SITE STORMWATER MANAGEMENT PLAN

PROPOSED PORTARLINGTON DEVELOPMENT

CORNER OF GEELONG-PORTARLINGTON ROAD

AND BATMAN ROAD, PORTARLINGTON

July 2015

Insight Planning Consultants Pty Ltd

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1. INTRODUCTION

Water Technology has been engaged by Insight Planning Consultants Pty Ltd to prepare a Site Stormwater Management Plan for the proposed development in Portarlington at the corner of Geelong-Portarlington Road and Batman Road.

This report focuses on surface stormwater quantity and quality treatment to mitigate any potential impacts due to the proposed development. The report provides an introduction to the site and the proposed plan for development as well as the proposed strategy for managing stormwater in relation to both stormwater volumes and stormwater quality. Discussions with the City of Greater Geelong have been conducted to inform the strategy within this report.

2. SITE OVERVIEW

The site, located in Portarlington, east of Geelong, is a parcel of land bound by Geelong-Portarlington Road to the west, Allens Road to the east, Tower Road to the north and Batman Road to the south. The site is shown in Figure 2-1 below. The site is currently used for agricultural purposes, specifically for growing olives. The site is bound by existing residential areas to the north and west and agricultural areas to the east and south.

The highest elevation of the site is around 76m AHD, located in the south east corner of the site at the corner of Tower Road and Allens Road. The site slopes generally in a north westerly direction towards Geelong-Portarlington Road. The site drains to Geelong-Portarlington Road at three locations – the first at the corner of Batman Road (where existing surface levels are around 36 m AHD), the second at Waterview Close (approximately 30 m AHD) and the third at Tower Road (approximately 39 m AHD).

An existing 525 mm diameter RCP drains the existing flows under Geelong-Portarlington Road at the corner of Batman Road. There is a 750 mm diameter RCP at the Waterview Close intersection and also a 300 mm diameter RCP at the intersection of Tower Road.

All existing drainage is directed down via existing Council drainage pipes towards the intersection of Point Richards Road and Tower Road, and continues down Point Richards Road in a northern direction and discharges into Port Phillip Bay.

A catchment plan for the site is shown in Figure 2-4. It is assumed that external flows from the agricultural areas south of the site will flow down Batman Road with majority of flows bypassing the site except for minor flows diverted through an existing road culvert which are presumed to enter the site. External flows from the agricultural areas to the east of the site fill flow into the site down Allens Rd and then Pidgon Road and ultimately draining into the proposed retention basins within the site.

Figure 2-1 – Subject Site Locality Plan

Figure 2-2 – Elevations throughout the area

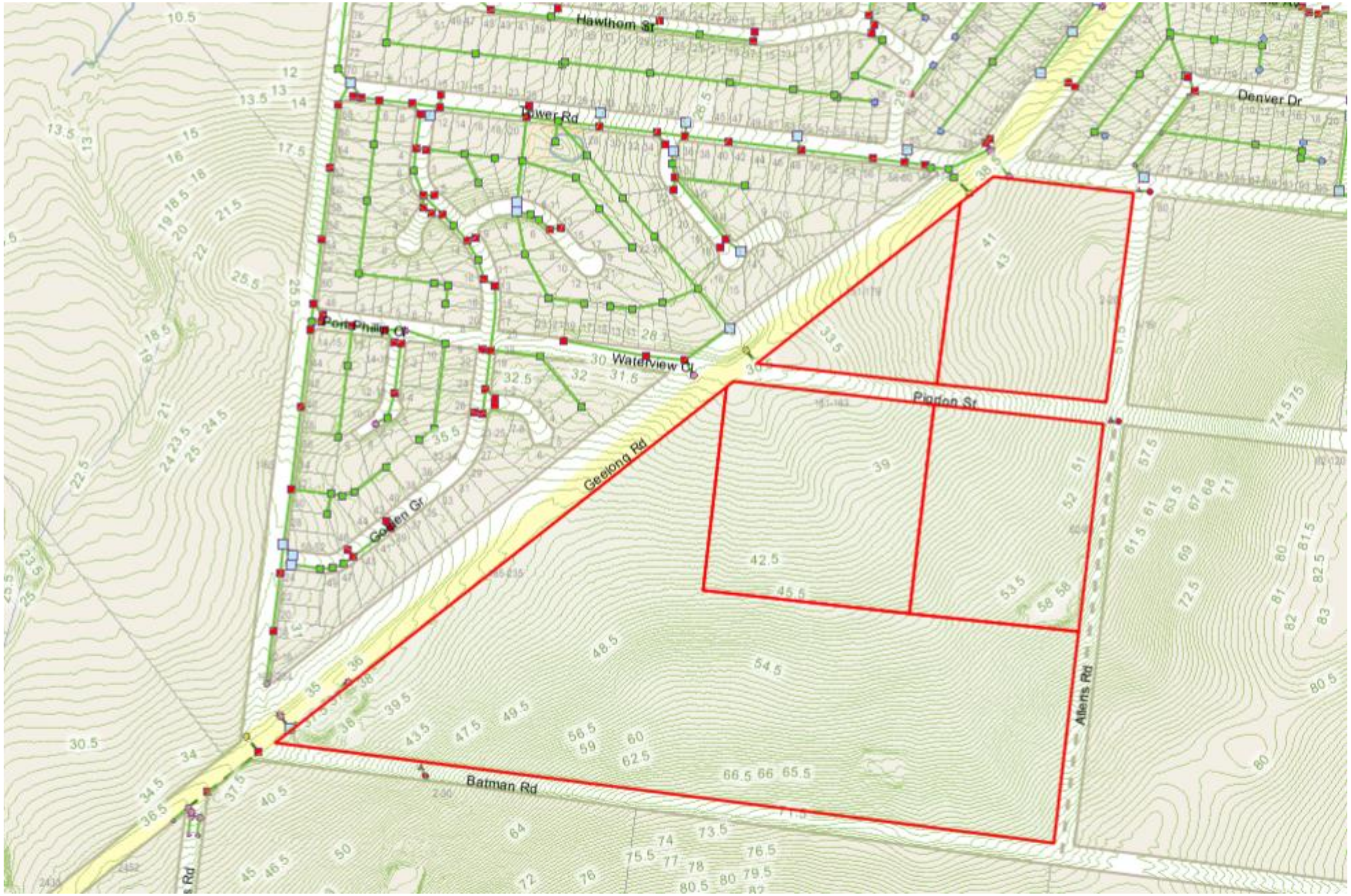

Figure 2-3 – Existing Drainage Assets

(Source: City of Greater Geelong)

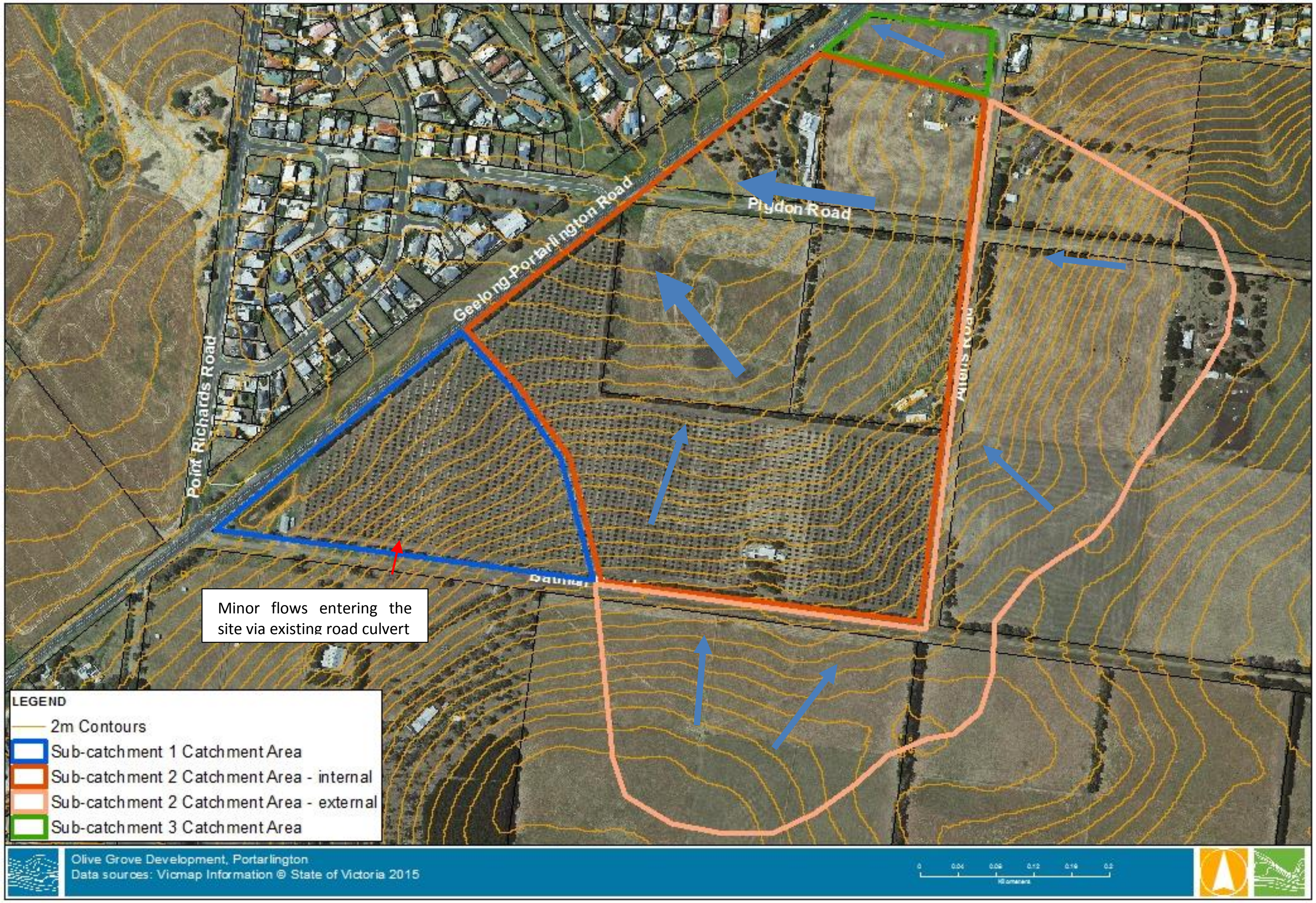

Figure 2-4 – Catchment Area Plan

3. SITE ANALYSIS

3.1 Study Hydrology

The hydrologic analysis of the site has been undertaken using the modelling program RORB. Models were run for existing, developed and mitigated conditions to determine the flooding mechanisms across the site and to size the flood retention features. Details of the RORB modelling are provided in Appendix A.

