe invert levels at 29 pit locations, where either upstream or downstream invert levels were missing, were surveyed. The inverts at these locations were either unable to be interpolated using the methods described in Section 4.2.2.2 or were considered critical to the completion of the pipe network within the hydraulic model.

Stringlines were taken at the spillway crests of the two formal retarding basins within the catchments. The shape and elevation of the spillways is critical to accurately defining the amount of flood storage and flow from the retarding basins. Accordingly, the stringlines were incorporated into the hydraulic model.

Spot heights were also taken along a selection of road centre lines throughout the study area in order to confirm the accuracy of the topographic data provided. The elevations at each point were compared to the levels from the Photogrammetry DEM. The differences in elevation were then reviewed to see how strongly the data sets agreed. Elevation differences within +/- 100mm were considered reasonable given the inherent accuracy of the two survey methods. Some values outside the range are acceptable within the stated tolerances of the photogrammetry (i.e. 1 sigma of points within +/- 100mm). The comparisons showed strong agreement and the photogrammetry DEM was considered appropriate for the purposes of the study. Figure 3-3 illustrates the results of the checks.



Title:

Newcomb - Whittington Photogrammetry Digital Elevation Model

Figure: 3-1

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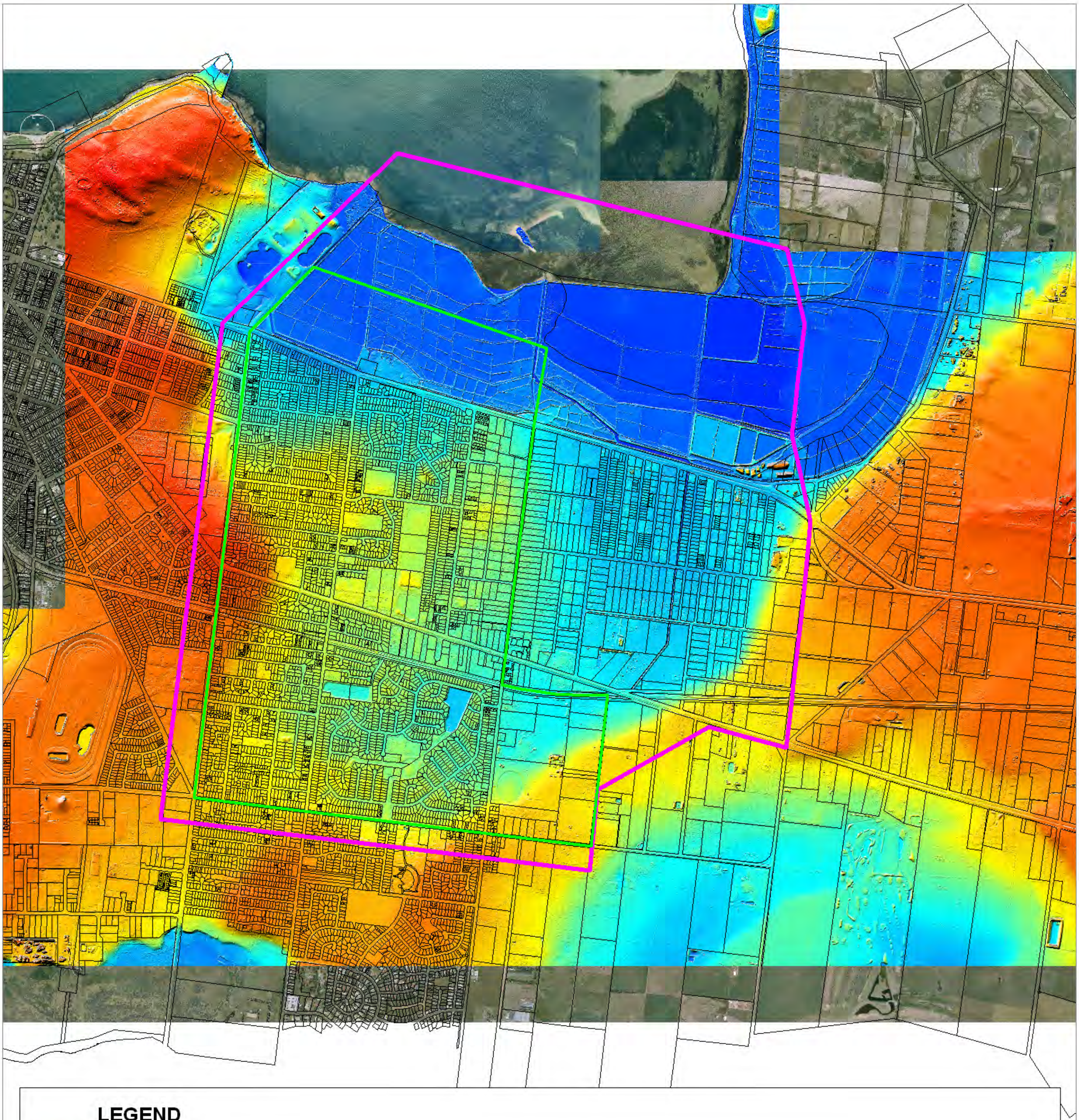

Title:

Newcomb - Whittington
LiDAR Digital Elevation Model

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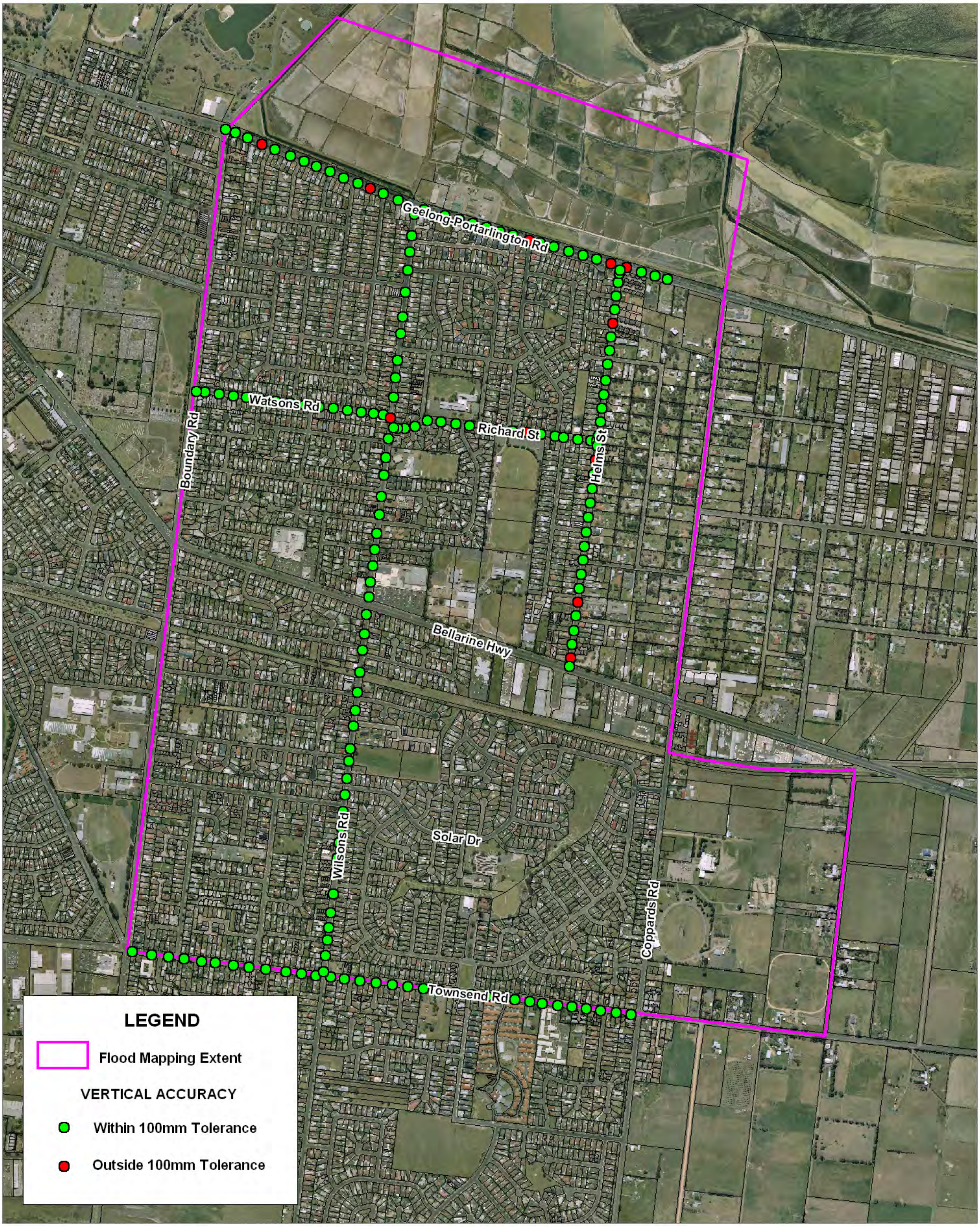

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Photogrammetry DEM Check

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3-3

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4 FLOOD MODEL DEVELOPMENT

4.1 Hydrologic Model

Hydrologic modelling of the study catchments was undertaken using RORB. A RORB model had previously been developed for the catchments as part of the Moolap Industrial Area Flood Management Study (WBM 2004). This previously developed RORB model was refined to ensure it could meet the requirements of the current study and subsequently provide total and sub-area hydrographs as boundary conditions for the TUFLOW hydraulic model. The RORB modelling process and results are discussed in the following sections.

