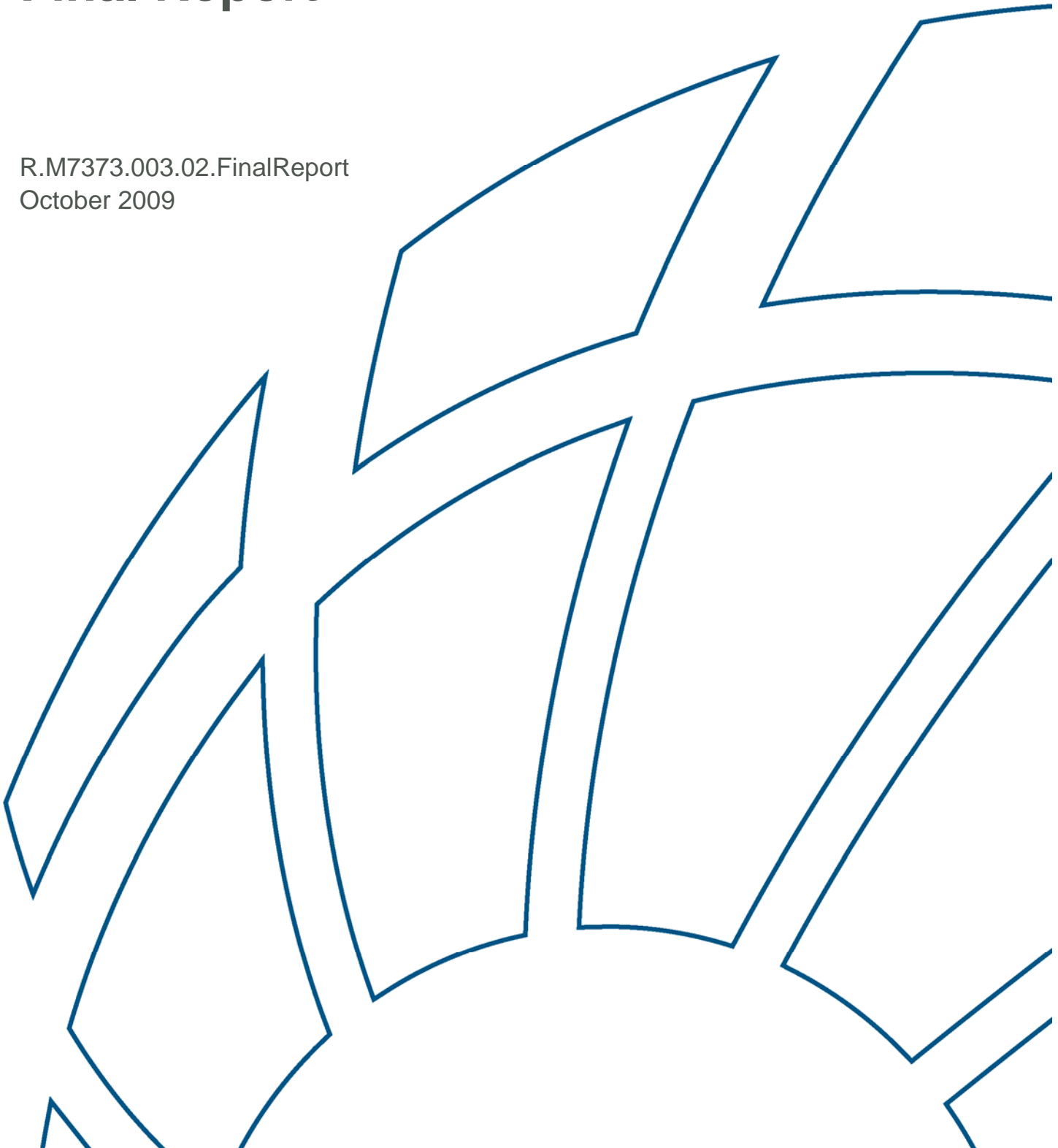




Portarlinton East Drainage / Flood Study Final Report

R.M7373.003.02.FinalReport
October 2009



Portarlington East Drainage/Flood Study Final Report

Prepared For: City of Greater Geelong

Prepared By: BMT WBM Pty Ltd (Member of the BMT group of companies)



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Title :	Portarlington East Drainage/Flood Study Draft Report
Author :	Joel Leister and Dr Mark Jempson
Synopsis :	This report documents the methodology and results for the Portarlington East Drainage/Flood Study.

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EXECUTIVE SUMMARY

Study Objective

The Portarlington East catchments have some of the most serious and frequent drainage-related or “stormwater” flooding within the municipality. Like many areas of similar age, the stormwater pipe network within Portarlington East is generally ageing and hydraulically constrained by today’s standards. There are only limited open channel flow systems or overland floodways to convey runoff during larger flood events, so when the capacity of the network is exceeded, above floor flooding may occur in the residential areas. 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 of the study area was commissioned and supplied by Photomapping Services. LiDAR data of the study area was also provided by the City of Greater Geelong (CoGG). The aerial survey data was used to prepare a topographic model of the study area, which was used to create a Digital Elevation Model (DEM). Additional continuous elevation strings representing features of hydraulic importance 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%, 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 two (2) alternative flood mitigation schemes were selected and 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 \$67,000. 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 catchment, that have flooding within the property boundaries in the 100 Year ARI event. This information is also detailed in Table E-1.

Hazard mapping was undertaken using the methodology prescribed in the Melbourne Water document *Guidelines for Development in Flood-prone Areas*. 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 mapping area is shown in Figure E-2. There are sections of the main overland drainage paths that are classified as unsafe in a 1% AEP event. In addition to these main overland flowpaths, the Portarlington Golf Course, the retarding basins between Fisher Street and Gellibrand Street, the retarding basin at Fairfax Street and Seaforth Drive are all regions classified as unsafe in a 1% AEP flood. During the 20% AEP flood, areas considered unsafe are generally located where flood water is 'ponding' and include the retarding basins, the golf course, along table drains and behind road embankments.

Table E-1 Number of Flooded Properties

AEP	Number of Flooded Properties	
	Portarlington East	
	Within Property	Above Floor*
1%	334	29
5%	285	21
20%	211	12

*Floor levels are based on properties surveyed. Refer to Section 3.6 (of the Main Report).



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Depth (m)

- 0.00 to 0.10
- 0.10 to 0.25
- 0.25 to 0.50
- 0.50 to 1.00
- > 1.00

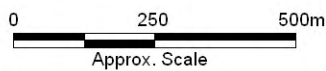
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Title:
Existing 1% AEP Peak Flood Depth

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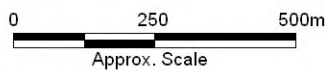


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Existing 1% AEP Hazard Mapping

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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 upgrading 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 Portarlington East 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, ie, the existing flood conditions, was also considered.

Each scheme was assessed using the hydraulic model for each flood event. Table E-2 shows the number of flooded properties under each scenario. 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 each scheme.