3.1.1 Existing Conditions

The pre-development conditions model represents the site under current conditions, draining to three main points along Geelong-Portarlington Road. Currently the site is an undeveloped agricultural area with a few small agricultural properties located within it, with a gravel section of Pigdon Street crossing the site from Allens Road to Geelong-Portarlington Road.

As mentioned above, the site drains to three main locations at Geelong-Portarlington Road. The individual catchment areas will be referred to as:

- Sub-catchment 1 – corner of Geelong-Portarlington Road and Batman Road
- Sub-catchment 2 – corner of Geelong-Portarlington Road and Waterview Close
- Sub-catchment 3 – corner of Geelong-Portarlington Road and Tower Road

The flood behaviour of the site under existing conditions for each sub-catchment is outlined below:

Sub-catchment 1

The flows from this portion of the site discharge into an existing 525 mm diameter RCP that crosses under Geelong-Portarlington Road at its intersection with Batman Road.

Flows from the external catchment to the south flow towards the site. The majority of these flows travels west along Batman Road and does not enter the site. It is assumed however that under existing conditions a small portion of these flows enters the site after crossing Batman Road via an existing 300 mm diameter culvert crossing. It is assumed that majority of flows from this catchment keep flowing along Batman Road towards the intersection.

The peak 100 Year ARI existing flows leaving the site was 0.39 m^3/s for the critical 9 hour storm duration. Peak flows for the other durations are shown in Table 3-1.

Sub-catchment 2

The flows from this portion of the site discharge into an existing 750 mm diameter RCP that crosses under Geelong-Portarlington Road at its intersection with Waterview Close.

Flows from the external catchment to the east flow towards the site. The flows currently enter the site along Allens Road and traverse the site, ultimately flowing towards the Geelong-Portarlington Road culvert.

The peak 100 Year ARI existing flows leaving the site was 1.22 m^3/s for the critical 9 hour storm duration. Peak flows for the other durations are shown in Table 3-1.

Sub-catchment 3

The flows from this portion of the site discharge into an existing 300 mm diameter RCP culvert that crosses under Geelong-Portarlington Road at its intersection with Tower Road.

There are no external flows that enter the site at this point as the external flows from the agricultural areas to the east are assumed to flow within Tower Road north towards Denver Drive.

The peak 100 Year ARI existing flows leaving the site was 0.07 m^3/s for the critical 2 hour storm duration. Peak flows for the other durations are shown in Table 3-1.

Table 3-1 – Peak 100 Year ARI Flows leaving the site under Existing Conditions

Duration	Sub-Catchment 1 (m^3/s)	Sub-Catchment 2 (m^3/s)	Sub-Catchment 3 (m^3/s)
10min	0.05	0.06	0.01
15min	0.13	0.20	0.04
20min	0.20	0.31	0.05
25min	0.25	0.41	0.05
30min	0.28	0.50	0.06
45min	0.34	0.70	0.06
1hr	0.35	0.82	0.07
1.5hr	0.34	0.93	0.07
2hr	0.35	0.98	0.07
3hr	0.35	0.98	0.05
4.5hr	0.34	0.95	0.06
6hr	0.36	1.02	0.06
9hr	0.39	1.23	0.05
12hr	0.34	1.09	0.05
18hr	0.25	0.79	0.03
24hr	0.29	0.97	0.03
30h	0.20	0.73	0.03
36h	0.21	0.76	0.02
48h	0.24	0.73	0.03
72h	0.14	0.47	0.02

3.1.2 Developed Conditions

The proposed development plan is shown in Appendix B. The land parcel is proposed for standard density residential development.

A comparison of the existing and developed conditions (unmitigated) peak flows is shown in Figure 3-1.

In unmitigated conditions, peak 100 year ARI flows leaving Sub-catchment Area 1 have increased from 0.39 m^3/s to 1.63 m^3/s following development of the site.

Peak 100 year ARI flows leaving Sub-catchment Area 2 have increased from 1.22 m^3/s to 5.07 m^3/s following development of the site.

Peak 100 year ARI flows leaving Sub-catchment Area 3 have increased from 0.071 m^3/s to 0.33 m^3/s following development of the site.

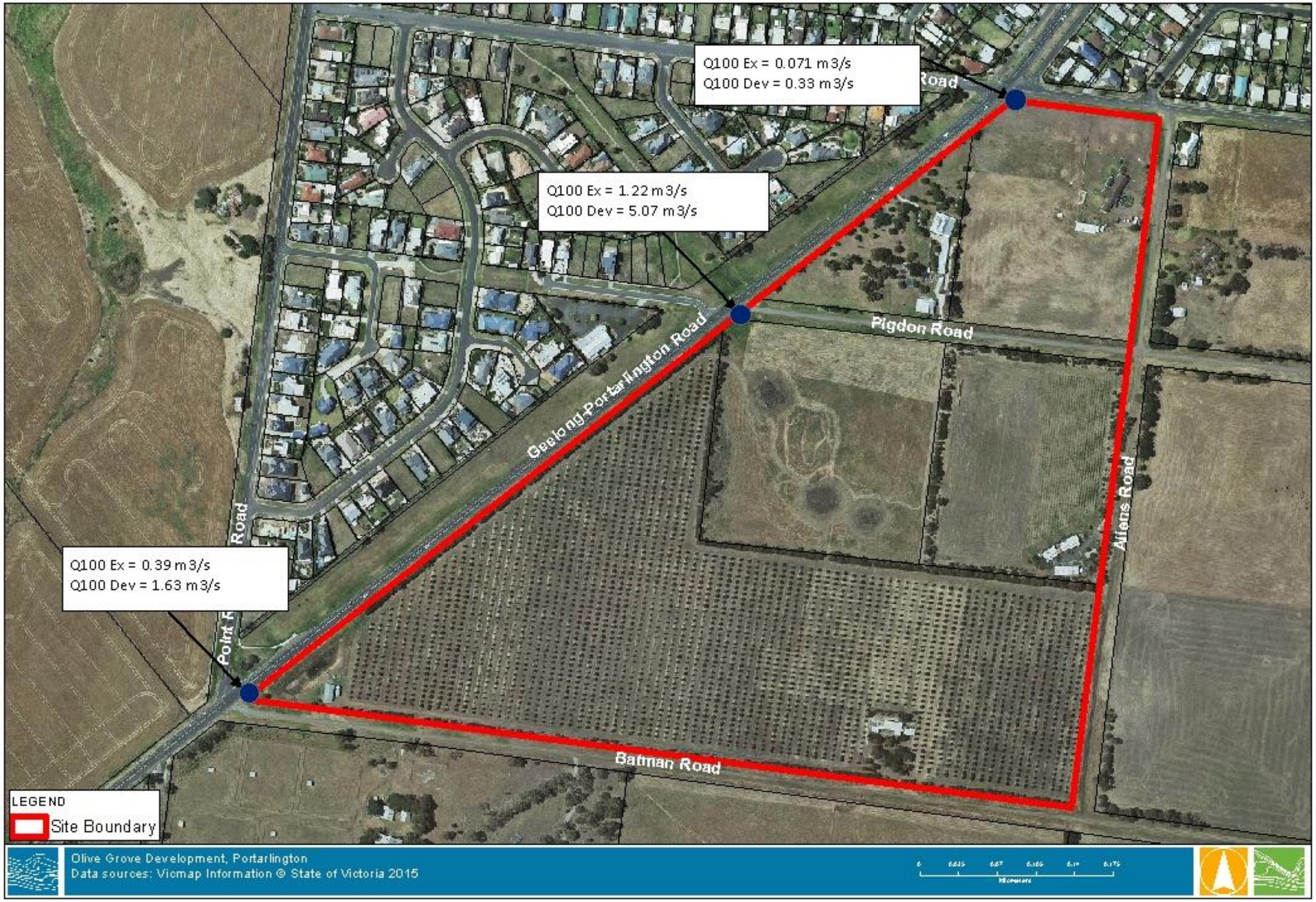

Figure 3-1 – Comparison of Existing and Developed Peak 100 year outflows from the site

3.1.3 Retarding Basin Sizing

Three retarding basins are proposed within the open space areas for sub-catchments 1, 2 and 3 to retard development peak flows leaving the site back to existing conditions.

The developed conditions RORB model was updated to include three retarding basins at the outlet of each sub-catchment. The basins were iteratively designed to ensure that the peak 100 year ARI flows matched existing peak flow conditions. The results of the mitigated RORB modelling included storage sizings is shown below in Table 3-2.