4.1.1 RORB Model

RORB simulates the linkages between sub-catchments as reach storages with the storage discharge relationship defined by the following equation;

$$S = 3600kQ^m$$

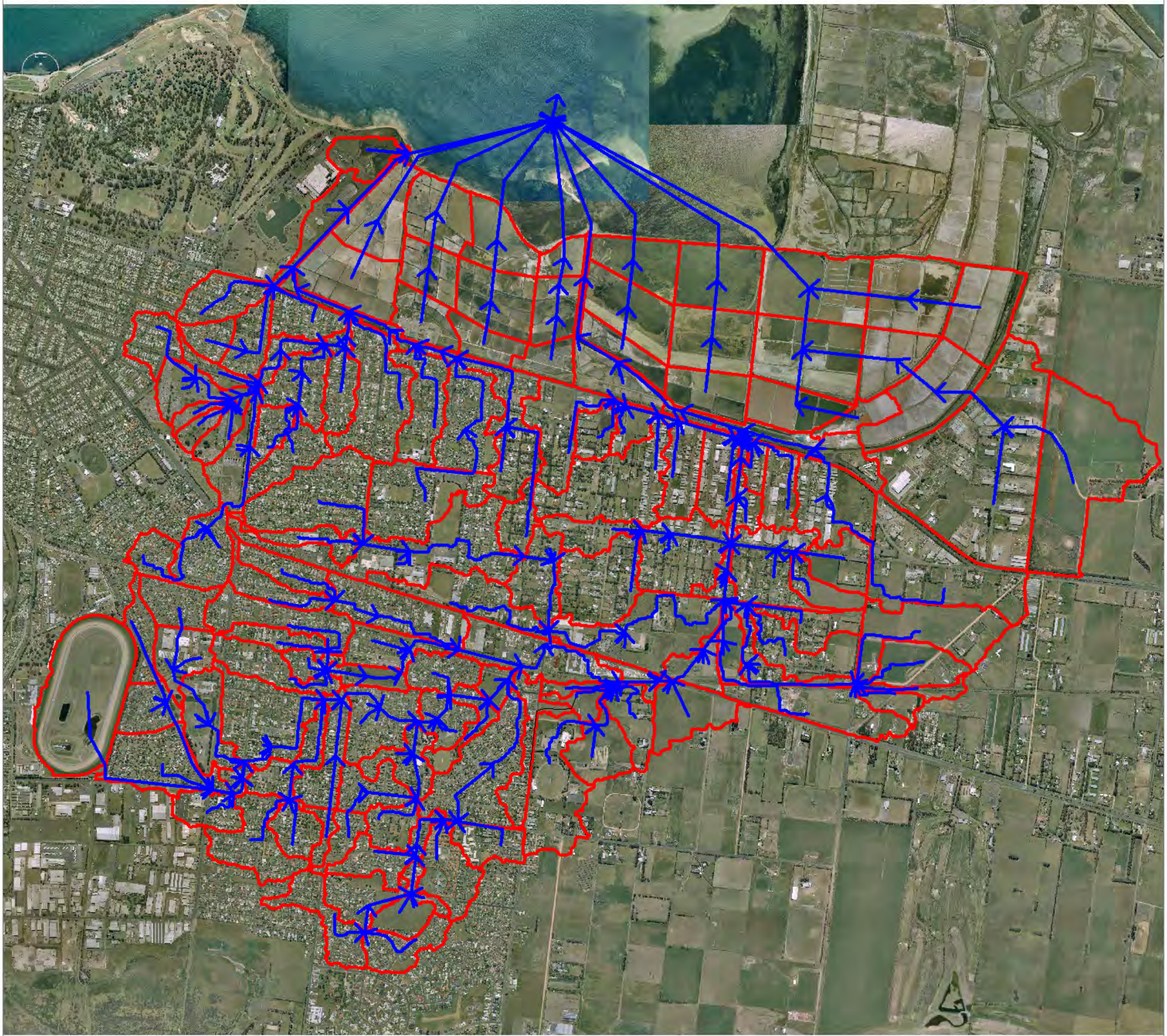
Where „s' represents the storage (m^3), „Q' is the discharge (m^3/s), „m' is a dimensionless exponent and „k' is non-dimensional empirical coefficient. „k' is defined by the product of the catchment value „k_c'„ and the individual reach k_i. Both m and k_c are defined as calibration parameters. Based on industry best practice, in the absence of calibration events, „m' of 0.8 was adopted.

4.1.1.1 Model Description

The RORB model incorporates an area of approximately 14.13 square kilometres, including the entire Newcomb-Whittington study area (and previously modelled Moolap Industrial Estate). To ensure accurate representation of the overall catchments, the model was divided into 148 individual sub-catchments. Conceptual reaches (approximate overland flow paths) were defined and storage relationships were included for the existing retarding basins within the catchments. The schematic RORB layout is shown in Figure 4-1

4.1.1.2 Sub-Catchment Definition

The catchment and sub-catchment boundaries were determined using the software package CatchmentSIM, and were refined based on the LiDAR topographic data and photogrammetry (supplied by CoGG), CoGG drainage plans and the underground drainage network layout. The sub-catchment breakdown is shown in Figure 4-1.



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Sub-Catchment Boundary

Drainage Path

Title:

RORB Catchment Layout

Figure: 4-1

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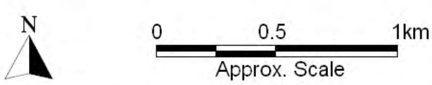

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4.1.1.3 Global Parameters

RORB model parameters for the Newcomb - Whittington drainage catchments are summarised in Table 4-1 and are discussed further in Sections 4.1.1.4 and 4.1.1.5.

Table 4-1 RORB Parameters

RORB Parameter	Value
Storm Data	Newcomb-Whittington
Catchment Area	14.13 km^2
Initial Loss	15.0 mm
Volumetric Runoff Coefficient	Varies (refer to Section 4.1.1.5)
m	0.80
k_c	6.78
Fraction Impervious	Varies, as per land use (Table 4-4)
Reach Types	Type 2, 3 and 4
Peak Flow Events	Varies

4.1.1.4 IFD Parameters

Storm data was based on IFD parameters sourced from the Bureau of Meteorology, which are based on Figures 1.8 to 6.8 and 7d to 9 of Australian Rainfall and Runoff (AR&R) Volume 2. The adopted values for the catchments are presented in Table 4-2.

Table 4-2 IFD Parameters

IFD Parameter		Adopted Value
Rainfall Intensity (mm/hr)	2 Year ARI, 1 Hour Duration	18.0
	2 Year ARI, 12 Hour Duration	3.37
	2 Year ARI, 72 Hour Duration	0.9
	50 Year ARI, 1 Hour Duration	34.34
	50 Year ARI, 12 Hour Duration	6.18
	50 Year ARI, 72 Hour Duration	1.8
Skew Coefficient		0.42
Geographical Factor F2		4.28
Geographical Factor F50		14.85
Zone		1

4.1.1.5 Loss Parameters

The loss model adopted was the “initial loss/volumetric runoff coefficient” loss model. This modelling approach is consistent with previous flood studies, including the Moolap Industrial Estate Flood Study. RORB generates runoff by subtracting losses at each timestep from the rainfall occurring in that time period. The adopted initial loss was 15 mm and the runoff coefficients for pervious areas were adopted as per Melbourne Water recommendations, as shown in Table 4-3. For impervious areas, RORB has a "hardwired" initial loss of 0 mm and runoff coefficient of 0.9.

Table 4-3 Runoff Coefficients for different AEPs

AEP	Runoff Coefficient
20%	0.25
10%	0.35
5%	0.45
1%	0.60

4.1.1.6 Fraction Impervious

The fraction of the catchments that is impervious is a key input to the hydrologic modelling. Impervious fractions for various planning scheme codes were based on advice from CoGG, values contained within Councils design guidelines, inspections of aerial photographs, and values presented in Australian Rainfall and Runoff (1987). Key impervious fractions adopted are reproduced in Table 4-4.

Table 4-4 Impervious Fraction for Planning Scheme Zone

Zone	Impervious Fraction
Residential (< 300 m^2)	0.95
Residential (300 to 400 m^2)	0.6
Residential (400 to 600 m^2)	0.5
Residential (600 to 1 000 m^2)	0.42
Residential (1 000 to 2 000 m^2)	0.4
Commercial	0.95
Industrial	1
Hospitals	1
Open Space / Sports Grounds	0.15
Schools with large sports fields / Developed Parks	0.5
Schools with few fields	0.75
Major Roads	1
Minor Roads	0.8

The planning scheme data was used to establish an area-weighted average of the impervious fractions for each sub-catchment used in the hydrologic model.

4.1.1.7 Retarding Basins

Wilson Street and Solar Drive Retarding Basins are located within the TUFLOW hydraulic model extent. The 2D hydraulic model will determine the storage and discharge characteristics of the basins based upon the underlying topography and outlet structures, and therefore do not need to be included in the RORB hydrologic model. For the majority of sub-catchments within the hydraulic model extent, only local sub-area inflows are applied, not total hydrographs.

However, as the retarding basins relationships had previously been defined as part of the Moolap Industrial Estate flood modelling, these relationships have been included in the Newcomb-Whittington RORB model for completeness.

4.1.2 Hydrologic Model Verification

Due to the lack of historical rainfall and flood height data, calibration of the model to real data was not possible. Verification of results from the hydrological model was undertaken using empirical methods and the previous modelling of the catchments undertaken as part of the Moolap Industrial Estate Flood Mapping Project (WBM 2004).