Table E-2 Flood Affected Properties

Case	Flood Affected Property Floors **		
	1% AEP	5% AEP	20% AEP
Existing	29	21	12
Scheme One	12	5	3
Scheme Two	19	12	7
Scheme Three (Do Nothing)	29	20	10

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

Table E-3 Mitigation Option Economic Summary

Case	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	\$26,000	\$41,000	\$509,000	\$4,431,000	\$106,000	\$1,315,000	\$5,746,000	0.09
Scheme Two	\$44,000	\$23,000	\$285,000	\$4,362,000	\$105,000	\$1,303,000	\$5,665,000	0.05
Scheme Three	\$67,000							

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

Preferred Mitigation Scheme

Scheme One is the preferred scheme for the Portarlington East drainage catchment. 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 Portarlington East. 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 significant reduction in the extent of the areas classified as unsafe, particularly in the areas of concern previously mentioned.

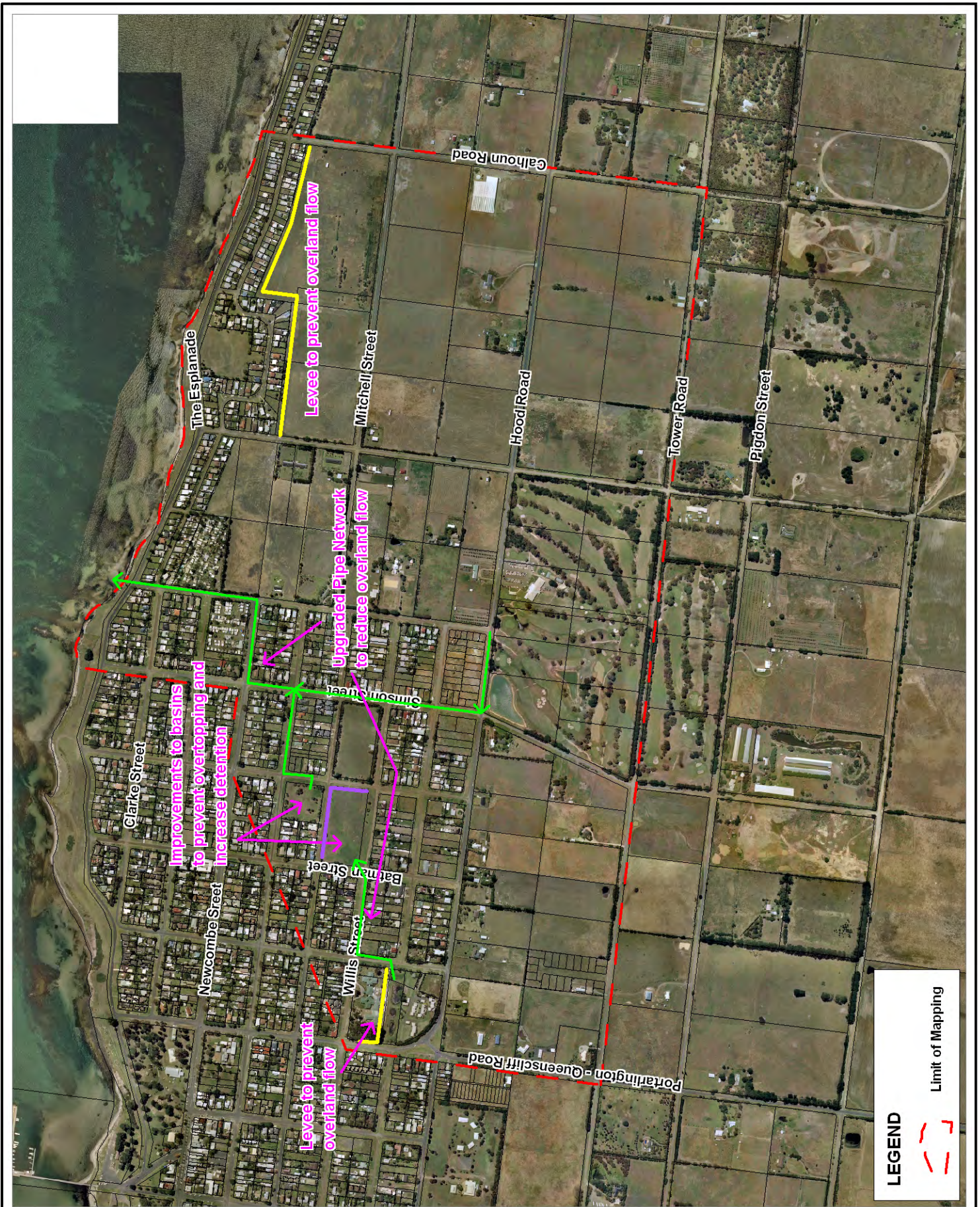
Scheme One has a higher capital cost and a higher BCR. The scheme results in reductions of flood levels along most of the major flow paths and reduces the overall flood extent, resulting in a reduction of flood affected properties. Although regions of high hazard still exist within the catchment, these areas are dramatically reduced by the preferred scheme.

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 schemes;
- the social impacts arising from Scheme One are very limited; and
- the feasibility and performance are comparable to the other schemes.

A number of non-structural components are recommended for implementation in conjunction with the structural measures. These additional measures are summarised below.

- Further investigation into the feasibility of property-specific measures to manage risk from residual flooding (with Scheme One 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, although opportunities for the latter may be limited because of recent increases in property values.
- Education and awareness program to inform landowners how to minimise the magnitude of damage during 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 catchment 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 (planning permit) 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.



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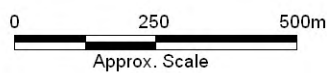
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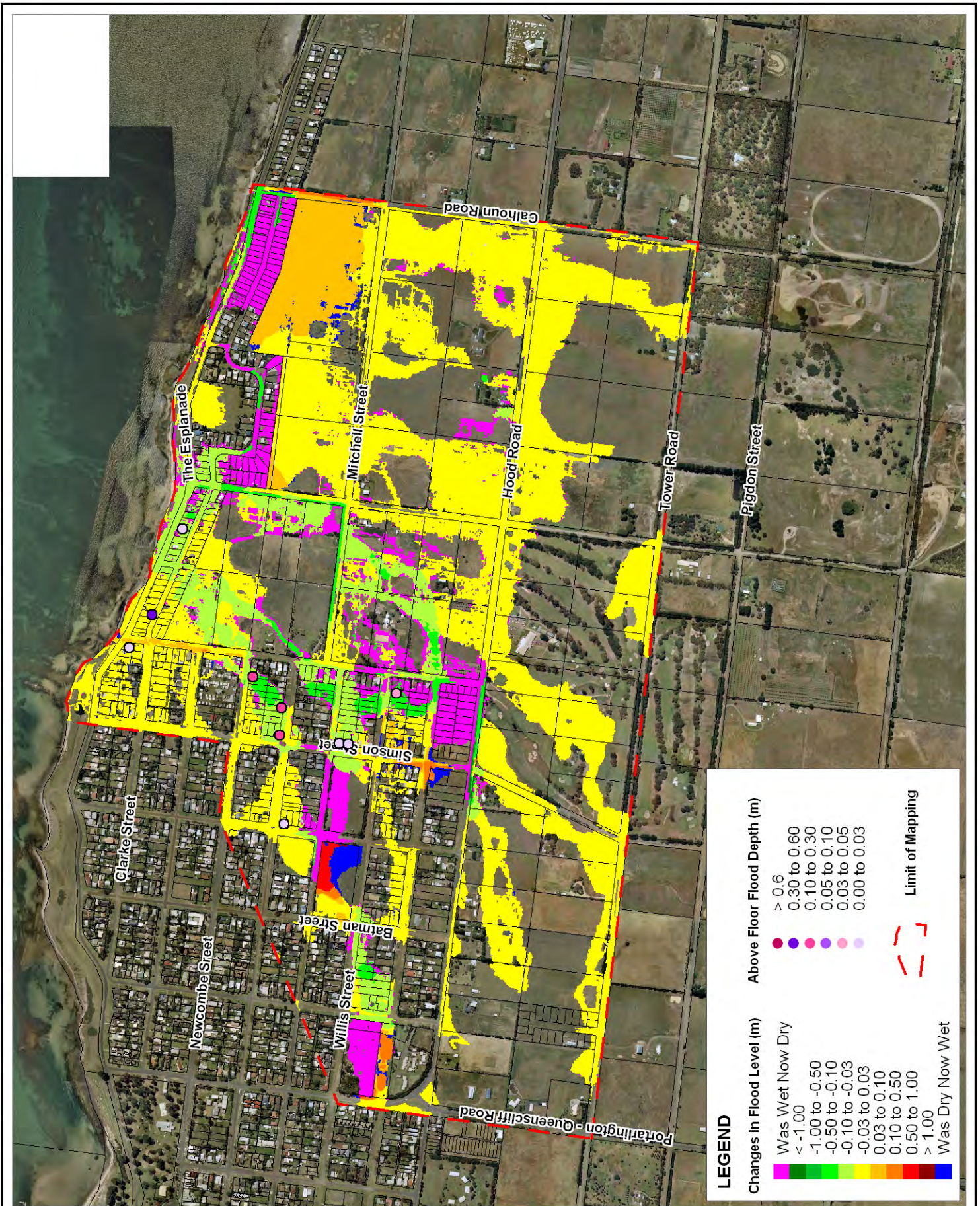
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Scheme One Drainage Works