Table 3-2 – RORB Mitigation Scenario Results Summary

Sub-catchment	Existing Peak 100 year ARI Flow (m^3/s)	Mitigation Peak Flow (m^3/s)	Storage Required (m^3)
Sub-catchment 1	0.39 (9hr)	0.39 (2hr)	1,050
Sub-catchment 2	1.22 (9hr)	1.21 (9hr)	6,520
Sub-catchment 3	0.071 (2hr)	0.069 (1hr)	185

Retarding Basin Sub-catchment 1

Currently an existing farm dam is located at the lowest point of Sub-catchment 1. It is proposed to utilise this existing infrastructure to accommodate the new flood storage and water quality requirements. It is proposed to utilise the existing dam embankment to provide the storage, however prior to construction the suitability of the dam embankment must be checked by a suitability qualified geotechnical engineer.

Minor and major flows will enter the retarding basin as piped flows and overland flows respectively from the contributing residential areas upstream of the retarding basin. The retarding basin has been designed to have ensure it can cater for the 100 year ARI flows, with the existing informal spillway at the northern edge of the existing embankment to be utilised for events greater than the 100 year ARI.

The 9 hour storm was found to be critical for flood storage in the proposed basin, with a storage volume of **1,050 m^3** required.

The proposed basin parameters are as follows:

- Base level: 35.575 m AHD (1/2 TED Depth)
- 100 Year ARI flood level: 36.85 m AHD
- Spillway Level: 37.50 m AHD
- Normal Water Level: 35.40 m AHD
- Top of Extended Detention of Sediment Pond: 35.75 m AHD

It is proposed that the outlet of the basin will connect to the existing Council 525 mm diameter pipeline within Portarlington Road. No detailed design has been undertaken on the outlet configuration of the basin, only preliminary outlet sizing and checks on existing Council drainage inverts and capacities has been done. It was found through this process that the above proposed design levels allow an outlet design to be determined later during the more formal detailed design stages.

The stage-storage relationship for the basin is shown in

Table 3-3 and is based on the design contours shown in Figure 4-1.

Table 3-3 – Retention Basin 1 Stage-Storage Relationship

Depth	Stage (m AHD)	Flood Storage (m^3)	Comments
	35.40	0	Normal Water Level
	35.50	0	
0.00	35.58	0	Half Top Extended Detention Depth
0.02	35.60	13	
0.13	35.70	69	
0.17	35.75	99	Top Extended Detention Depth
0.22	35.80	129	
0.32	35.90	200	
0.42	36.00	277	
0.52	36.10	360	
0.63	36.20	447	
0.72	36.30	537	
0.82	36.40	632	
0.92	36.50	730	
1.03	36.60	833	
1.13	36.70	938	
1.18	36.75	993	
1.22	36.80	1048	
1.28	36.85	1104	Q100
1.33	36.90	1161	
1.43	37.00	1277	
1.53	37.10	1397	

Retarding Basin Sub-catchment 2

Minor and major flows will enter the retarding basin as piped flows and overland flows respectively from the contributing residential and agricultural areas upstream of the retarding basin. The retarding basin has been designed to have ensure it can cater for the 100 year ARI flows, with flows greater than the 100 year ARI to flow from the basin into the Portarlington Road-Pidgon Street intersection when flood levels exceed 31.0 m AHD. No formal spillway is proposed as no embankment is proposed.

The 9 hour storm was found to be critical for flood storage in the proposed basin, with a storage volume of **6,520 m³** required.

The proposed basin parameters are as follows:

- Base level: 23.235 m AHD (1/2 TED Depth)
- 100 Year ARI flood level: 30.80 m AHD
- Spillway Level: 31.00 m AHD
- Normal Water Level: 29.15 m AHD
- Top of Extended Detention of Wetland: 29.50 m AHD

It is proposed that the outlet of the basin/wetland will connect to the existing Council 750 mm diameter pipeline within Portarlington Road. No detailed design has been undertaken on the outlet configuration of the basin/wetland, only preliminary outlet sizing and checks on existing Council drainage inverts and capacities has been done. It was found through this process that the above proposed design levels allow an outlet design to be determined later during the more formal detailed design stages.

It should be noted that the basin design was based on proposed levels and a conceptual stage-storage volume calculated based on 1 in 5 batter slopes to the natural surface. The stage-storage relationship for the basin is shown in Table 3-4.

Table 3-4 – Retention Basin 2 Stage-Storage Relationship

Depth	Stage (m AHD)	Flood Storage (m^3)	Comments
	29.15	0	Normal Water Level
	29.20	0	
	29.30	0	
0.00	29.33	0	Half Top Extended Detention Depth
0.07	29.40	203	
0.18	29.50	493	Top Extended Detention Depth
0.28	29.60	839	
0.38	29.70	1235	
0.48	29.80	1664	
0.57	29.90	2106	
0.68	30.00	2562	
0.78	30.10	3031	
0.88	30.20	3515	
0.98	30.30	4012	
1.08	30.40	4524	
1.18	30.50	5051	
1.28	30.60	5592	
1.38	30.70	6149	
1.43	30.75	6434	
1.48	30.80	6720	Q100
1.58	30.90	7307	
1.68	31.00	7909	

<u>Retarding Basin Sub-catchment 3</u>

Minor and major flows will enter the retarding basin as piped flows and overland flows respectively from the contributing residential and agricultural areas upstream of the retarding basin. The retarding basin has been designed to have ensure it can cater for the 100 year ARI flows, with flows greater than the 100 year ARI to flow from the basin into the Portarlington Road road reserve when flood levels exceed 38.6 m AHD. No formal spillway is proposed as no embankment is proposed.

Outflows from the basin, up to the 100 year ARI event, will discharge via an outlet pipe and discharge into the 300 mm diameter Council pipe running under Geelong-Portarlington Road.

The proposed basin parameters are as follows:

- Base level: 38.00 m AHD
- 100 Year ARI flood level: 38.60 m AHD

The 1 hour storm was found to be critical for flood storage in the proposed basin, with a storage volume of **185 m^3** required.

The stage-storage relationship for the basin is shown in Table 3-5 and is based on the design contours shown in Figure 4-3.

Table 3-5 – Retention Basin 3 Stage-Storage Relationship

Depth	Stage (m AHD)	Flood Storage (m^3)	Comments
0.0	38.0	0	
0.1	38.1	7	
0.2	38.2	30	
0.3	38.3	59	
0.4	38.4	95	
0.5	38.5	137	
0.6	38.6	186	Q100
0.7	38.7	242	
0.8	38.8	304	
0.9	38.9	370	
1.0	39.0	439	

3.1.4 Downstream Capacity

The capacity of the downstream system has also been checked to confirm available capacity within the existing network. The 5 year ARI flows for each Sub-catchment were obtained from the RORB model. The capacity of the downstream pipes were also calculated, as shown in Table 3-6. It can be seen that there is ample capacity within the existing Council network to cater for the 5 year ARI flows from the proposed development.

Table 3-6 – Downstream Capacity Check

Sub-catchment	5 Year ARI Flow (m^3/s)	Downstream Pipe Diameter (mm)	Downstream Pipe Capacity (m^3/s)
Sub-catchment 1	0.104	525	0.64
Sub-catchment 2	0.259	750	1.88
Sub-catchment 3	0.015	300	0.15

Each basin outlet is proposed to cater for the 100 year ARI flow. Further design of the connection between each basin and the existing Council infrastructure will need to be considered. A simple surcharge pit arrangement could be employed to ensure the 100 year flows are conveyed downstream of each basin. . The existing Council overland flow paths downstream of each of the basins are more than adequate to cater for the 100 year ARI flow rates discharging from each of the basins.

Figure 3-2 below presents the existing, developed and mitigated 100 year peak outflows for each Sub-catchment.