The original RORB model established for the Moolap Industrial Estate Flood Model used a K_c value of 5.94. The Newcomb-Whittington RORB model had a higher D_{av} value (4.05) when compared to the Moolap RORB model (3.55) and consequently, a higher K_c was required in order to preserve the K_c/D_{av} ratio. The K_c value is a routing parameter and is therefore influenced by the average reach length of the RORB hydrological model. A K_c of 6.78 was adopted for the Newcomb-Whittington RORB model in order to maintain the K_c/D_{av} ratio from the previous modelling.

The peak outflow of the RORB model at the outlet of the Moolap Industrial Estate Drain at the Geelong-Portarlington Road was compared against a Rational Method calculation (calculated as part of the original Moolap study) and the previous RORB model results (WBM 2004).. The comparisons showed the revised model was replicating the previous modelling and therefore the adopted K_c was appropriate for these catchments.

Table 4-5 documents the peak RORB outflows from the Moolap Industrial Estate and compares them to the peak flow calculated using the Rational Method and the peak flow from the previous modelling.

Table 4-5 Comparison of Rational Method and RORB

RORB Model	Rational Method	RORB Peak Flow (m^3/s)	% Difference (to Rational Method)
Moolap Industrial Estate RORB Model	31.1	30.0	- 3.6%
Newcomb-Whittington RORB Model	31.1	31.2	0.3%

4.1.3 Design Event Modelling

4.1.3.1 Design Event Probabilities

Hydrological analysis was undertaken for the 20%, 10%, 5% and 1% Average Exceedance Probability (AEP), i.e. the 5, 10, 20 and 100 year Average Recurrence Interval (ARI) or return period, design storm events. Hydrographs were derived by RORB to provide external and internal boundary conditions to the hydraulic model at a number of locations throughout the catchments.

4.1.3.2 Design Rainfall

Intensity Frequency Duration (IFD) parameters for the Newcomb-Whittington catchments were determined from the Bureau of Meteorology using a method based on the maps from Volume Two of Australia Rainfall and Runoff (AR&R). These IFD parameters are an input to RORB and are used to generate design rainfall intensities and depths using standard AR&R procedures. The IFD parameters are presented in Table 4-2.

Filtered temporal patterns were used to derive the design storm events. Aerial Reduction Factors (ARF) were not applied due to the catchments' small size. The resulting design storms were run through the RORB model of the catchments and the results summarised to determine the critical durations.

4.1.3.3 Critical Duration Derivation

For each design probability, the peak discharge at various locations within the drainage system may be generated by storm events of different durations. Therefore, consideration of peak discharges for a range of durations is important. For example, a 2 hour duration event may result in the peak discharge in the upper portion of a catchment, while a 9 hour duration event could result in the peak discharge at the bottom of a catchment. Alternatively, the peak flood level may be more related to volume than discharge, and a high volume event may be more appropriate for consideration. Accordingly, to assess the peak discharges and volumes over the catchments, all storm durations for each AEP were modelled.

4.1.4 Peak Inflows

The hydraulic model extent covers almost the entire catchments and hence, there are no major external inflows into the hydraulic model. The peak inflows for the hydraulic model will be a series of local hydrographs that represent the local sub-catchment flows. Table 4-6 summarises the peak outflows from the hydrological model at the two locations where the catchments discharge across Geelong-Portarlington Road. The peak outflow hydrographs are also presented in Figure 4-2 and Figure 4-3.

Table 4-6 Peak Outflows

Outlet	Peak Inflow (m^3/s)			
	20% AEP	10% AEP	5% AEP	1%AEP
Australian Animal Health Laboratory Channel	3.84	5.22	7.27	12.69
Geelong – Portarlington Road (Moolap Industrial Estate Drain)	7.82	9.80	12.99	24.60

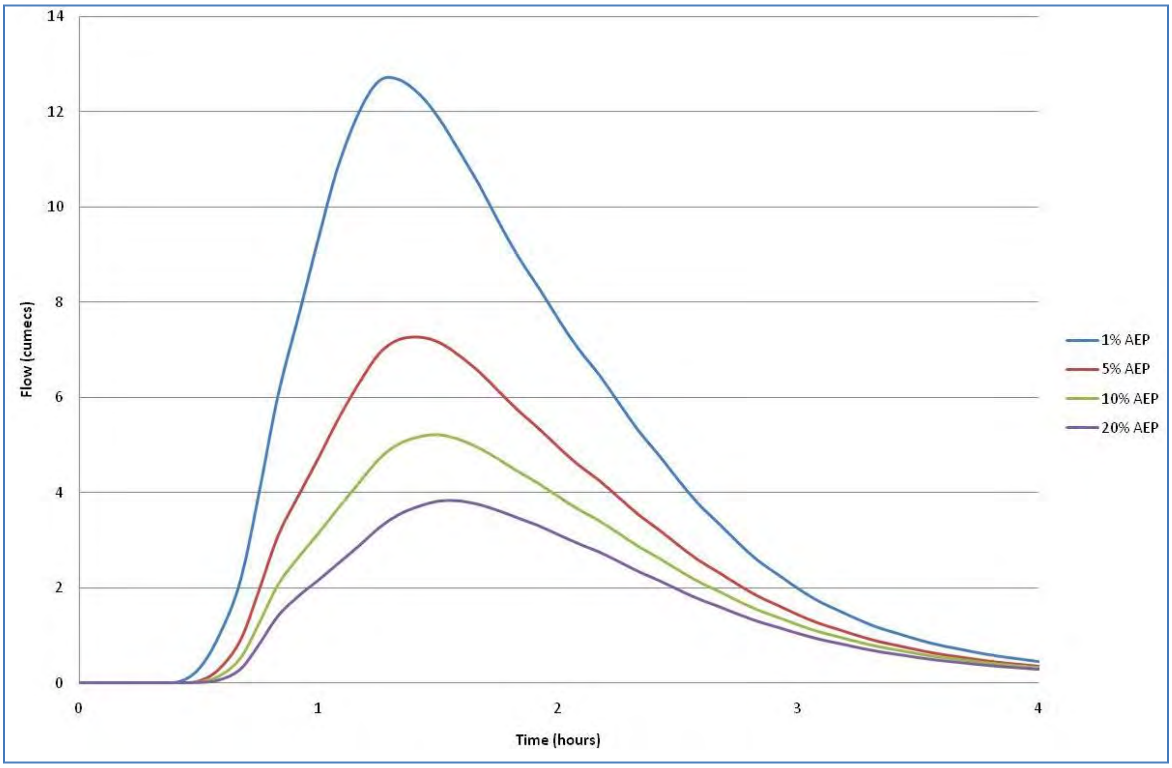


Figure 4-2 Peak Outflows at Australian Animal Health Laboratory Channel

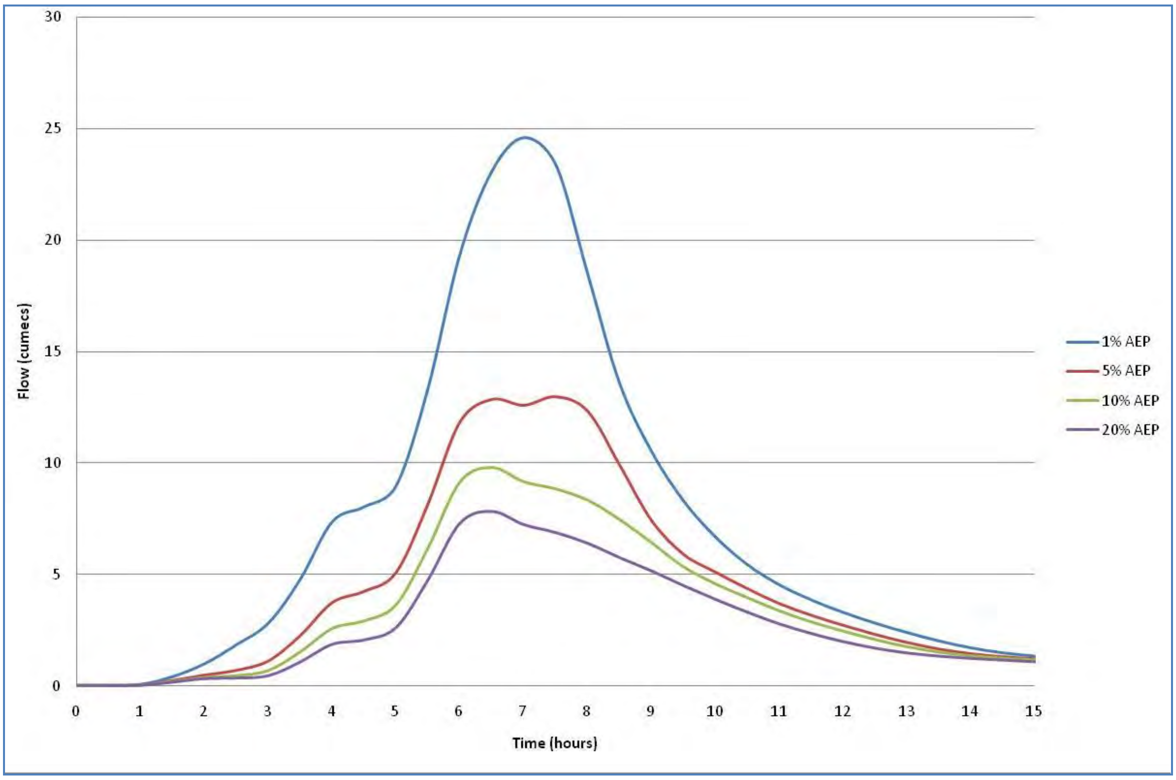


Figure 4-3 Peak Outflows at Geelong – Portarlington Road (Moolap Industrial Estate)

4.2 Hydraulic Model

TUFLOW, a fully 2D hydraulic modelling package with the ability to dynamically nest 1D elements, was adopted for this study. This TUFLOW model contains nested 1D channels representing table drains and the open drainage channels, Council's pipe network (DN300 and greater) and stormwater entry pits.