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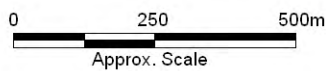


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Preferred Scheme
1% AEP Peak Flood Impact

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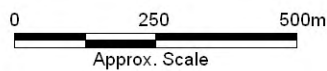


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1 INTRODUCTION

1.1 Background

The City of Greater Geelong (CoGG) engaged BMT WBM Pty Ltd (BMT WBM) to undertake an assessment of stormwater flooding within the Portarlington East drainage catchments (C57, C59, C60, C61 and C62) (Figure 1-1), and to investigate flood management options to manage and minimise the effects of flooding on the community.

BMT WBM has previously issued two interim reports (BMT WBM, 2008b & BMT WBM, 2008a) that presented the hydrological modelling and hydraulic modelling respectively. This report presents the study findings in its entirety including;

- The study approach;
- Data collation;
- Hydrological and hydraulic modelling;
- Flood mapping;
- Flood Damages assessment;
- Mitigation scheme options assessment;
- Preferred mitigation scheme; and
- Public consultation outcomes

1.2 Study Area

The Portarlington East study area is detailed in Figure 1-2. The study area is defined as the outer limit to the contributing catchments to the Portarlington East drainage catchments. The study area is also limited to Port Phillip 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 catchment. The area that is to be flood mapped is also shown in Figure 1-2.

1.3 Catchment Description

The Portarlington East drainage catchments are located in Portarlington, on the northern coast of the Bellarine Peninsula, around a 40 minute drive from Geelong (Figure 1-1). The catchments drain approximately 840 hectares of predominately rural land into Port Phillip Bay through a series of outfalls on the beach and via overland flow paths. The drainage catchments originate in the foothills to the South West of Portarlington and drain in a North East direction towards the Portarlington East sub-divisions and the Portarlington Golf Course. Once out of the foothills, the catchments are very flat and floodwaters have a tendency to build up behind road formations and localised high points.

This section of Portarlington was 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 catchment where drainage and street construction are yet to be implemented. In those areas already serviced the infrastructure consists of a series of underground pipes, open table drains alongside roads and natural overland flow paths. Three retarding basins have also been constructed within the catchment to help alleviate local flooding issues. The existing drainage network is generally undersized by today's standards and therefore contributes to localised 'stormwater' flooding.

Throughout the catchment many of the natural flow paths have been rendered ineffective due to the positioning of the streets in a grid style layout without appropriate drainage infrastructure. Consequently, floodwaters have been noted to build-up behind road embankments (CoGG 2008).

1.4 Flooding Complaints

Council has received numerous reports of flooding problems within the Portarlington East drainage catchments. The majority of the problems relate to floodwaters ponding in low-lying areas or behind localised high points, roads and intersections being inundated, and floodwaters flowing through the properties located within and around the natural drainage paths.

1.5 Key Objectives

The key objectives of this study are to:

1. develop a Digital Terrain Model (DTM) of the study catchment from digital data supplied by Photomapping Services;
2. determine the flood extents, depths and associated hazard of the critical 1%, 5% and 20% 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 1%, 5% and 20% 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.

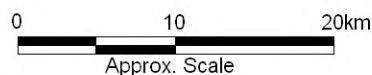


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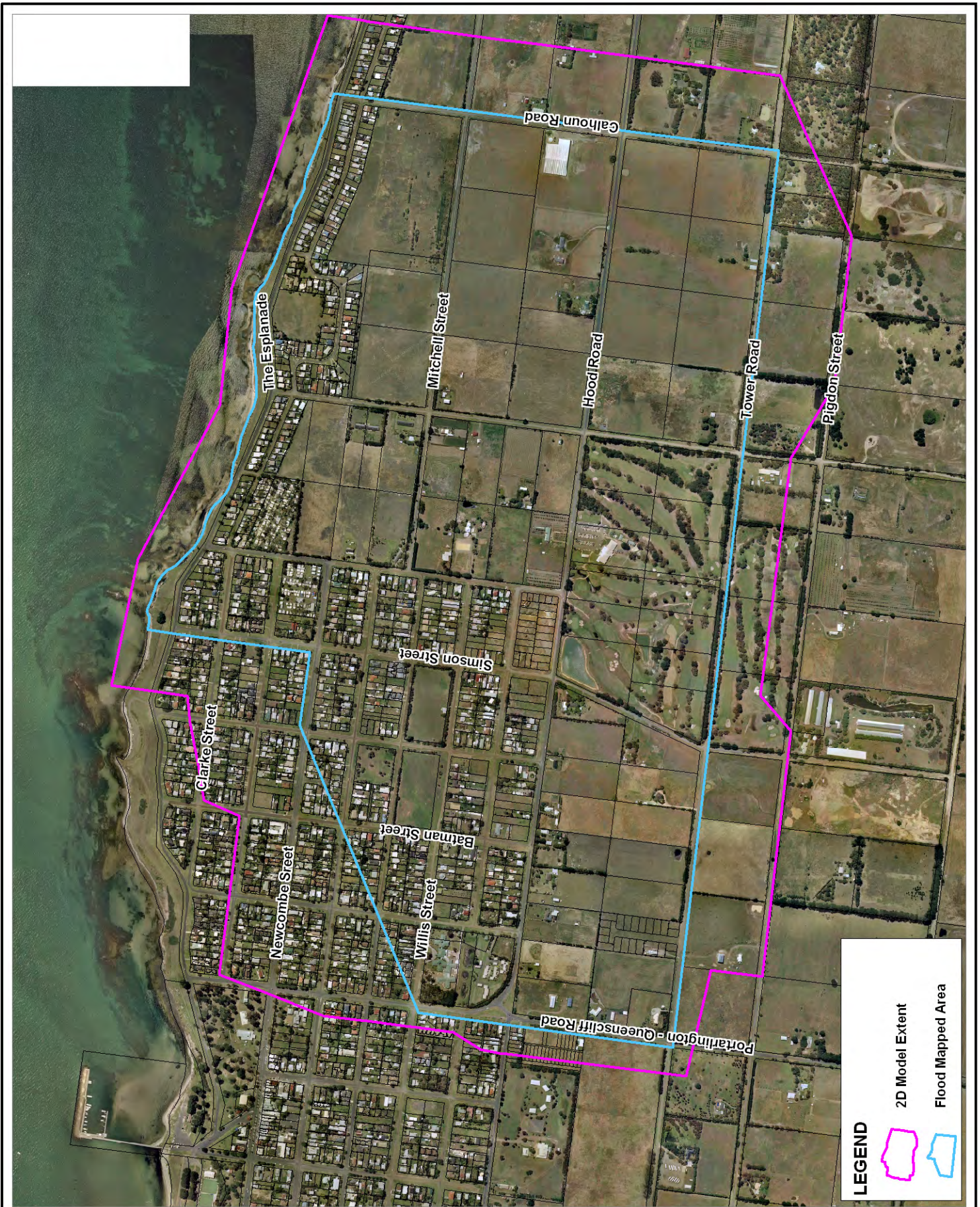
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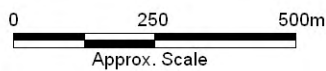