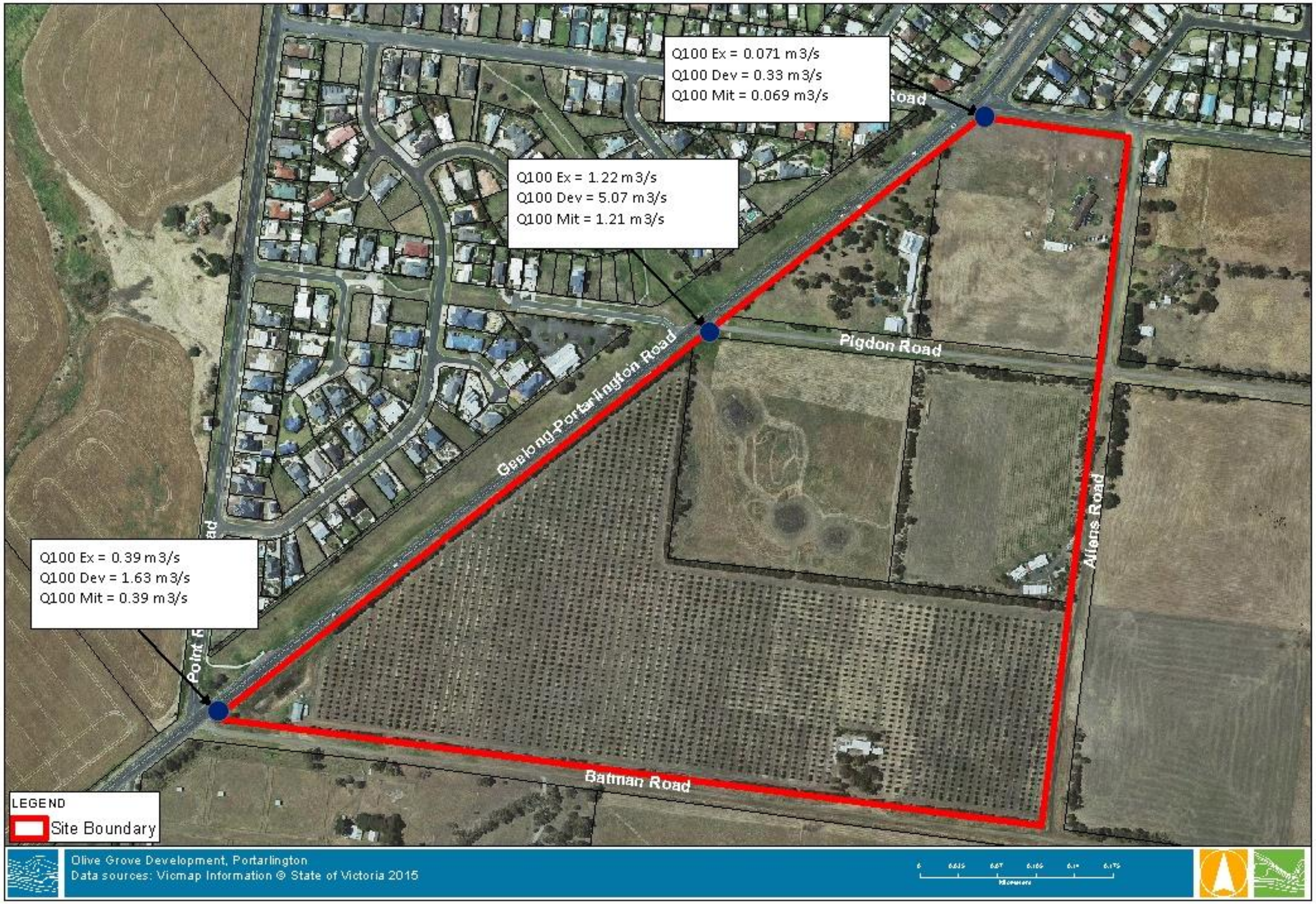

Figure 3-2 – Comparison of Existing, Developed and Mitigated Peak 100 year outflows from site

3.2 WSUD Option Analysis

3.2.1 Preferred Option

The treatment train components were optimised using the MUSIC (Model for Urban Stormwater Improvement Conceptualization) modelling program. The predicted performance of the treatment train has been assessed against the targets described in the Urban Stormwater Best Practice Guidelines.

Streetscape WSUD features were assessed as a potential option for the development. In similar developments, streetscape WSUD may take the form of linear swales running along road reserves or rain gardens within road reserves. Given the steep nature of the road reserve, combined with the requirement for significant flood storage within the development (and hence the availability of space for a wetland treatment), best practice will be met through the use of a treatment device at the outlet of each of the three sub-catchments.

For Sub-catchment 1 a sedimentation basin is proposed. A treatment train consisting of a sediment pond and a wetland within the retarding basin is proposed for Sub-catchment 2. Sub-catchment 3 on the other hand requires a vegetated swale due to the small contributing area and limited available space for WSUD implementation.

3.2.2 MUSIC Modelling and Treatment Train Sizing

A MUSIC model (Version 6.0.1) was established in line with the Melbourne Water MUSIC Guidelines with the proposed WSUD features input for the site. The layout of the MUSIC model is shown in Figure 3-3 below.

The model was run using 6 minute rainfall data from the Geelong North rainfall station (station number 87133) for the 1985 reference year, as per the Geelong MUSIC Guidelines developed for the City of Greater Geelong.

The catchment breakup for the model was based on the proposed development layout and was presented in Figure 2-4.

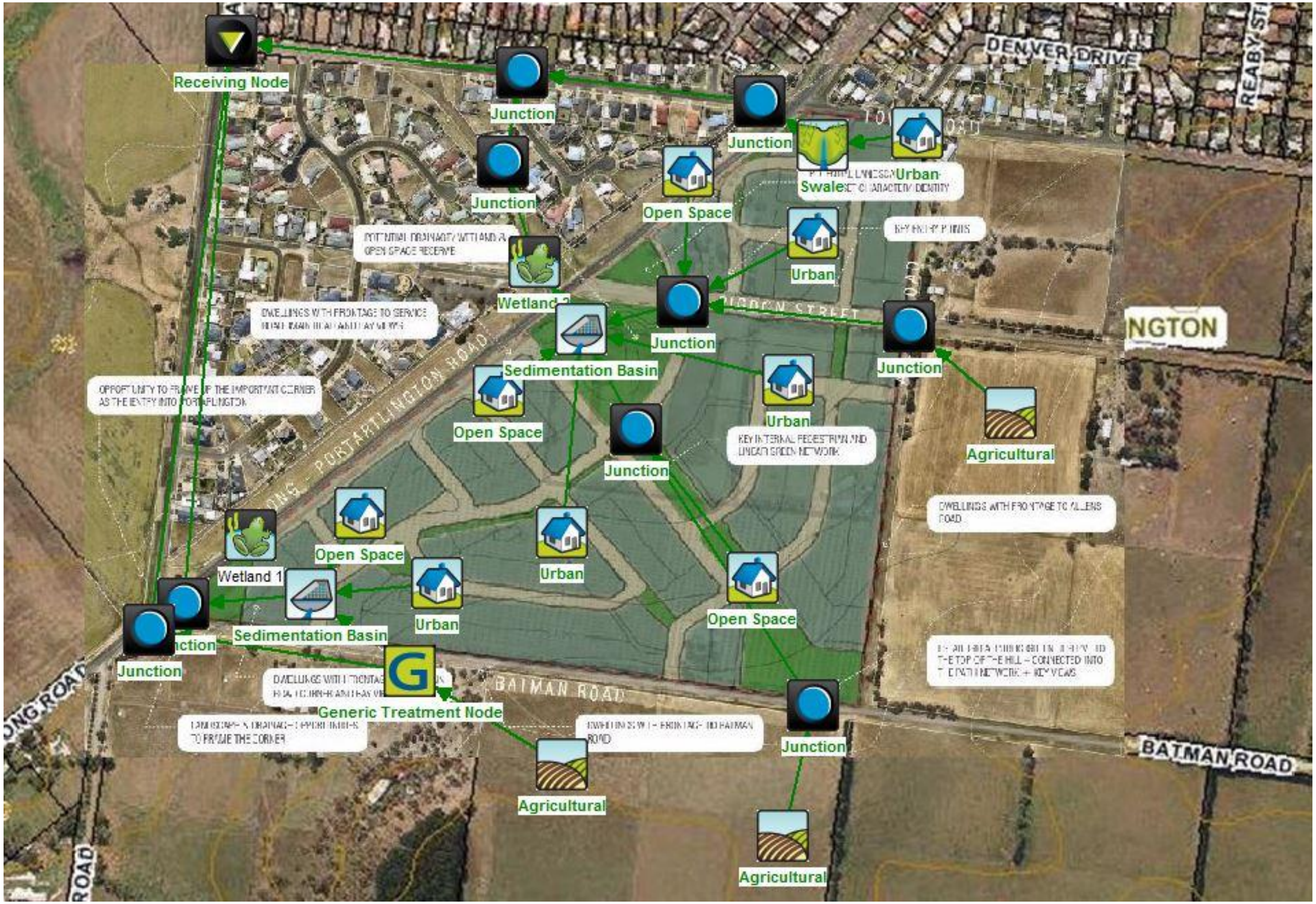

Figure 3-3 – MUSIC model layout for developed mitigated conditions

WSUD for Sub-catchment 1

For Sub-catchment 1 a sedimentation basin of an area of 466 m^2 was deemed sufficient to be able to treat the runoff from the contributing areas to the desired standards. The best practice target treatment requirements is as follows:

- 80% retention of the typical urban annual load runoff for Total Suspended Solids (TSS)
- 45% retention of the typical urban annual load runoff for Total Phosphorus (TP)
- 45% retention of the typical urban annual load runoff for Total Nitrogen (TN)

The WSUD sizing requirements have been determined, as shown in Table 3-7.

Table 3-7 – WSUD Sizing Data – Sub-catchment 1

Component	Sediment Pond
Area (m^2)	466
Extended detention depth (m)	0.35
Permanent pool volume (m^3)	500

The modelled treatment train performance of this sedimentation basin is as per Table 3-8.

Table 3-8 – Pollutant Removal Rates – Sub-catchment 1

Pollutant	Loads Produced by the Development (kg/yr)	Load Removed (kg/yr)	Percentage Treatment
TSS	3,770	2,827	102%
TP	5.9	4.8	83%
TN	43.0	19.6	46%

It must be noted that the sedimentation basin also treat the upstream agricultural catchment which results in a greater than 100% reduction.

The MUSIC results show that the proposed WSUD features meets the water quality requirements for the development.

WSUD for Sub-catchment 2

For Sub-catchment 2 a sedimentation basin combined with a wetland is required to treat the runoff from the contributing areas to best practice standards.

The WSUD sizing requirements have been determined, as shown in Table 3-9.

Table 3-9 – WSUD Sizing Data – Sub-catchment 2

Component	Sediment Pond	Wetland
Area (m^2)	750	1,900
Extended detention depth (m)	0.35	0.35
Permanent pool volume (m^3)	850	760

The modelled treatment train performance of this wetland and sedimentation basin is as per Table 3-10.

Table 3-10 – Pollutant Removal Rates – Sub-catchment 2

Pollutant	Loads Produced by the Development (kg/yr)	Load Removed (kg/yr)	Percentage Treatment
TSS	11,095	10,470	94%
TP	23.4	18.9	81%
TN	170.2	77	45%

The MUSIC results show that the proposed WSUD features meets the water quality requirements for the development.

WSUD for Sub-catchment 3

The WSUD sizing requirements for the Vegetated Swale for Sub-catchment 3 have been determined, as shown in Table 3-11.

Table 3-11 – WSUD Sizing Data – Sub-catchment 3

Component	Swale
Length (m)	60
Bed Slope (%)	0.50
Base Width (m)	3
Top Width (m)	9
Vegetation Height (m)	0.10

The modelled treatment train performance of this vegetated swale is as per Table 3-12.