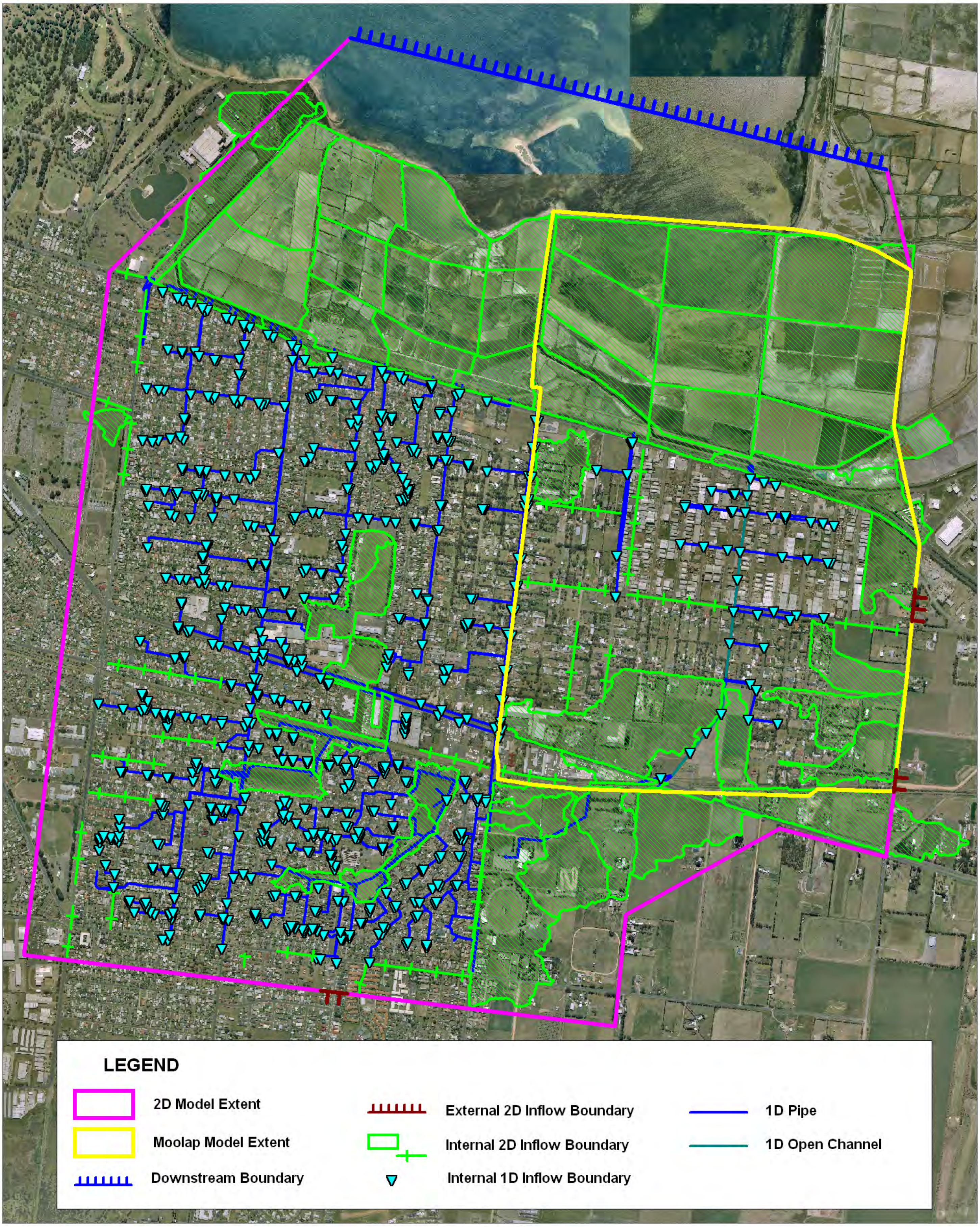
In catchments such as those at Newcomb - Whittington, where storage and timing of the rainfall inflows in the catchments are important, modelling using flow varying with time (unsteady state) rather than peak flow (steady state) is required. Accordingly, TUFLOW was run in unsteady state.

4.2.1 Model Description

The 2D model domain extends beyond the limit of the study's flood mapping region and incorporates the Moolap drainage catchment, an existing 2D model developed by BMT WBM (WBM 2004). The model covers about 11.3 square kilometres of the Newcomb - Whittington and Moolap drainage catchments (as shown in Figure 4-4). The geometry of the 2D model was established by constructing a uniform grid of square elements. One of the key considerations in establishing a 2D hydraulic model relates to the selection of an appropriate grid element size. Element size affects the resolution, or degree of accuracy, of the representation of the physical properties of the study area as well as the size of the computer model and its resulting run times. Selecting a very small grid element size will result in both higher resolution and longer model run times.

In adopting the element size for the Newcomb - Whittington drainage catchments, the above issues were considered in conjunction with the final objectives of the study. Given the size of the study area, run times could be kept to an acceptable length using a small grid element size of 3 metres across the entire model domain. As part of the detailed hydraulic modelling, a number of 1D elements were embedded into the 2D model to improve the modelling of the catchments. The modelling of these 1D elements, culvert/pipe networks, will ensure that the capacity and conveyance of these systems are accurately modelled.

Each square grid element contains information on ground topography, sampled from the DEM at 1.5 m spacing, surface resistance to flow (Manning's „n' value) and initial water level. Fifteen areas of different land-use type, determined from planning maps, aerial photography and site inspections, were identified for setting Manning's „n' values. These are summarised in Table 4-7.



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TUFLOW Model Layout

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Table 4-7 2D Domain Manning's ‚n' Coefficients

Land use	Manning's n
Low Density/Rural Residential	0.15
Residential	0.2
High Density Residential	0.4
Commercial	0.4
Industrial (low density)	0.25
Grass Maintained	0.035
Grass Unmaintained	0.05
Low Density Vegetation	0.055
Medium Density Vegetation	0.065
High Density Vegetation	0.1
Roads	0.025
Open Water	0.035
Schools	0.2
Sports Ovals	0.03
Commentary	0.03

4.2.2 Model Development

4.2.2.1 Moolap TUFLOW Model

The Moolap drainage catchment is adjacent to the Newcomb - Whittington drainage catchments on the eastern side north of the Bellarine Hwy (refer to Figure 4-4). In April of 2004 BMT WBM, formerly WBM Oceanics Australia, completed the *Moolap Industrial Area Flood Management Plan Final Report* (WBM Oceanics Australia 2004). This study included the development of a TUFLOW 2D hydraulic model.

Given the nature of the topography in the area; overland flow from the Newcomb - Whittington drainage catchments drains into the Moolap drainage catchment, particularly south and immediately north of the Bellarine Highway. As a result the Moolap model was incorporated into the Newcomb - Whittington model as the nature of flooding in the area will impact on the Newcomb - Whittington Drainage/Flood Study.

Since the Moolap model was developed there have been many changes in the way a 2D hydraulic model is setup due to advancements in the TUFLOW modelling software. In order to incorporate the Moolap model, several changes were required in order to adopt the same modelling approach within the two areas of the model.

The following alterations were made to the Moolap TUFLOW model:

- Updated grid orientation and element size to that adopted in the Newcomb - Whittington model (refer to Section 4.2.1). The topography information was then sampled from the same DEM as used in the original Moolap model.
- The fixed downstream water level was increased to 1.95 mAHD to match that adopted for the Newcomb - Whittington Drainage/Flood Study (refer to Section 4.2.3).
- The drainage pits, connectors between the 2D domain and the 1D pipe network, were updated to match the approach used in Newcomb - Whittington (refer to Section 4.2.2.3).
- The manner in which flow is applied to the model was updated to match the current modelling techniques and assumptions used in the Newcomb - Whittington model.
- North of Geelong - Portarlington Road the 1D open channel has been removed and the channel is now modelled in 2D. This is due to the open channel being entirely inundated with the higher downstream water level.
- The Manning's 'n' value in the salt ponds has been increased to 0.035 for consistency with those values adopted in the new model.

4.2.2.2 Pipe Network Setup

Pipe elements within the Newcomb - Whittington drainage area of 300mm diameter/width and above have been included as 1D elements within the model. The 1D pipe network (Figure 4-4) was developed using data supplied by the CoGG (refer to Section 3.3). There were many gaps in the supplied GIS data which required filling before the pipe network could be represented accurately within the 2D model. The steps and assumptions made to complete the 1D pipe network are detailed below:

1. Circular pipes of less than 300mm diameter and rectangular box culverts of less than 300mm width were removed from the pipe network. As result of this there was a number of "stand alone" pipes present which did not connect to the remaining pipe network. These were also removed on a case by case basis.

2. Missing pipe dimensions and inverts were filled using information from adjacent pipes.

3. Where possible missing inverts and dimensions were interpolated.

4. All plans supplied by the CoGG were reviewed to either fill missing data or to confirm the accuracy of present data.

   During this process it became evident that plans that were in imperial measurements had not had their datum converted to mAHD. As a result all inverts to the GW&ST datum were lowered by 0.487m to convert to mAHD.