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2 STUDY APPROACH

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 some of the 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 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. During the course of the project, additional site inspections were undertaken by BMT WBM staff to clarify flood and drainage issues.

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.

Two Triangulated Integrated Networks (TIN) of the study area were developed from LiDAR survey supplied by CoGG and photogrammetric data commissioned as part of this study. The TINs were used to produce Digital Elevation Models (DEM), which are a three-dimensional representation of the ground surface. The DEM forms the basis of the hydraulic model.

Additional ground survey was obtained to verify the accuracy of the DEMs and to infill missing data relating to the underground pipes, open channels and spillway levels within the study catchment.

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

- cadastral information over the study area;
- planning scheme zones over the catchment;
- aerial photography; and
- ground and floor heights for selected properties (as determined by CoGG).

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 catchment was undertaken using RORB. RORB models had been previously utilised in the catchment by other consultants and were reviewed to ensure consistency with the models developed for this flood study. As discussed further in section 4.1.1, although there are a number of retarding basins within the catchment, they are all located within the hydraulic model extent and therefore not included within the RORB hydrologic model. No calibration data was available for the hydrological model, so for most parameters, typical values appropriate for the catchment characteristics were adopted and the model was then verified against the Rational Method and previous RORB models for the catchment. The adopted loss model was an initial loss/runoff coefficient model.

Hydraulic modelling of the catchment was undertaken using the 2D/1D dynamic hydraulic modelling system TUFLOW. The model incorporated both the overland flow paths and the underground trunk drainage system. No data was available for calibration so typical values based on experience and 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 occurring 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 1%, 5% and 20% 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 financial impacts of each scenario 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 the next stage. 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 was supplied:

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

3.3 Drainage Data

CoGG supplied drainage network data for the catchment 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.

3.4 Topographic Data

CoGG provided BMT WBM with topographic data from LiDAR flown over the entire Bellarine Peninsula. The vertical accuracy of the LiDAR data was $\pm 150\text{mm}$, and was initially considered to be unsuitable for the purposes of this study. Therefore, new aerial photogrammetry, with a higher vertical accuracy was required. Photomapping Services was commissioned to provide photogrammetry data with a vertical accuracy of $\pm 100\text{mm}$ on hard surfaces. The photogrammetry was flown in May 2008. The data was supplied in AutoCAD (DWG) format and was subsequently converted to a Triangulated Integrated Network (TIN), which was imported into MapInfo Professional.

Field data was obtained by TGM to supplement the photogrammetry data (see Section 3.5). The point heights along road crests were used to check the vertical accuracy of the DTM. The photogrammetry was found to be outside tolerance at an unacceptable number of these points (see Figure 3-1). A similar check of the LiDAR DTM found that it was within $\pm 100\text{ mm}$ at most locations (see Figure 3-2). The only exception was along Oxley Street where the LiDAR was consistently lower than the ground survey by more than 100 mm. TGM completed a desktop check and found no errors

in their survey along this street. At the time of writing Photomapping had not provided a response to these observed differences. Based on these findings it was decided to adopt the LiDAR DTM in the hydraulic model. The resulting DEM is shown in Figure 3-3.

Within the flood mapping area the sampling resolution for the DTM is 0.3m. 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.