Table 3-12 – Pollutant Removal Rates – Sub-catchment 3

Pollutant	Loads Produced by the Development (kg/yr)	Load Removed (kg/yr)	Percentage Treatment
TSS	499	37	92%
TP	1.02	0.3	68%
TN	6.9	4.3	39%

The removal potential of Total Nitrogen of a vegetated swale this size is below the desired target of 45%. However, it has been determined that this additional TN (0.4 kg/yr) can be achieved by the additional treatment that the wetland within Sub-catchment 2 provides. The three sub-catchments all drain to the same place. Therefore, this offset would mean that when assessing the outflows from the site as a whole, pollutant loads would all meet best practice standards. The calculations to confirm this are presented below.

The amount of TN that enters the vegetated swale, according to the MUSIC modelling, is 6.94 kg/yr. The vegetated swale treats 38.6% of this TN which is 4.26 kg/yr. An additional 6.4% still needs to be removed, which equates to 0.44 kg/yr.

The MUSIC modelling shows that the wetland within Sub-catchment 2 removes 77 kg/yr of TN. The total loads that are generated from the developed areas are 170.2 kg/yr. If the additional TN that is required to be removed from Sub-catchment 3 is added onto this, the total loads that are generated from the developed areas are 170.64 kg/r. This in turn, in order to achieve 45% treatment of TN, is required to treat 76.8 kg/yr. As the actual TN removal that the wetland undertakes is 77 kg/yr, therefore the additional TN reduction requirement is satisfied within this treatment system.

The overall modelled treatment train performance of all of the systems within the site is shown in Table 3-13 below.

Table 3-13 – Pollutant Removal Rates – overall

Pollutant	Total Loads Produced by the Development (kg/yr)	Load Removed (kg/yr)	Percentage Treatment
TSS	15,364	13,334	87%
TP	30.3	24.0	79%
TN	220.1	100.9	46%

4. CONCEPTUAL DESIGN

4.1 Minor and Major Flow Management

Stormwater runoff from the site will be directed into the proposed wetland and retarding basins for Sub-catchments 1 and 2 and into the vegetated swale for Sub-catchment 3.

Outflows from the retarding basin, up to the 100 year ARI, will discharge into existing pipes that run under Geelong-Portarlington Road. The capacity of these pipes has been checked and was deemed large enough to be able to receive the flows up to the 5 year ARI. This arrangement allows for most of the runoff from the development site to discharge into the existing drainage network.

Flows above the 5 year ARI may surcharge the Council network depending on the final connection design of the basin outlets. The existing Council overland flow paths downstream of each of the basins are more than adequate to cater for the 100 year ARI flow rates discharging from each of the basins.

Minor Flows

- Minor flows from the site will be conveyed through drainage pipes and overland flows into the retarding basins

Major Flows

- Major flows are to be catered for along the developments road network. It should be noted that calculations on capacity or flood modelling of the overland flows along the roads has not been undertaken and will be a requirement of Functional and Detailed design.

4.2 Retarding Basin and WSUD Features

The concept plan and levels associated with the each of the two retarding basing/treatment designs are shown in Figure 4-1 and Figure 4-2 however these are subject to functional and detail design.

During functional design of the system, the following components should be considered:

- Total area including batters and freeboard;
- Retarding basin embankments;
- Access for maintenance;
- Sediment drying areas within the reserve
- Sediment pond and wetland outlet structure;
- Potential for wetland bypass systems;
- Wetland macrophyte zone bathymetry.

For Sub-catchment 3 a vegetated swale is proposed. The 100 Year ARI flood level is 38.60 m AHD. The vegetated swale has an invert level of 38.15 m AHD grading to the outlet level of 38.00 m AHD, as per Figure 4-3.