   The inverts of all pipes within the area enclosed by Wilsons Road and Coppards Road to the east and west respectively and the Bellarine Rail Trail and Townsend Road to the north and south respectively, with the exception of the pipes upstream of the corner of Solar Drive and

Bean Court have been lowered by 0.512m to match the inverts shown in plan "HC_Whittington_MD_Drg_40427.pdf" which is in mAHD.

5. Following the review of the plans, remaining inverts were interpolated assuming a minimum of 450mm cover for drains within easements and 600mm for those within road reserves. A minimum of 300mm cover was assumed for the drains which run under Geelong - Portarlington Rd.

6. To fill the remaining missing inverts survey was required (refer to Section 3.5). The survey was taken at pits which would allow for a section of pipe network with missing inverts to be filled in using the previously mentioned assumptions, where appropriate.

   Pits were also surveyed along the two main drainage paths, Wilsons Road and Coppards Road to verify the accuracy of existing data and assumptions made. Along Coppards Road survey was taken from the pits at the upstream end of small inlet pipes which service the main drain. The inverts did not provide an accurate representation of the invert of the main pipe, therefore the interpolated inverts remained as previously assumed.

   Site photos captured during the field survey show that the pit on Miller Street, at the intersection with Geelong - Portarlington Road, has 3 pipes entering the pit with their obverts above ground level. The council data shows only two 525mm pipes entering this pit. These pipes have been modelled as two 525mm pipes with their obvert at ground level.

4.2.2.3 Pits

In order to connect the 1D pipe network with the 2D domain hydraulic connectors are required. For the Newcomb - Whittington study this has been achieved using pit connections. All side entry and grated entry pits that connect to pipes modelled were included in the model as connectors. Side entry pits were assumed to be 1200m wide and 150mm high kerb inlets. For double and triple side entry the width was adjusted accordingly. Grated pits were assumed to have a 900mm x 900mm square inlet.

Other connection types such as wing wall entries or exits were modelled as appropriate on a case by case basis.

All pits within the model, including sealed junction pits, which were not modelled as connections, have been assigned an appropriate form loss. These losses are based on those described in Melbourne Water's *Land Development Manual* (Melbourne Water 1998).

4.2.3 Boundary Conditions

The TUFLOW model has been developed using a fixed water level of 1.95 mAHD for the downstream boundary condition. This level was adopted based on recommendations by BMT WBM outlined in a memorandum to Richard Wojnarowski (CoGG) on the 16th of March, 2010 and approved by CoGG. The recommendations, which included a consideration of sea level rises due to climate change, were informed by a key CSIRO publication and the Victorian Coastal Strategy.

4.2.4 Design Event Modelling

The 20%, 10%, 5% and 1% AEP design storm events were modelled in the initial TUFLOW models for a wide range of storm durations. Model results were reviewed and it was determined that the 30 minute, 1 hour, 2 hour, 9 hour, 12 hour and 18 hour duration storms were critical. These storms were run for subsequent TUFLOW simulations.

The critical storm duration varied across the catchments and with AEP. In the 1% AEP event at the upstream end of the modelled pipe network and north of the Bellarine Highway in Newcomb - Whittington drainage area, the 30 minute, 1 hour and 2 hour durations were critical. In the areas surrounding the two formal retarding basins at Wilson Road and Solar Drive, the larger volume 9 hour, 12 hour and 18 hour duration storms were critical. This is also the case in the area directly south east of the Coppards Road - Bellarine Highway intersection where water ponds behind the embankment. A peak flood height envelope was developed from the six durations and the peak envelope for each AEP event was mapped. The mapping is presented in Figure 5-1 to Figure 5-4. These flood events also formed the basis of the hazard assessment as discussed in Section 5.

As no data was available to calibrate the hydraulic model, a sensibility check was undertaken by comparing the flood extents with historical flooding patterns. Preliminary flood extents for each of the design runs were provided to CoGG. CoGG reviewed the extents in the context of their experience with historical flooding problems in the study area and advised that the flooding patterns indicated by the model were consistent with their understanding of historical flooding in the catchments.

5 FLOOD MAPPING

This section provides a brief overview of the floodplain mapping process used in the investigation and the flood extent map for each of the AEP events presented.

TUFLOW produces a geo-referenced data set defining peak water levels throughout the model domain at the corners of its computational cells. For a given AEP flood event, the peak flood level from each of the storm durations was selected for each computational cell to generate an envelope of peak flood levels. These data were imported into GIS to generate a digital model of the flood surface. Contours of flood height (relative to AHD) were extracted directly from the flood surface.

The flood depth for existing conditions is mapped for the 20%, 10%, 5% and 1% AEP events in Figure 5-1 to Figure 5-4 respectively. The flooding within the Newcomb - Whittington drainage catchments is quite extensive, particularly on the southern side of the Bellarine Hwy and north of Watsons Road between Boundary Road and Wilsons Road.

The embankments of the Bellarine Hwy and the Bellarine Rail Trail cross the main flow path resulting in flood storage and thus attenuation of the flood peak.

Hazard mapping was undertaken using the methodology prescribed in the Melbourne Water document *Guidelines for Development in Flood-prone Areas* (Melbourne Water 2008). The analysis is designed to determine if it is safe for people to move about on a property during a flood event. Safety is defined in terms of the depth and velocity as follows:

- depth should be no more than 0.35m;
- velocity should be no more than 1.5 m/s: and
- the velocity depth product should be no more than 0.35m^2/s

Hazard maps of the Newcomb - Whittington drainage catchments for the four flood events are presented in Figure 5-5 to Figure 5-8. As expected, the majority of the main overland flow through the catchments is classified as safe in a 1% AEP event. This is due to the relatively flat nature of the catchments resulting in flows of low velocity. Properties located on Geelong - Portarlington Road west of Wilsons Road have been deemed unsafe. This is a result of the adopted Tail Water Level being higher than the elevation of many of these low lying properties, thus leaving these properties inundated in the modelling. Elsewhere within the residential area, unsafe flooding is contained within the road reserves. In addition to these main overland flow paths, Wilsons Road and Solar Drive Retarding Basins and the storage created the Bellarine Highway East of Coppards Road, are classified as unsafe in a 1% AEP flood.

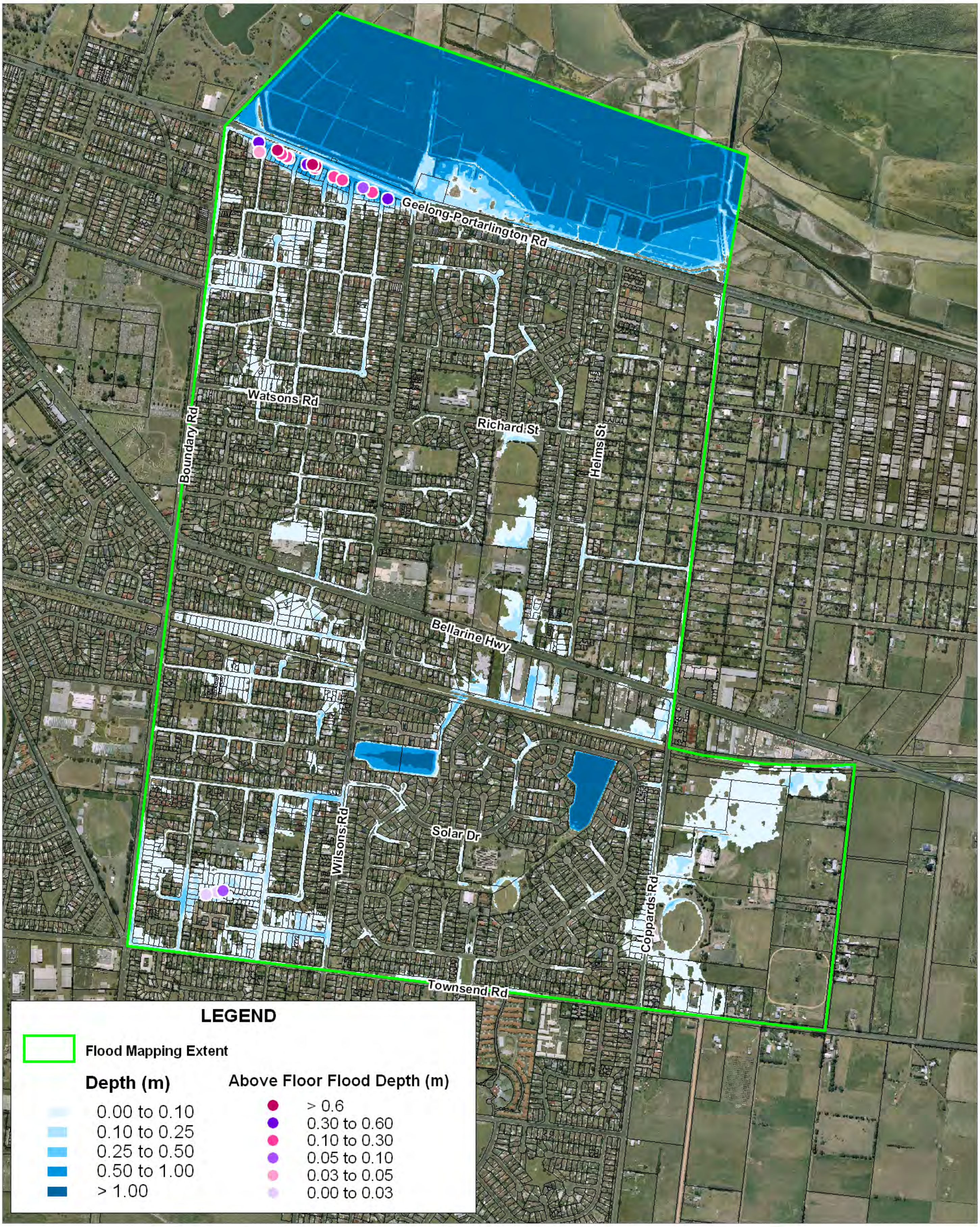
During the 20% AEP flood, areas considered unsafe are restricted to the inundated properties on Geelong - Portarlington Road and the two retarding basins.