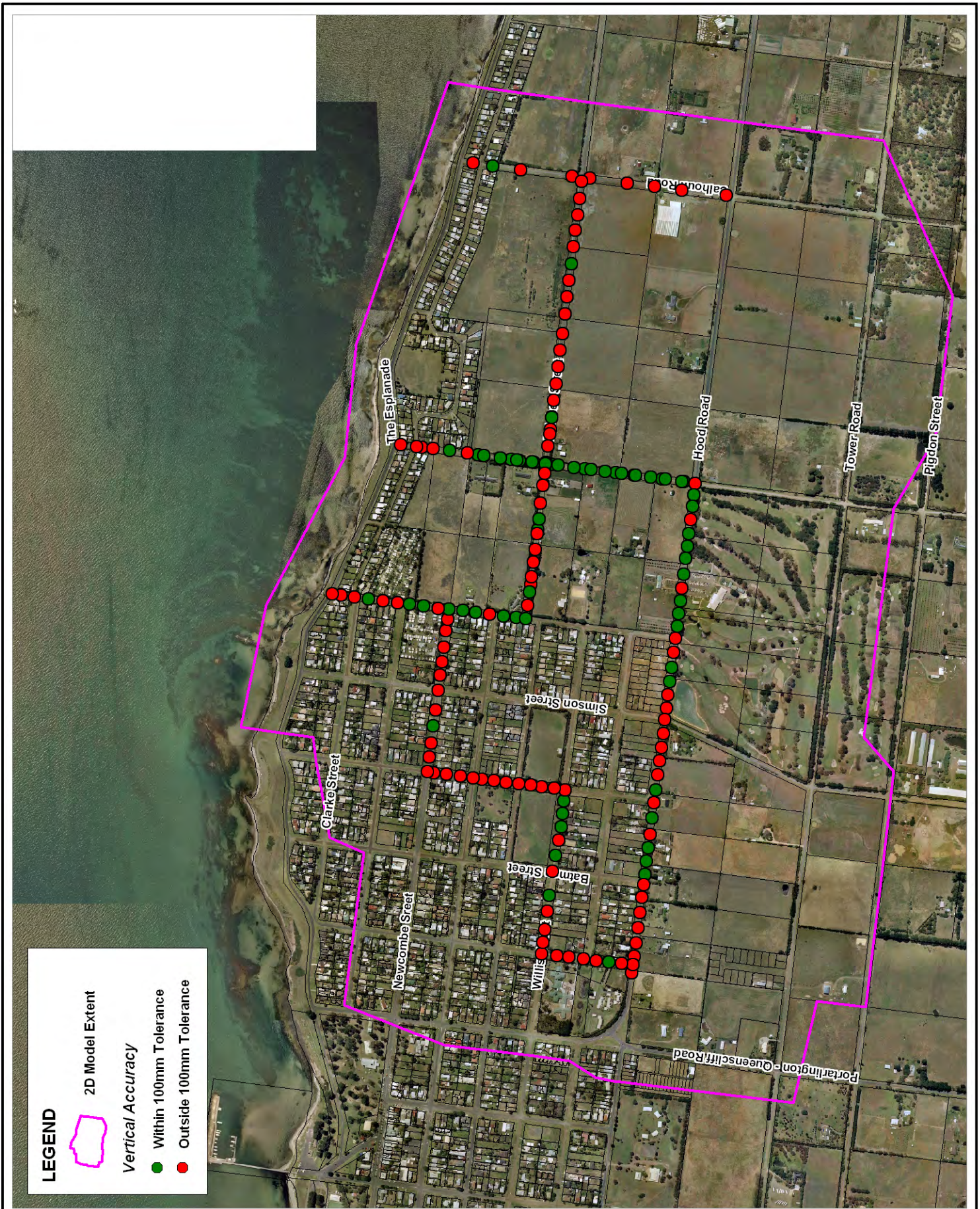
3.5 Field Survey

Field survey was required to capture missing data and to provide detailed data of drainage structures and features that could not be reliably captured using aerial survey techniques. TGM Group Pty Ltd was commissioned to capture pipe inverts, retarding basin levee crest heights, details of driveway crossings, cross sections of the table drains along Mitchell Street, Oxley Street and Calhoun Road, and road centrelines. This data was provided in data files and plotted on plans. Digital copies of the plans are provided as a CD in Appendix B.




3.6 Floor Levels

Floor level information was supplied to BMT WBM from new survey commissioned by CoGG. The data supplied included the floor level and ground levels with associated location (as x,y coordinates). The location of surveyed floor levels is shown in Figure 3-4.

The extent of floor level survey was determined by CoGG and does not exist for all properties within the study area. As a result, all numbers of flooded properties, damage figures and hence Benefit Cost Ratios, are based on the data set provided.



LEGEND

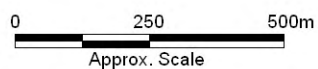
-  2D Model Extent
- Vertical Accuracy**
-  Within 100mm Tolerance
-  Outside 100mm Tolerance

Title:
Vertical Accuracy Check of Photogrammetry DTM

Figure:
3-1




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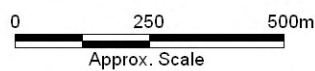
-  2D Model Extent
- Vertical Accuracy**
-  Within 100mm Tolerance
-  Outside 100mm Tolerance

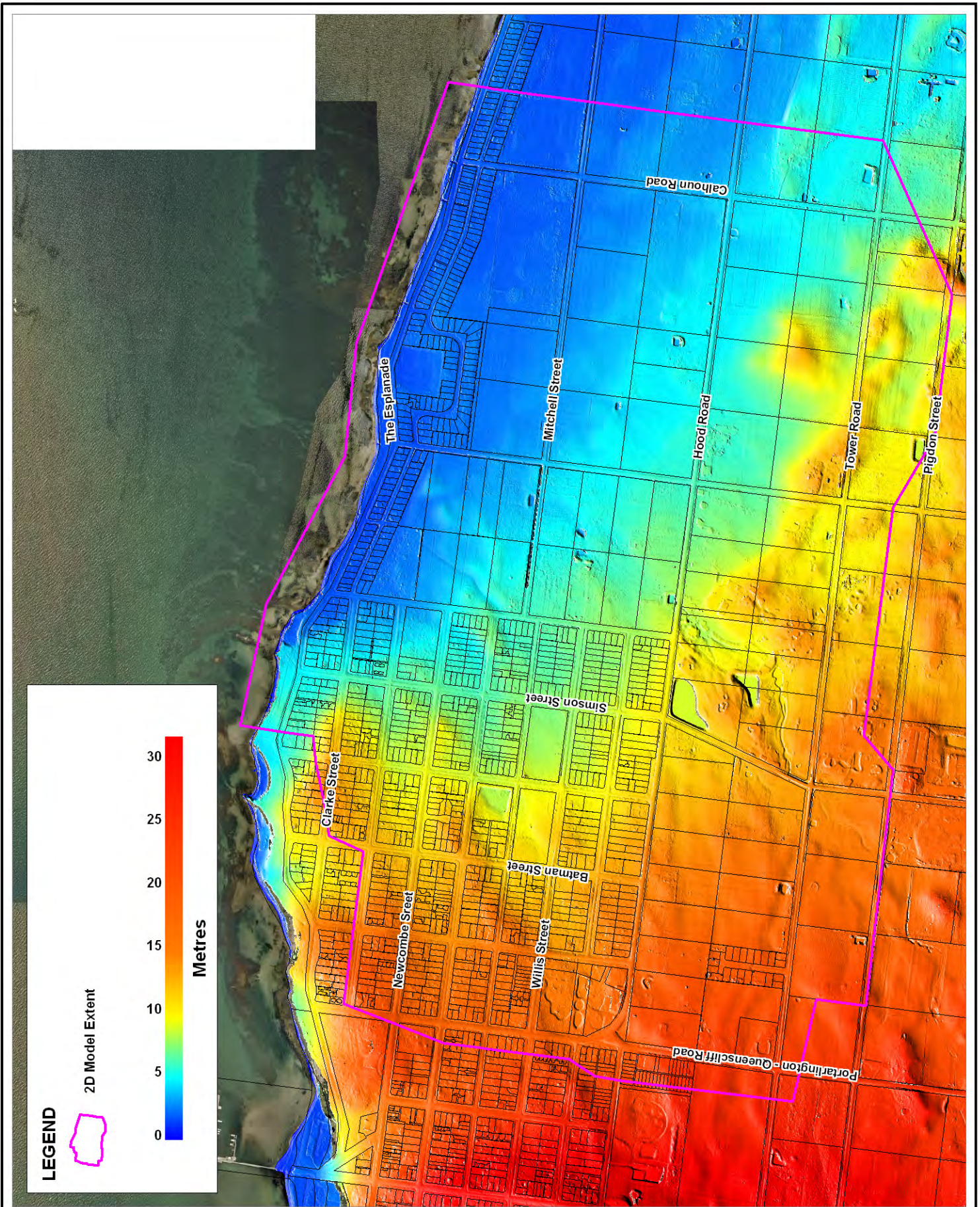
Title:
Vertical Accuracy Check of LiDAR DTM

Figure:
3-2

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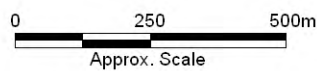


Title:
Portarlington East LiDAR DEM

Figure:
3-3

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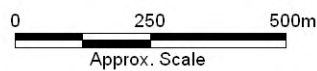