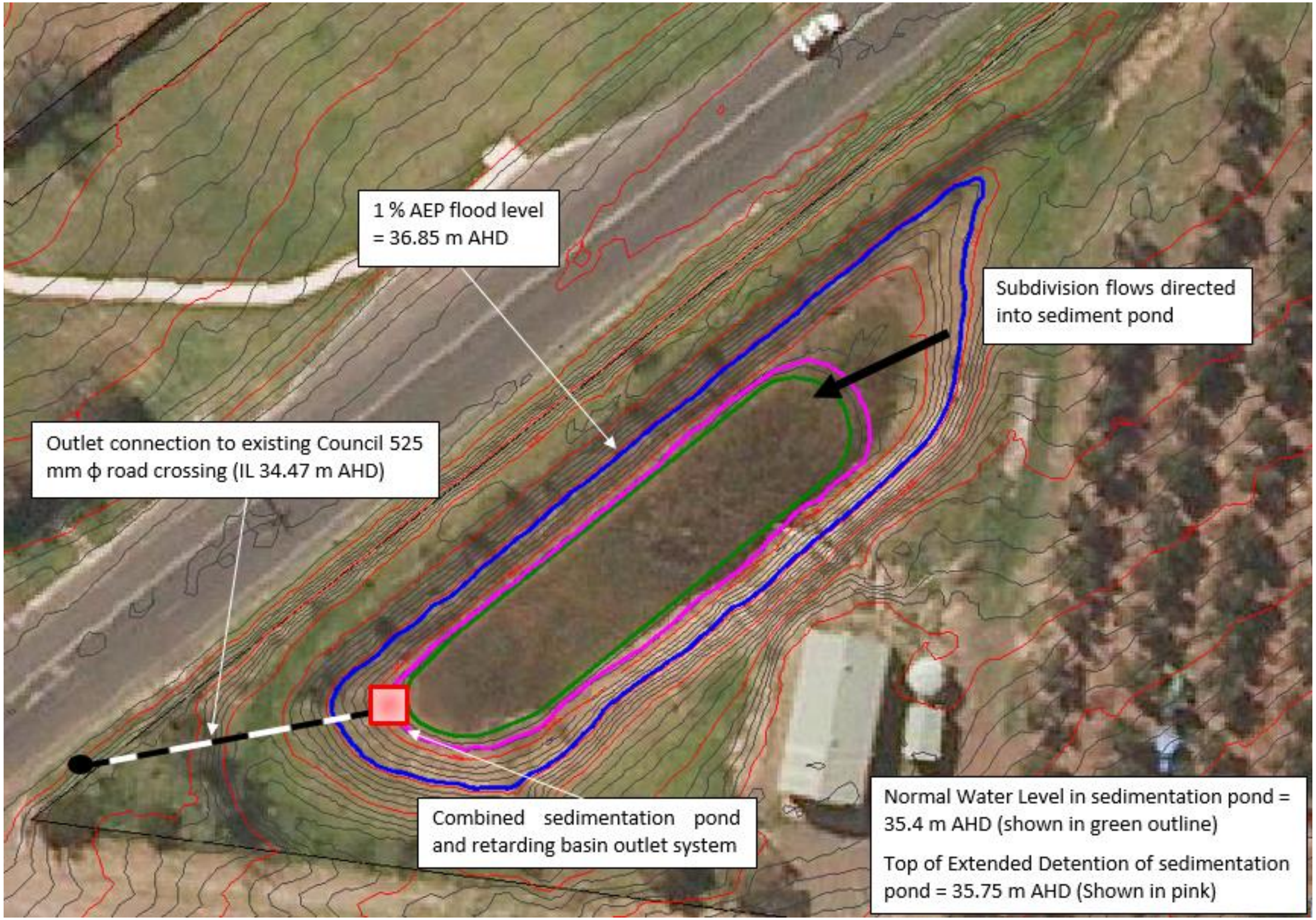

Figure 4-1 – Concept Design of Retention Basin at Sub-catchment 1

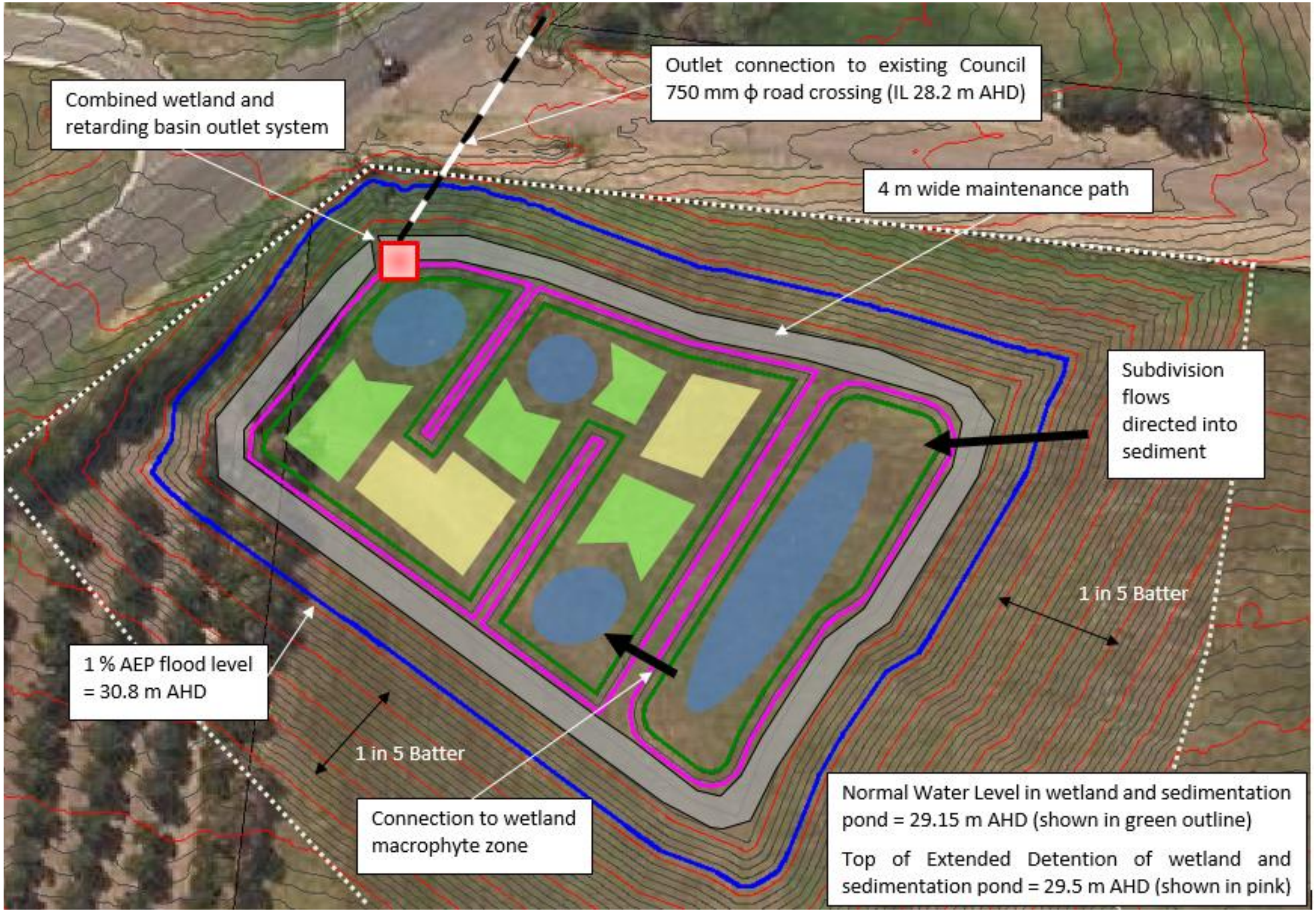

Figure 4-2 – Concept Design of Retention Basin at Sub-catchment 2

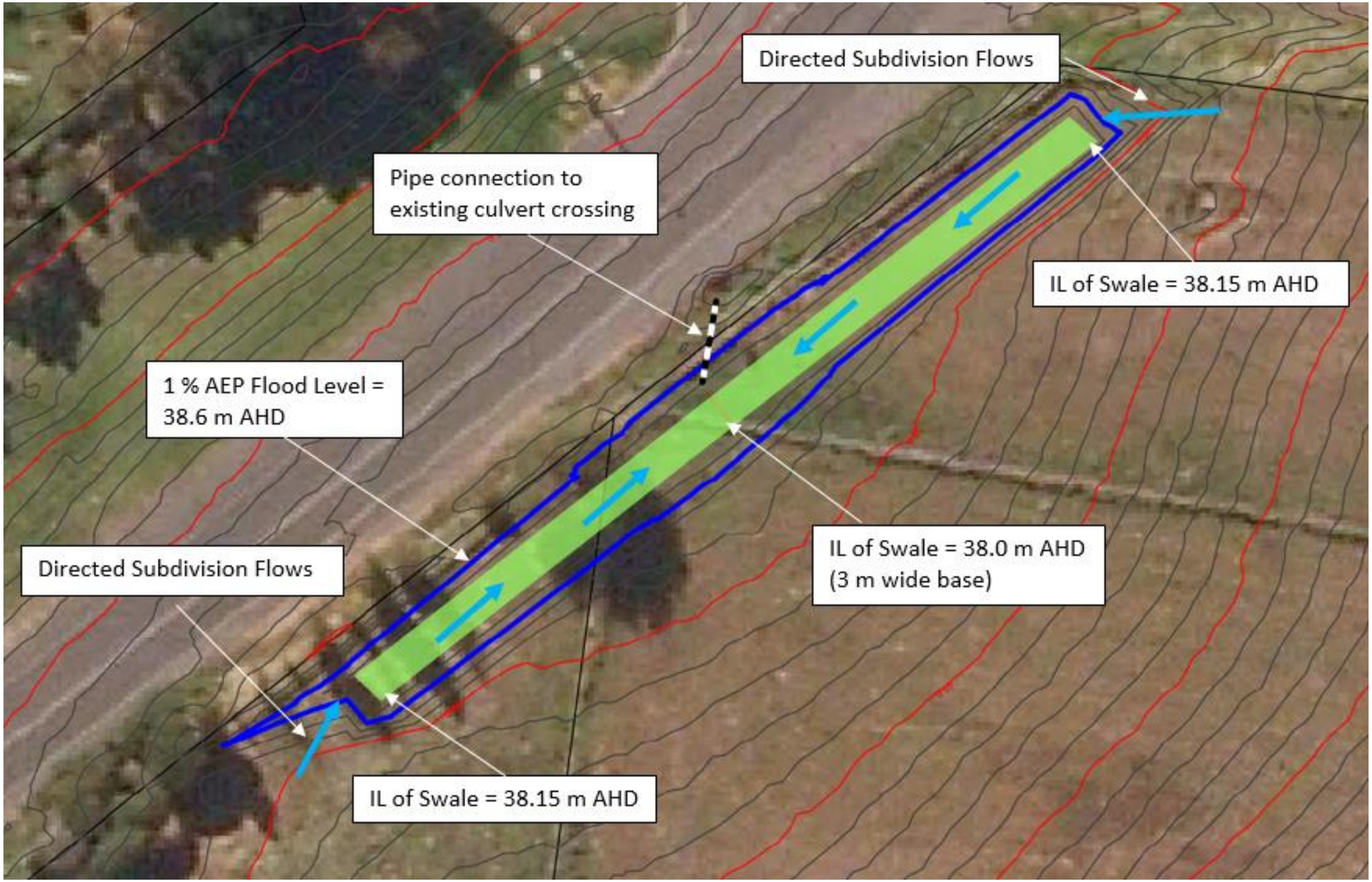

Figure 4-3 – Concept Design of Vegetated Swale at Sub-catchment 3

5. CONCLUSIONS

This Site Stormwater Management Plan for the proposed development in Portarlington considered works required to manage water quantity and quality for the development. The proposed stormwater works include:

- A 1,050 m^3 retarding basin located in the open space reserve in the most western part of the site – near the corner of Geelong-Portarlington Road and Batman Road. A sedimentation basin is required in this location in order to treat the runoff from the contributing areas to best practice standards.
- A 6,520 m^3 retarding basin located in the open space reserve in the central part of the site - corner of Geelong-Portarlington Road and Waterview Close. A sedimentation basin and a wetland is required at this location.
- A 60 m long and 9 m wide vegetated swale located in the open space reserve in the most north eastern part of the site – near the corner of Geelong-Portarlington Road and Tower Road.

The proposed retarding basins will retard 100 year ARI peak flows leaving the site back to existing conditions.

The proposed works at the site (retention basins) ensure that the flood peaks are at or below existing levels, therefore we believe that this will not exacerbate existing flooding issues downstream of the site, including any interaction with possible storm surges.

The capacity of the existing downstream Council pipe network has been checked and has sufficient capacity to cater for the 5 year ARI flows leaving each of the basins.

The proposed water quality features, namely the sedimentation basins, wetland and the vegetated swale allow the best practice water quality requirements for the development to be met.

APPENDIX A RORB MODELLING

RORB Modelling

RORB

RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce the hydrograph. The model is areally distributed, nonlinear, and applicable to both urban and rural catchments. It was used to size the retarding basins.

Fraction Impervious Data

The FI values for each sub catchment were applied as detailed in Melbourne Water's MUSIC Guidelines.

Under existing conditions most of the site was assigned a fraction impervious (FI) value of 0.1 for all of the agricultural areas of the site. Under developed conditions, the fraction impervious value across the developed areas was set to 0.6.

Model Reconciliation

A conceptual existing conditions RORB model was built draining the agricultural areas towards the three discharge points at Geelong-Portarlington Road. The model was run for a 100 year storm event with an initial loss of 20mm. The k_c value was adjusted to calibrate to the 100 year ARI peak flow, obtained for each sub-catchment. The following parameters were adopted:

- m: 0.8
- Initial loss: 20mm
- Runoff Coefficient: 0.6 (100yr)

The adjusted k_c parameters were as per the table below:

Drainage Point	k_c
Sub-catchment 1	0.45
Sub-catchment 2	1.11
Sub-catchment 3	0.15

The RORB model was then amended to include the proposed developed residential sub-catchments, as well as the agricultural areas flowing into the drainage points from outside the subject site. The initial loss was reduced to 10 mm in the modelling.

A mitigated developed option was then created, with retention basins inserted at the outlet of each sub-catchment. These were then sized to be able to take the additional flows coming off the residential areas, making sure that the outflows from the retention basins did not exceed the outflows under pre-developed conditions.

The mitigated RORB model layout plan is presented below.