The number of properties with flooding within the property boundaries and with above flooding above floor level has been determined and is summarised Table 5-1. The location of properties with flooding above floor for each AEP is shown in Figure 5-1 to Figure 5-4.

Table 5-1 Number of Flooded Properties

AEP	Flooding Occurs Within Property Boundary	Flooding Occurs Above Flood Level*
20%	1011	19
10%	1656	33
5%	2162	44
1%	2696	65

* Results based on properties surveyed by CoGG.



Title:

Existing Conditions 20% AEP Peak Flood Depth

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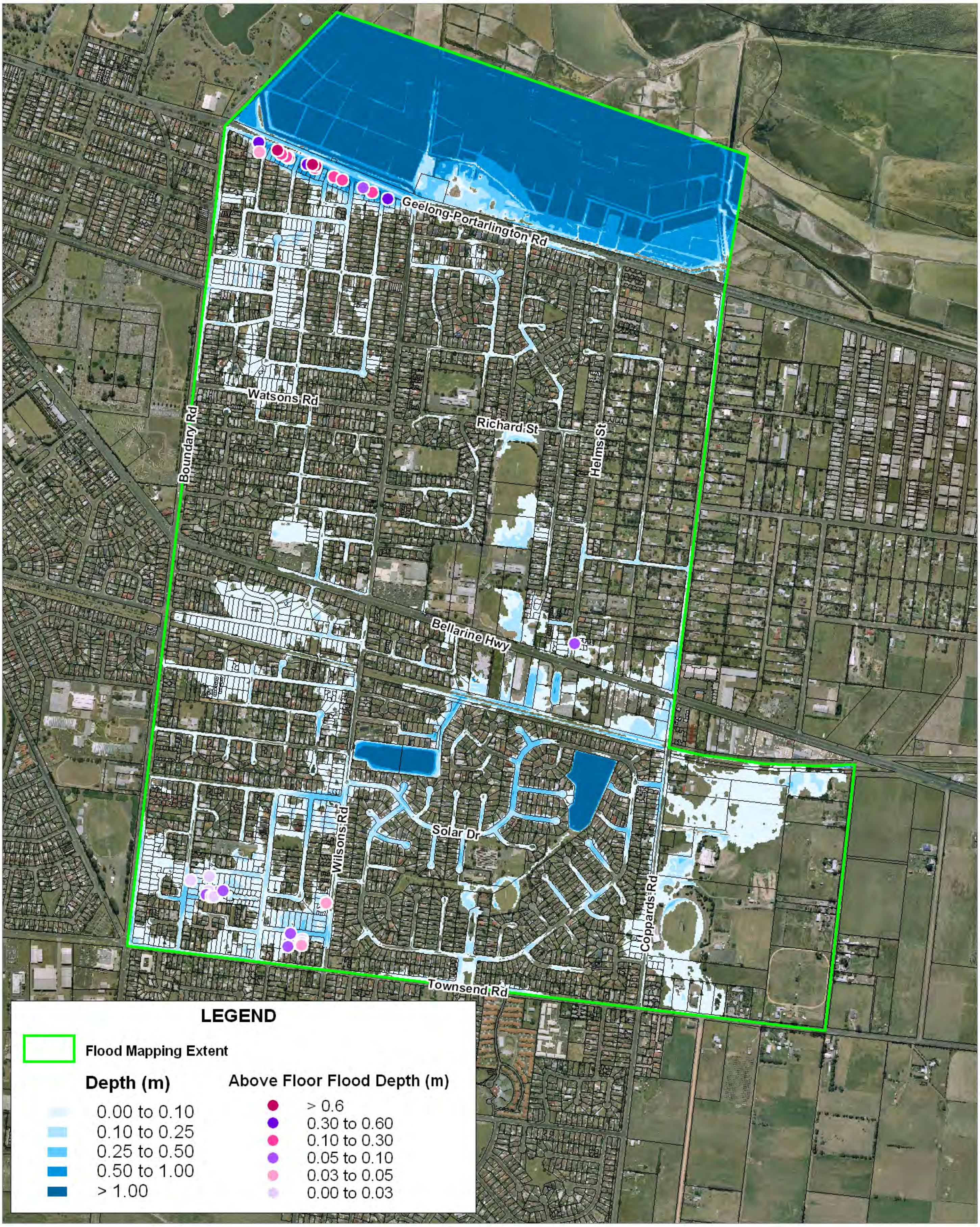

Title:

Existing Conditions 10% AEP Peak Flood Depth

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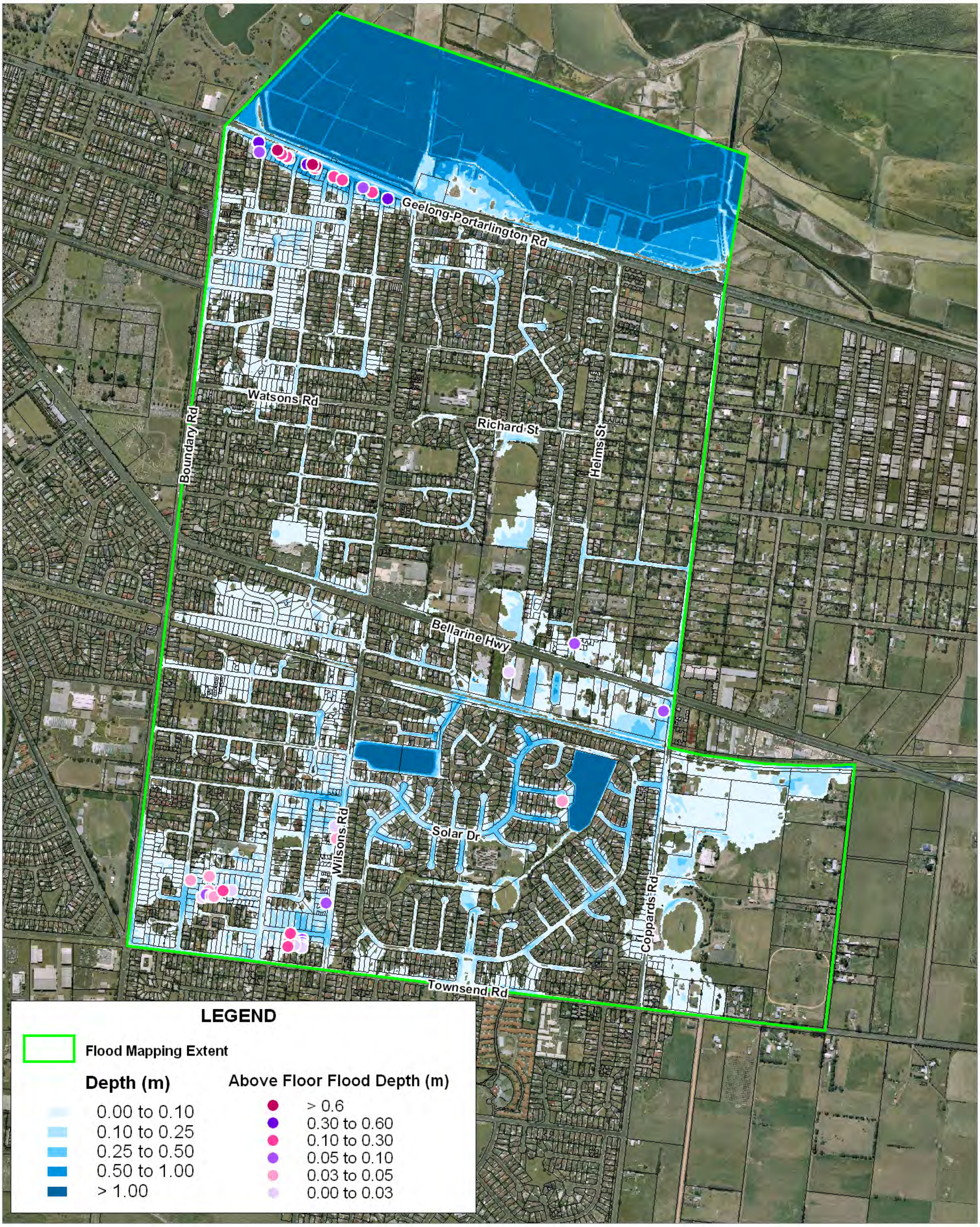

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Existing Conditions 5% AEP Peak Flood Depth