Title:
Surveyed Floor Levels

Figure:
3-4

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4 FLOOD MODEL DEVELOPMENT

4.1 Hydrological Model

Hydrologic modelling of the study catchment was undertaken using RORB. A RORB model of each catchment was established for the purpose of extracting total and sub-area hydrographs to be used as boundary conditions for the TUFLOW hydraulic model (see Section 4.2 for details). A description of the RORB modelling process and results is 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;

$$S = 3600kQ^m$$

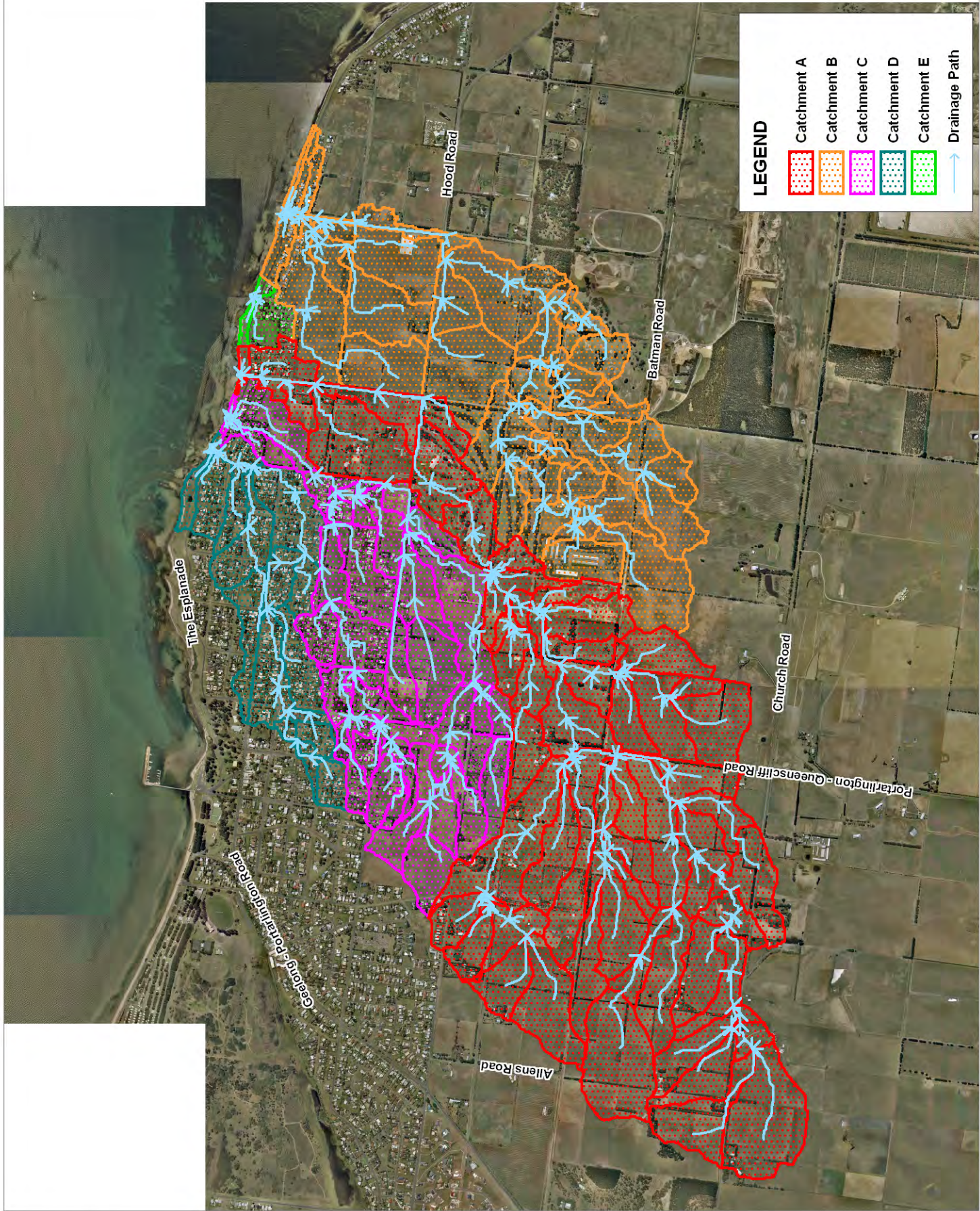
Where 's' represents the storage (m^3), 'Q' is the discharge (m^3/s), 'm' is a dimensionless exponent and 'k' is non-dimensional empirical coefficient. 'k' is defined by the product of the catchment value 'k_c' and the individual reach k_i. Both m and k_c are defined as calibration parameters. Based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

4.1.1.1 Model Description

The RORB models incorporate an area of approximately 8.4 square kilometres, including the Portarlington East study area. To ensure accurate representation of the overall catchment, five individual RORB models were developed, each corresponding to an outlet to Port Phillip Bay. The development of five individual models meant that routing parameters and loss factors could easily be applied to reflect the characteristics of each individual catchment. Overall, there are 162 sub-catchments within the RORB models and each sub-catchment has a conceptual reach (approximate overland flow path) defined. Although three retarding basins exist within the Portarlington East study catchments, they have not been included in the RORB hydrologic model. The retarding basins are located within the extents of the TUFLOW hydraulic model and hence modelling of the storage effects of each of the basins is done in the hydraulic model.

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 (supplied by CoGG) and the photogrammetry collected for this study. The schematic RORB layout is shown in Figure 4-1.



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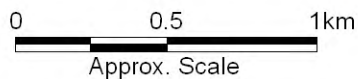
- Catchment A
- Catchment B
- Catchment C
- Catchment D
- Catchment E
- Drainage Path

Title:
RORB Catchment Layout

Figure:
4-1

Rev:
A

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4.1.1.3 Global Parameters

RORB model parameters for the Portarlinton East drainage catchment are summarised in Table 4-1 to Table 4-4 and are discussed further in Sections 4.1.1.4 to 4.1.1.6. Catchment E does not have a table summarising its parameters as it was only developed to apply local boundaries to the TUFLOW model. The routing parameters (k_c and m) are not applicable in this case. The losses applied to catchment E were identical to those adopted in catchments C and D.

Table 4-1 RORB Parameters – Catchment A

RORB Parameter	Value
Storm Data	Portarlinton
Catchment Area	3.9 km ²
Initial Loss	10.0 mm
Continuing Loss	15 mm/hr
m	0.80
k_c	8.30
Fraction Impervious	Varies, as per land use (Table 4-6)
Reach Type	Type 2
Peak Flow Event	2 Hour

Table 4-2 RORB Parameters – Catchment B

RORB Parameter	Value
Storm Data	Portarlinton
Catchment Area	2.1 km ²
Initial Loss	10.0 mm
Continuing Loss	15 mm/hr
m	0.80
k_c	2.35
Fraction Impervious	Varies, as per land use (Table 4-6)
Reach Type	Type 2
Peak Flow Event	2 Hour

Table 4-3 RORB Parameters – Catchment C

RORB Parameter	Value
Storm Data	Portarlington
Catchment Area	1.6 km ²
Initial Loss	10.0 mm
Continuing Loss	2.5 mm/hr
m	0.80
k _c	1.90
Fraction Impervious	Varies, as per land use (Table 4-6)
Reach Type	Type 2
Peak Flow Event	1 Hour

Table 4-4 RORB Parameters – Catchment D

RORB Parameter	Value
Storm Data	Portarlington
Catchment Area	0.7 km ²
Initial Loss	10.0 mm
Continuing Loss	2.5 mm/hr
m	0.80
k _c	1.15
Fraction Impervious	Varies, as per land use (Table 4-6)
Reach Type	Type 2
Peak Flow Event	2 Hour

4.1.1.4 IFD Parameters

Storm data was based on IFD parameters sourced from the Bureau of Meteorology and 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 catchment are presented in Table 4-5.