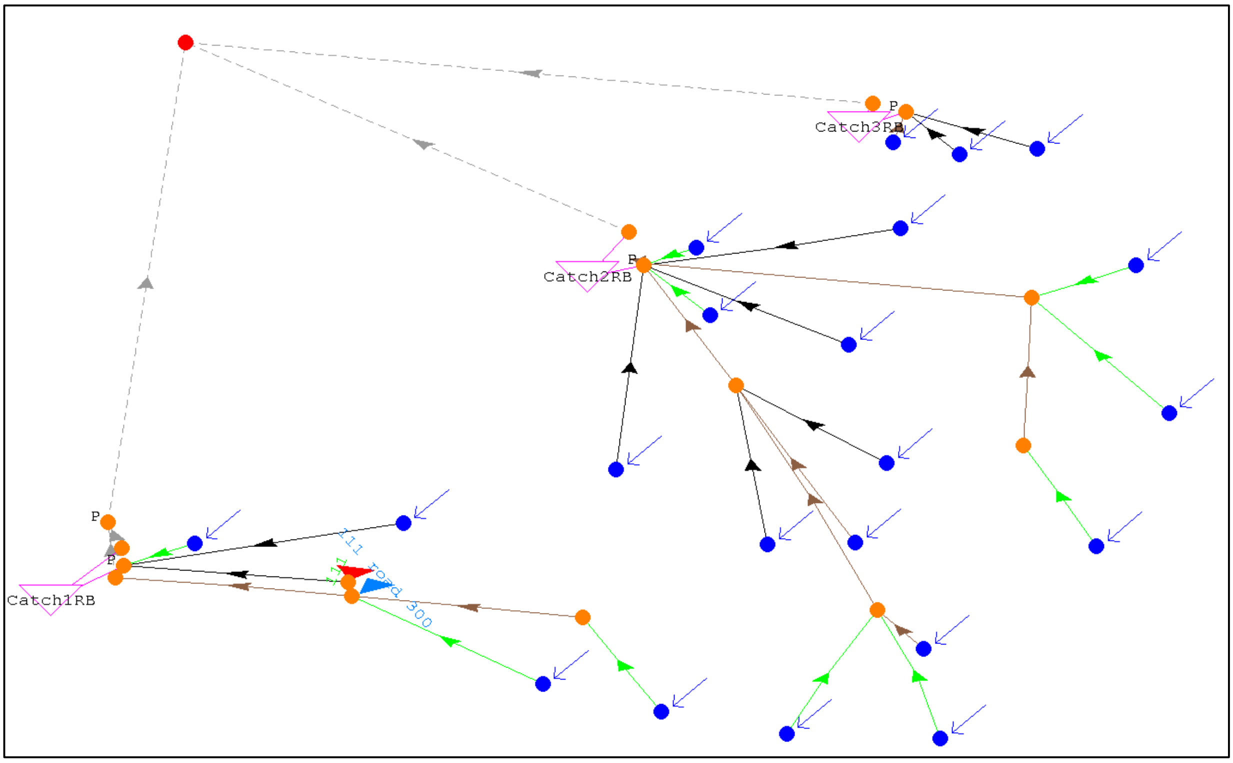

Figure 5-1 – RORB model schematisation

APPENDIX B PROPOSED DEVELOPMENT PLAN

A3
SCALE: 1:1000

ILLUSTRATIVE CONCEPT

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WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS
ABN: 60 093 377 283
ACN: 093 377 283

Memo

To:	Jason Black	**From:**	Thomas Cousland
Organisation:	Insight Planning	**Date:**	24/12/2015
Job Title:	Geelong-Portarlington Road Development, Portarlington		
Subject	Downstream Wetland Water Balance Modelling		

Dear Jason,

Water Balance modelling has been undertaken to determine the impact (if any) that the proposed development bounded by Batman Road, Allens Road, Tower Road and Portarlington Road has on the existing wetland system located downstream of the development on Point Richards Road (See Figure 1).

The key inputs and assumptions made in the water balance modelling were as follows:

- Storage details determined from LiDAR aerial survey of the wetland site.
- Seepage rate of 0.36 mm/hr applied to the wetland system (consistent with a clay based soil)
- Pluviographic rainfall data from 1980 to 1990 from a nearby gauge was input into a MUSIC model of the system to generated wetland inflows for the existing and developed scenario.
- Average monthly evaporation values used
- Existing Fraction Imperviousness value of 0.6 used in the modelling

Results of the water balance modelling is shown in Figure 2 and Figure 3.

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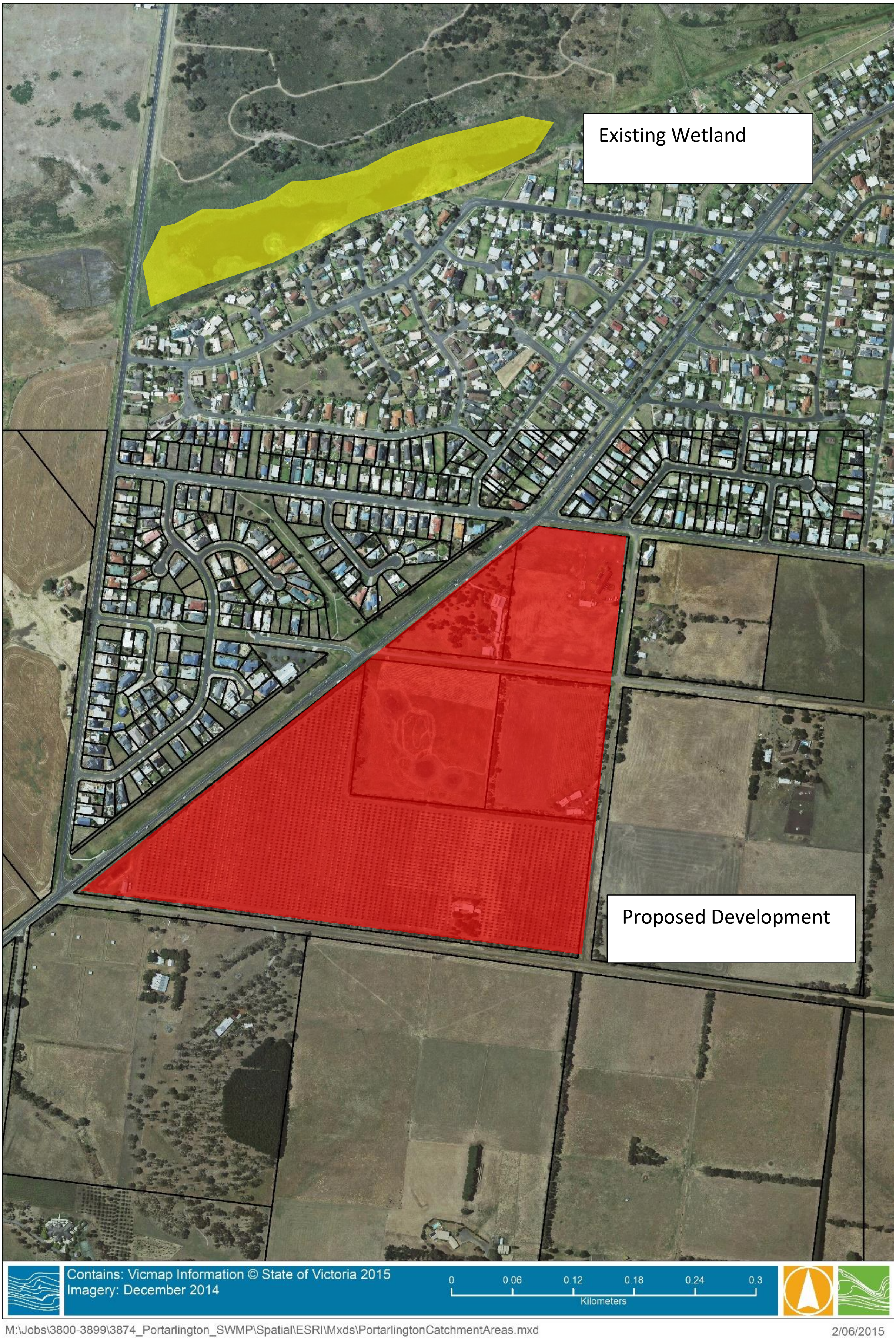