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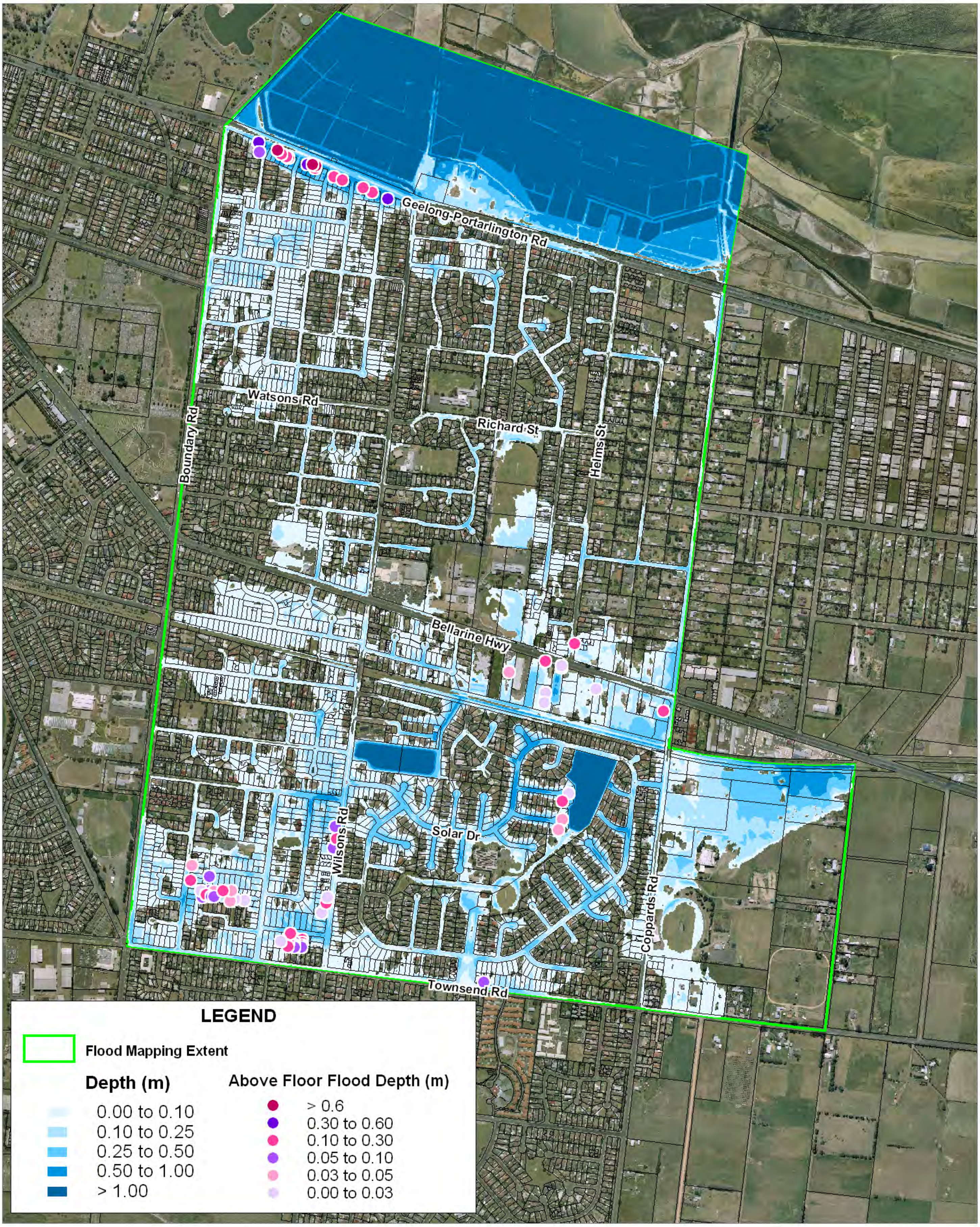

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Existing Conditions 1% AEP Peak Flood Depth

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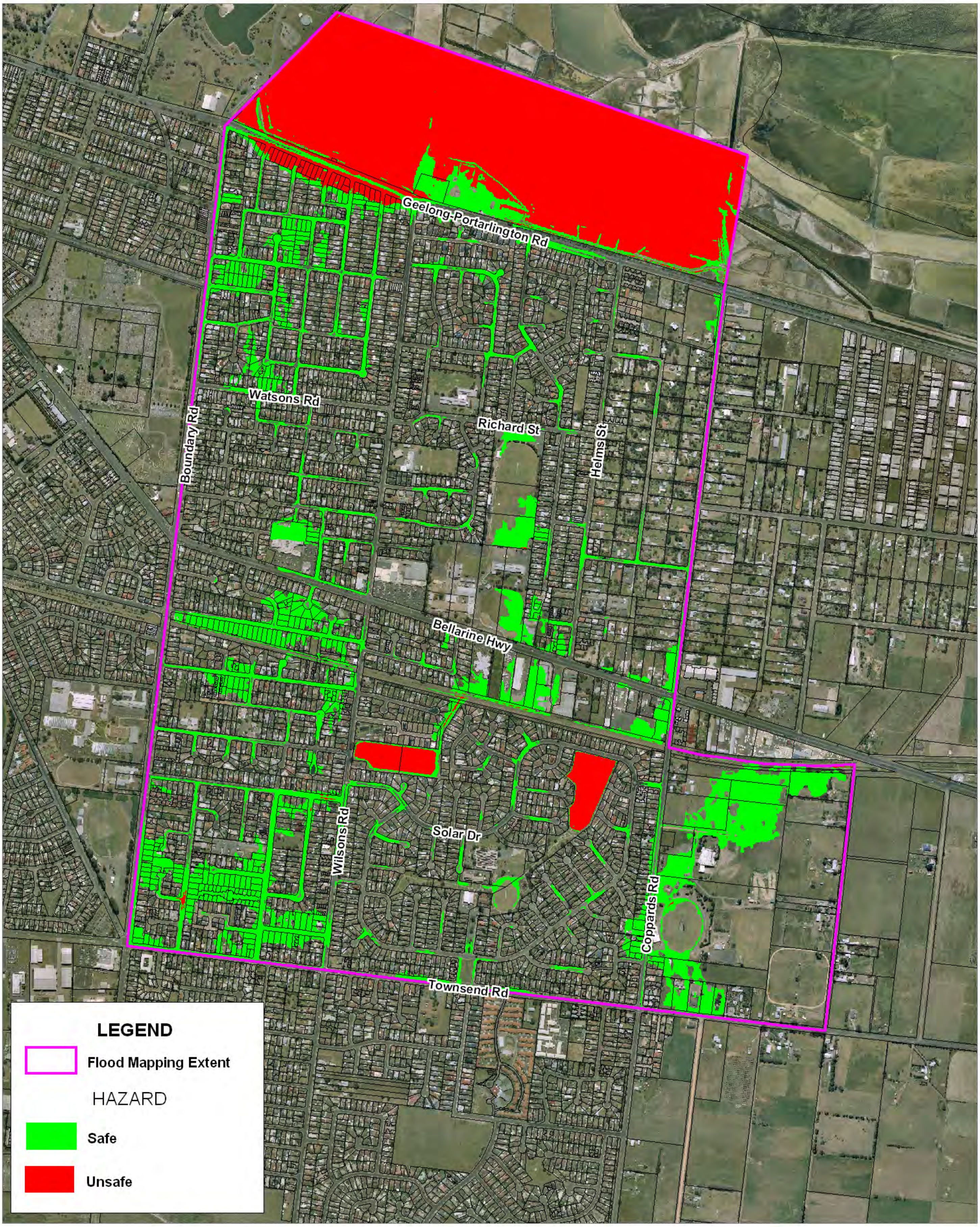

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Existing Conditions 20% AEP Peak Flood Hazard

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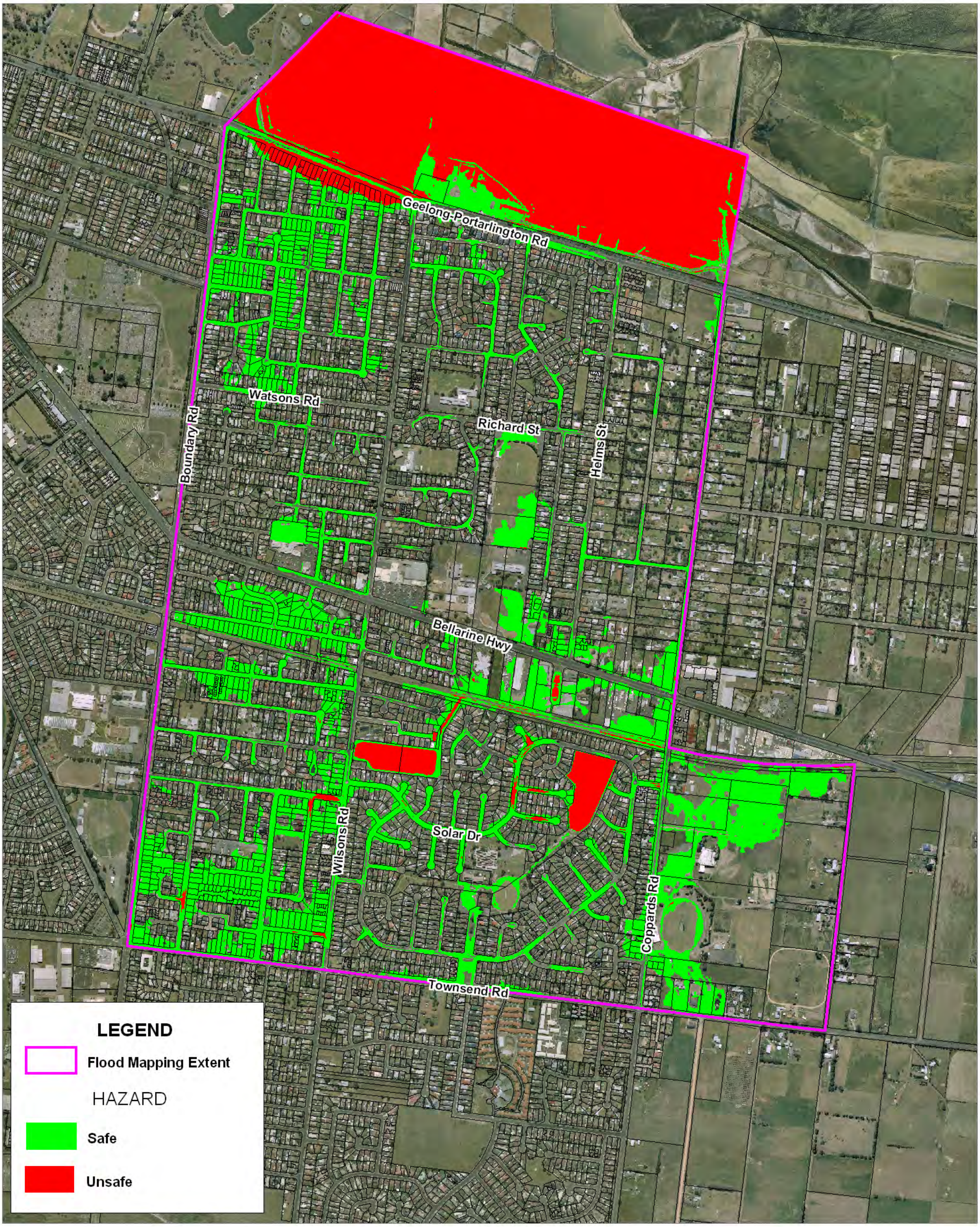