Table 4-5 IFD Parameters

IFD Parameter		Adopted Value
Rainfall Intensity (mm/hr)	2 Year ARI, 1 Hour Duration	17.99
	2 Year ARI, 12 Hour Duration	3.62
	2 Year ARI, 72 Hour Duration	0.92
	50 Year ARI, 1 Hour Duration	34.47
	50 Year ARI, 12 Hour Duration	6.70
	50 Year ARI, 72 Hour Duration	1.94
Skew Coefficient		0.40
Geographical Factor F2		4.28
Geographical Factor F50		14.85
Zone		1

4.1.1.5 Fraction Impervious

The fraction of the catchment 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 4-6.

Table 4-6 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.

4.1.1.6 Loss Model

The “initial loss followed by a continuing loss” loss model was adopted. This modelling approach is consistent with previous studies on the Bellarine Peninsula, including the Barwon Heads Drainage Flood Management Study (WBM 2005). RORB generates excess runoff by subtracting losses at each time-step from the rainfall occurring in that time period.

To establish loss rates for pervious areas BMT WBM collected data describing soil types and geology from CSIRO and the Australian Government’s Office of Spatial Data Management (OSDM). This data, along with observations made during the site inspections, confirmed that the underlying terrain in some parts of the catchment have a high proportion of sand. In these catchments, a higher continuing loss of 15 mm/hr was applied to account for the increased infiltration into the soil profile. This loss rate was consistent with the rate adopted in WBM (2005), which was for a catchment in coastal dunes. A lower continuing loss of 2.5 mm/hr was applied to catchments that were predominately urban to account for the soils being landscaped, compacted and more likely to have less infiltration capacity. For impervious areas RORB has hardwired initial and continuing loss rates of 0 mm and 0 mm/hr.

4.1.1.7 Retarding Basins

The three retarding basins within the Portarlington East drainage catchments 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 structure. Therefore it is not necessary to represent the retarding basins in the RORB hydrologic models. For all sub-catchments within the hydraulic model extent, only local sub-area inflows are applied, not total hydrographs. Therefore the routing of flows through the various storages associated with the retarding basins does not need to be modelled.

4.1.1.8 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.

The RORB models were initially run using a k_c derived from the Melbourne Water relationship between k_c and catchment area:

$$k_c = 1.19A^{0.56}, \text{ where } A \text{ is catchment area in square kilometres.}$$

The peak outflow from each catchment, apart from catchment E was compared against a Rational Method calculation to validate the routing parameters. Catchment E is fully within the hydraulic model extent and so only sub-area local flows are extracted from RORB. As the routing parameters are not significant in this context, a check on catchment E was not undertaken. For two of the catchments (catchments A and C), additional points of comparison were used because their unique shape causes partial area affects.

Initial comparisons showed that there were significant differences between the peak outflows from the RORB model and those determined using the Rational Method. The comparisons suggested that the value of k_c adopted was generally too low for the catchments. The value of k_c was adjusted for each of the catchments until reasonable agreement between the RORB peak flows and the Rational Method peak flows was achieved.

Table 4-7 documents the peak RORB outflows from four of the five catchments and compares them to the peak flow calculated using the Rational Method. As mentioned previously, good agreement was achieved at all locations except at the outlet of catchment A. The k_c for catchment A was adjusted to achieve a good agreement at the location u/s of the golf course as this an external flow boundary for the hydraulic model and hence is a more significant location. Downstream of the golf course in catchment A, the RORB sub-areas are fully within the hydraulic model extent and hence the routing parameters have no influence on the boundaries.

Table 4-7 Comparison of Rational Method and RORB

Location	Peak Flow (m ³ /s)		% Difference
	Rational Method	RORB	
Catchment A - Outlet	17.1	14.2	-20.6%
Catchment A (U/S of Golf Course)	16.6	16.7	0.5%
Catchment B	12.5	12.5	-0.2%
Catchment C - Outlet	20.3	20.9	3.2%
Catchment C (U/S Mercer Street)	21.1	21.1	-0.3%
Catchment D	10.2	10.2	0.3%

4.1.1.9 Arlington Rise Development

Arlington Rise is a proposed development in Portarlinton. Coomes Consulting Group (Coomes) developed a site stormwater management plan for the development (Coomes 2008). Coomes constructed a RORB model of the local catchment to determine peak flows and size a retarding basin to ensure 'No Net Change' on drainage infrastructure downstream. BMT WBM was provided with copies of both the site stormwater management plan and RORB model to review and include, where applicable, into the RORB model developed for the Portarlinton East Flood Study.

The peak 1% AEP developed case flow rate determined by Coomes (4.5 m³/s) was significantly lower than that calculated by BMT WBM (7.0 m³/s) at the same location using the whole-of-catchment RORB model. The most notable difference between the models was the loss model; Coomes adopted an initial loss/runoff coefficient model whilst BMT WBM used an initial loss /continuing loss model.

BMT WBM tested the whole-of-catchment RORB model with Coomes' initial loss of 25mm (BMT WBM had adopted 10mm). The peak flow rate of 4.3 m³/s was almost identical to that from the Coomes model.

As part of the development of the local catchment RORB model, Coomes calibrated their model to the Rational Method. BMT WBM undertook a similar process during the development of the whole-of-catchment RORB model (refer to section 4.1.1.8). The verification undertaken by BMT WBM involves the entire catchment and the initial losses adopted within this model are more representative of the entire catchment.

Following the review of the Coomes' RORB model of the Arlington Rise development, it was determined that the initial loss and runoff model already adopted by BMT WBM should remain. The peak flows determined by Coomes for the local development have not been used as inputs to the hydraulic TUFLOW model. However, the retarding basin designed as part of the development to ensure 'No Net Change' to drainage infrastructure downstream has been incorporated into BMT WBM whole-of-catchment model.

4.1.1.10 Design Event Modelling

4.1.1.10.1 Design Event Probabilities

Hydrological analysis was undertaken for the 1%, 5% and 20% Average Exceedance Probability (AEP), i.e. the 100, 20 and 5 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 catchment.

4.1.1.10.2 Design Rainfall

Intensity – Frequency – Duration (IFD) parameters for Portarlington East were derived using 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.

Filtered temporal patterns were used to derive the design storm events.

Aerial Reduction Factors (ARF) were not applied to this catchment due to its small size.

4.1.1.10.3 Critical Duration Derivation

For each design probability, the peak discharge at various locations within the drainage system may be generated by 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 catchment, events ranging from 1 hour to 12 hours duration were modelled.

4.2 Hydraulic Model

The drainage system in Portarlinton comprises underground stormwater pipes, open channels and complex overland flow paths. Overland flow paths are controlled to a greater or lesser extent by “hydraulic controls” such as road embankments, property accesses and retarding basins. TUFLOW, a fully 2D hydraulic modelling package with the ability to dynamically nest 1D elements, was adopted for this study. Overland flow paths and storages were modelled in 2D, whilst the underground pipe network and open drains were modelled as 1D elements dynamically linked to the 2D domain.