Figure 1 Location of Development and Wetland

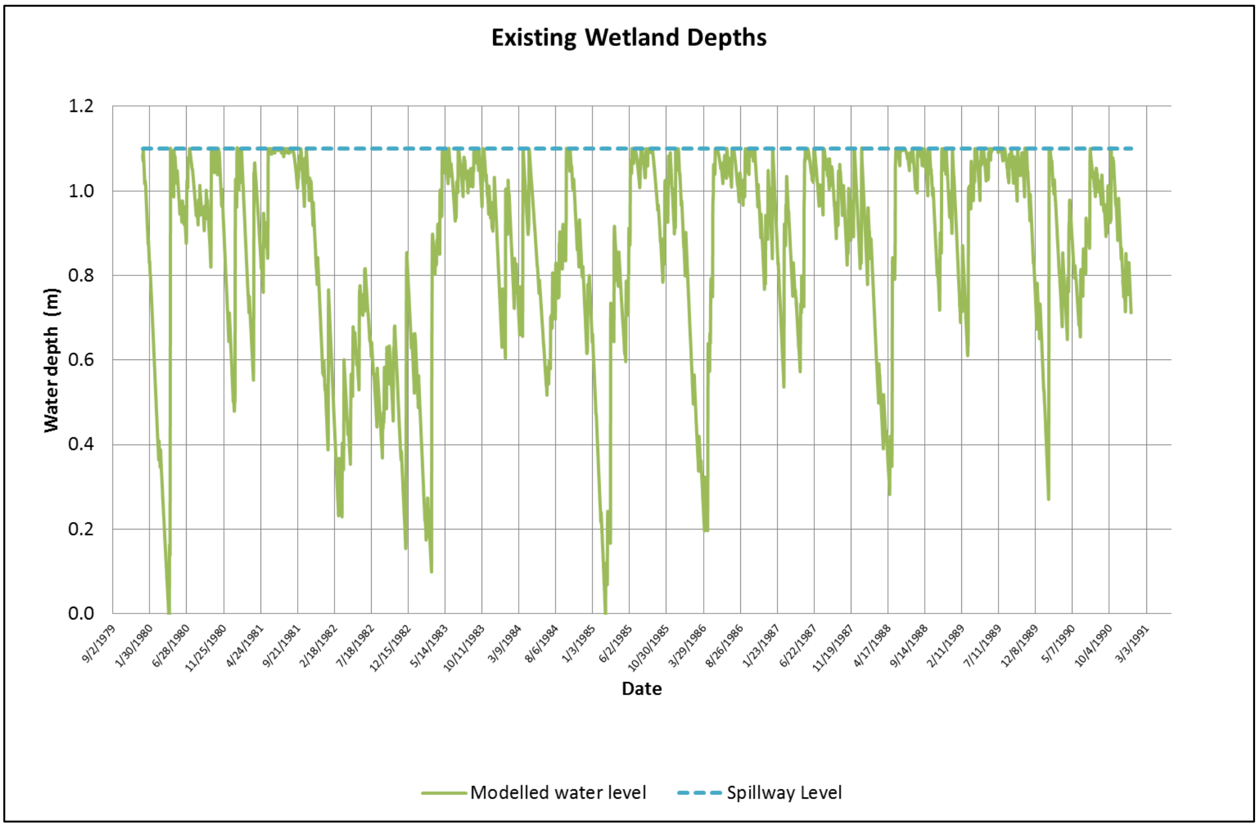

Figure 2 Existing Wetland Depths over Time

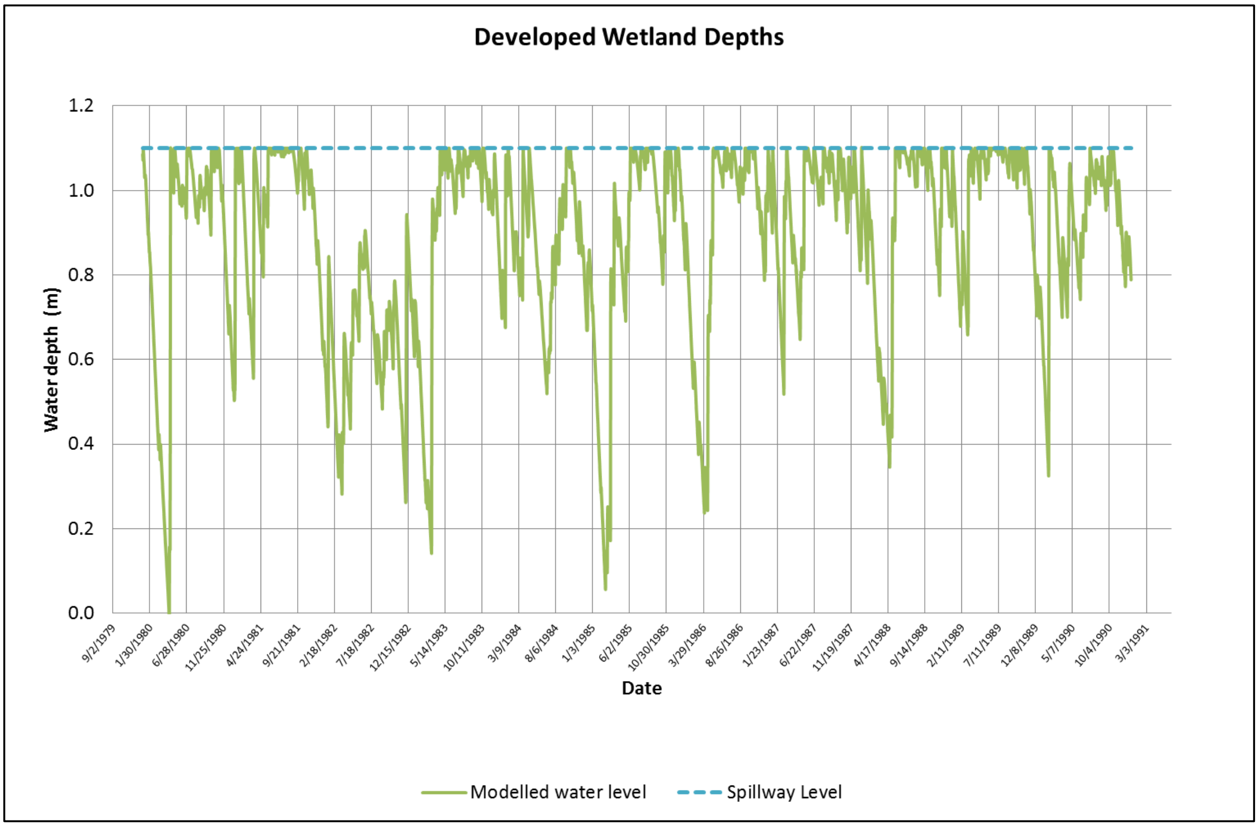

Figure 3 Developed Depths over Time

To the naked eye the results held within Figure 2 and Figure 3 appear to closely match each other. To determine more precisely the difference in water levels in the wetland between existing and developed, an exceedance curve for both scenario was developed for the wetland and is shown in Figure 4.

The exceedance curve illustrates that the difference in water level between the existing and developed scenarios is relatively low for all ranges of flow, with wetland inundation durations for both scenarios very similar.

The difference in water depths within the wetland between the scenarios range from 0 to 125 mm, with the average difference in water depth only 26 mm.

It is believe that this average increase in 25 mm will have negligible impacts on the wetland system

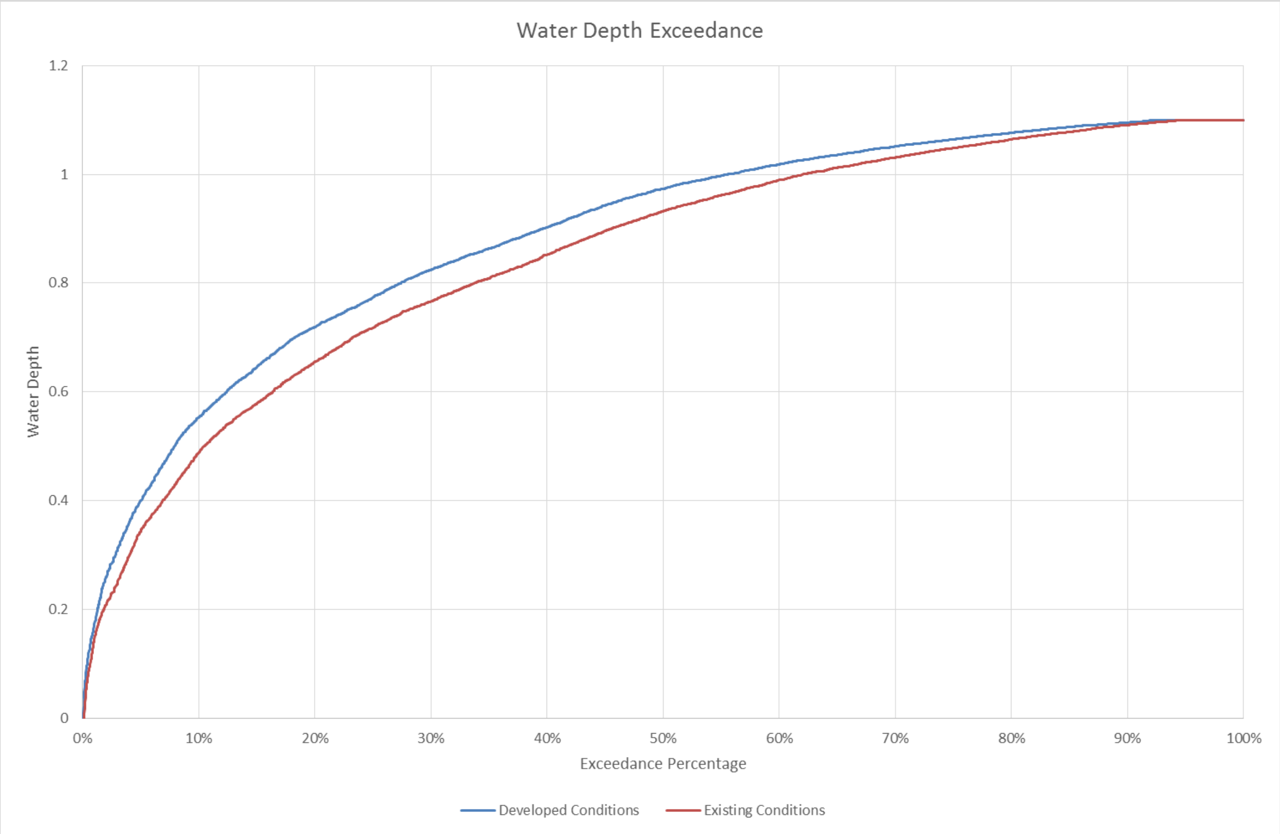

Figure 4 Wetland Exceedance Curve

To further illustrate the difference in volumes entering the wetland, Table 1 quantifies the additional water volume that will enter the wetland system. The increased number of flow days entering the wetland system is a result of the proposed water quality works within the proposed development which will elongate the flow hydrographs leaving the site resulting in longer trickle flows after rainfall events. All additional water volumes entering the wetland system will be treated to Best Practice requirements.

Table 1 Wetland Inflow Volumes

	Existing Conditions	Developed Conditions
Flow Days	162	222
Annual Inflow (ML)	221.7	259.0
Annual Overflow (ML/yr)	73.6	99.1

The results found within this initial water balance modelling indicate the increase runoff volumes from the development site will have negligible impacts on the wetland system in terms on water depths and inundation durations within the wetland. Generally wetland systems (including the flora and fauna within them) have a large ability to adapt to changes within upstream catchments, with fauna such as the growling grass frog (which are though to inhabit the wetland system) actually preferring deeper waters.

Considering the average increase water depth in the wetland system is 26 mm and that all additional water volumes entering the wetland system would be treated to Best Management Practice, it is believed that the upstream development will not have a detrimental impact on the existing wetland system located on Point Richards Road.

Regards

Water Technology Pty Ltd

Thomas Cousland

Senior Engineer