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Existing Conditions 10% AEP Peak Flood Hazard

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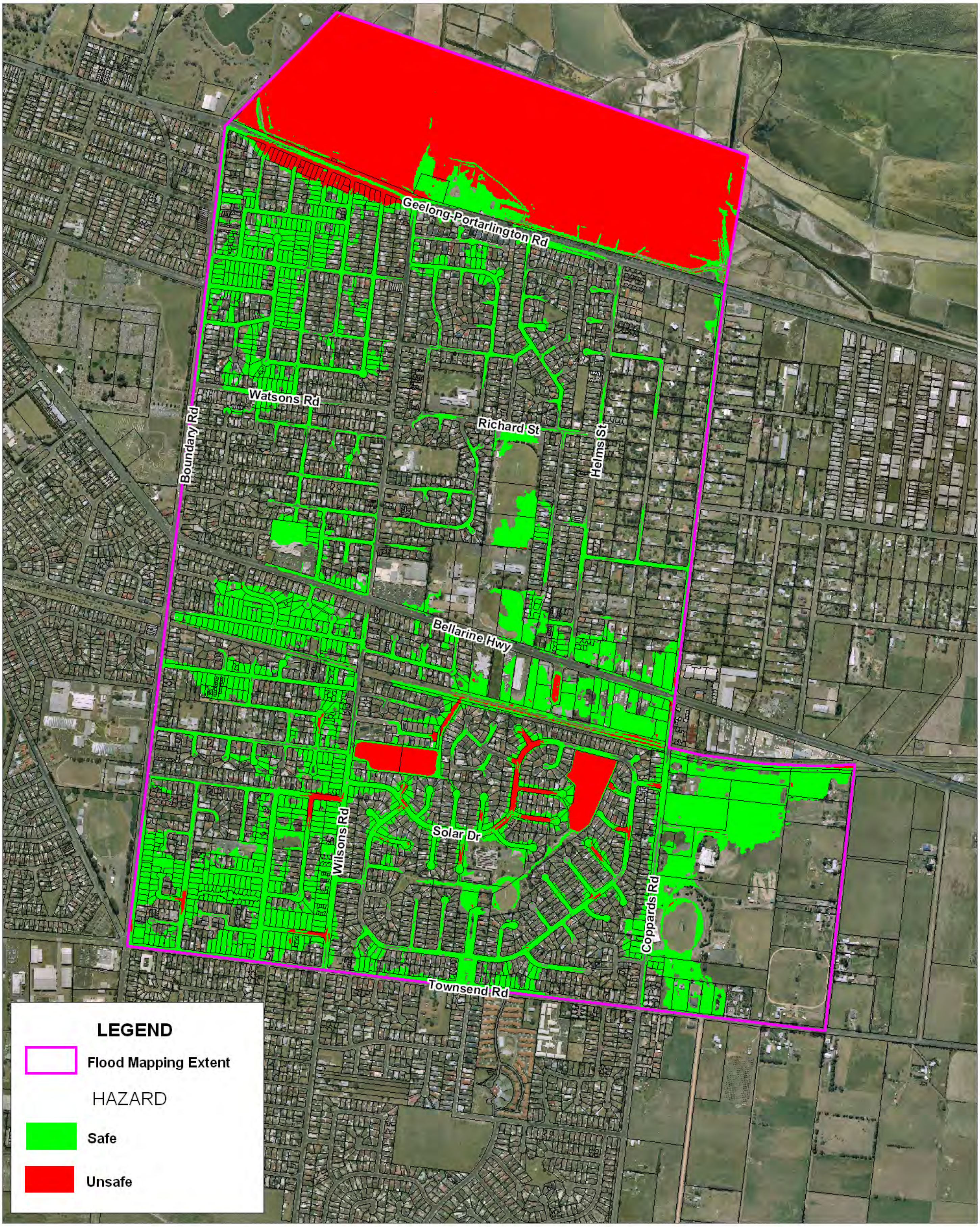

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Existing Conditions 5% AEP Peak Flood Hazard

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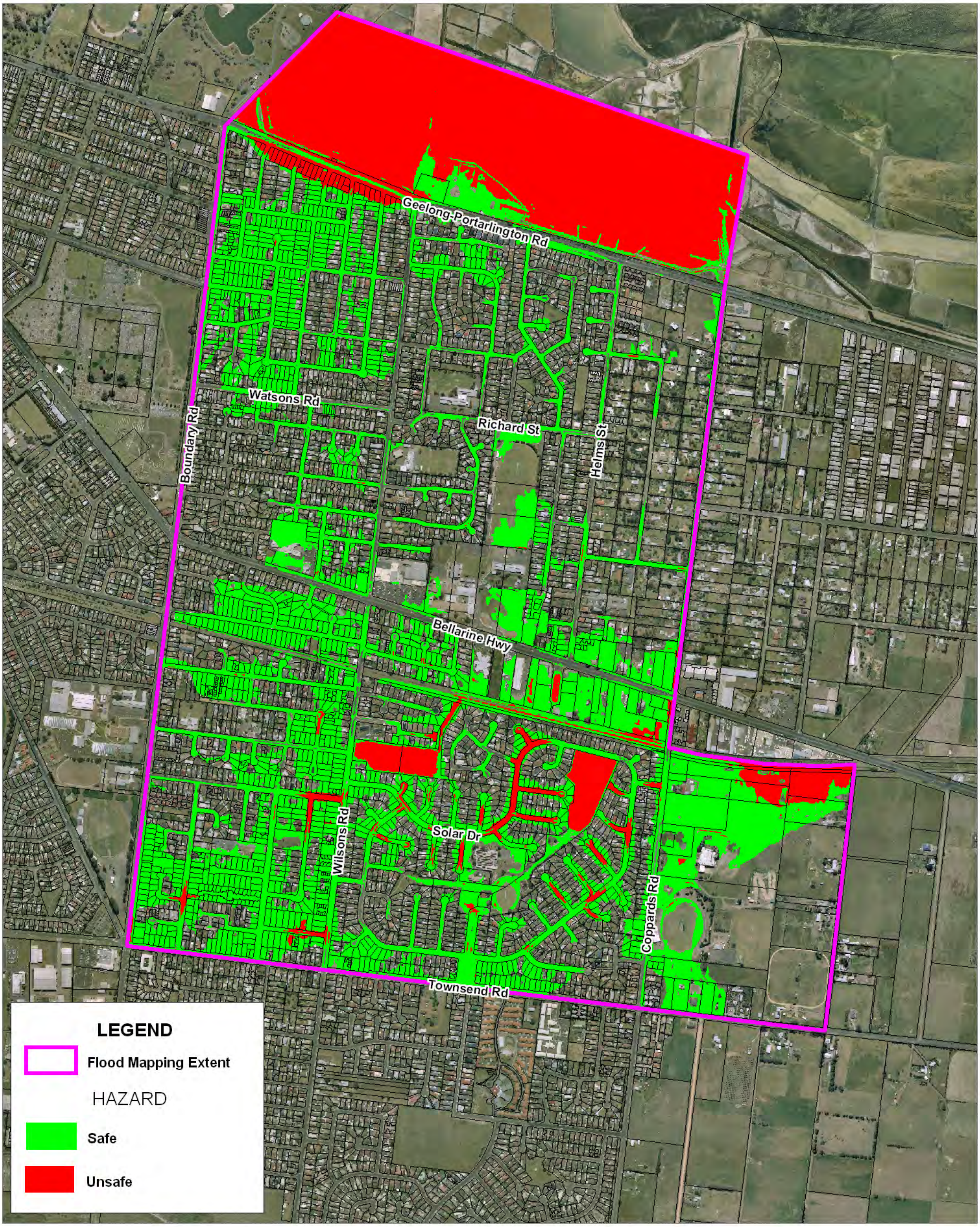

Title:

Preliminary 1% AEP Peak Flood Hazard

Figure: 5-8

Rev: B

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N 0 375 750m Approx. Scale

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