In a catchment such as the Portarlinton East, where storage and timing of the rainfall inflows in the catchment 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

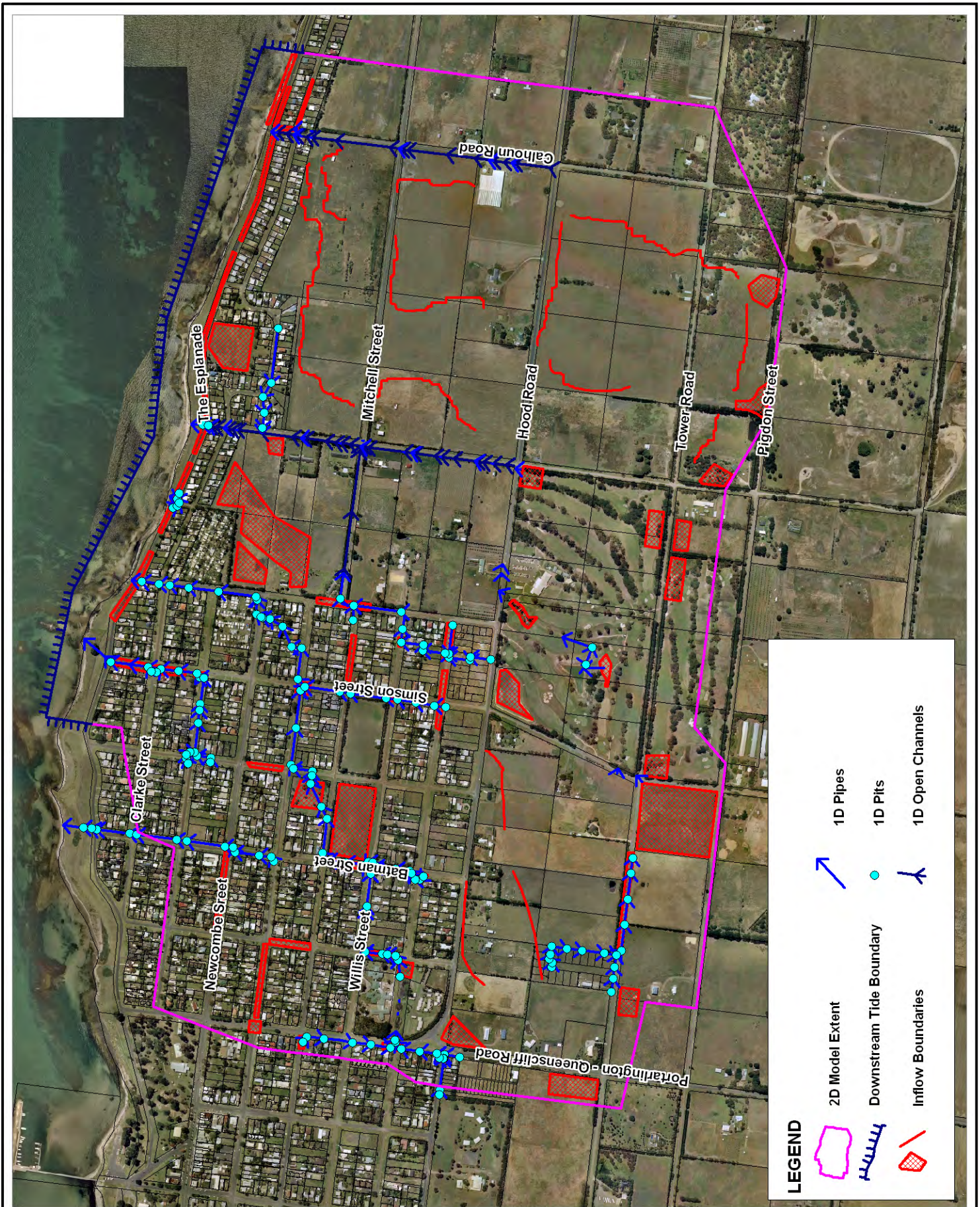
The 2D model domain extends beyond the flood mapping limit required to minimise boundary effects, and covers about 4.1 square kilometres of the Portarlinton East drainage catchments (as shown in Figure 4-2). The geometry of the 2D model was established by constructing a uniform grid of square elements. 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 fine grid element size will result in both a higher resolution and longer model run times.

In adopting the element size for the Portarlinton East 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.

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. Thirteen 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 4-8.

As discussed previously, the underground stormwater pipes were modelled as 1D components dynamically linked to the 2D domain. Key features of the drainage system, typically pipes with a diameter greater than and including 450mm, were modelled. Exceptions to this included a number of the culverts located beneath driveways along Oxley St and Calhoun Rd and some of the pipes in the retarding basin on Fairfax St. The pipe details included were based upon the drainage information provided by CoGG and the additional survey information captured by TGM. Interchange of water between the 1D and 2D domains (i.e. between the pipe network and the surface) was constricted to a typical side entry pit dimension (0.95m wide and 0.15m high) in the catchment. Some pit entry dimensions varied depending on their locations and the type of pit.

To ensure that the hydraulic conveyance of the table drains along Mitchell St, Oxley St and Calhoun Rd were accurately modelled, these drains were modelled as a series of 1D elements. There were many property accesses across these drains. All of these accesses were modelled as a culvert and weir arrangement within the 1D domain. Standard culvert entry and exit loss coefficients of 0.5 and 1.0 were adopted.



LEGEND

- 2D Model Extent
- Downstream Tide Boundary
- Inflow Boundaries
- 1D Pipes
- 1D Pits
- 1D Open Channels

Title:
TUFLOW Model Setup

Figure:
4-2

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B

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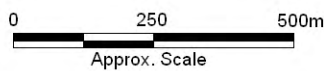


Table 4-8 2D Domain Manning's 'n' Coefficients

Land use	Manning's n
Residential	0.2
Nursing Home	0.1
Caravan Parks	0.08
Golf Course	0.035
Open Space – Poorly Maintained Grass	0.04
Open Space - Paddocks	0.05
Open Space – Heavy Vegetation	0.06
Roads – Gravel	0.02
Roads – Asphalt	0.016

4.2.2 Downstream Boundary Condition

The TUFLOW model uses a fixed water level of 1.12 m AHD for the downstream boundary condition. This level was adopted based on discussions between BMT WBM and CoGG regarding a suitable tailwater condition. The level is based on the Highest Astronomical Tide (HAT) for Williamstown (0.52 m AHD) and an allowance for increased tide levels caused by climate change (0.6 m).

4.2.3 Design Event Modelling

The 1%, 5% and 20% AEP design storm events were modelled in TUFLOW for five storm durations: 1 hour, 2 hours, 3 hours, 6 hours and 12 hours. The critical storm duration varied across the catchment, but the 1 hour and 2 hour storms were the critical duration in most areas. The 12 hour storm was included to ensure peak flood levels were obtained in areas where the flood is likely to 'pond', including in the retarding basins and lakes and to the South of the houses along Sunset Bvd. A peak flood height envelope was developed from the five durations and the peak envelope flood surfaces mapped. The mapping is presented in Section 5. These flood events formed the basis of the assessment of the mitigation options, including the damages assessment.

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 in WBM (2008). 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 catchment.

4.2.4 Sensitivity Testing

The flood model indicates floodwater entering the nursing home from Fisher Street. This region of the model is close to the model boundary and a sensitivity test was undertaken to ensure that flooding through the nursing home is not an artefact of the boundary location. The model boundaries in this

area were modified to ensure that the boundaries were applied to only the low points along the road. These changes made no substantial change to the flooding patterns indicating that the flooding through the nursing home is not sensitive to the boundary setup.

Additional site inspections undertaken by the project team confirmed that the flooding indicated by the hydraulic model in and around the nursing home is consistent with the local topography. The site topography is also consistent with the underlying DEM and the flooding patterns in this region are expected to replicate the existing flooding patterns.