Attachment Four

Integrated Stormwater Management Plan

TGM

Engineering | Surveying | Planning

799 – 815 HENDY MAIN ROAD, MORIAC

MULTI-LOT SUBDIVISION

SITE STORMWATER MANAGEMENT PLAN

REPORT V01

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Client: John & Wendy Earl

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Document Location

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EXECUTIVE SUMMARY

TGM Group Pty Ltd has been engaged by John & Wendy Earl to develop a Site Stormwater Management Plan (SSMP) in response to *Schedule 14* of the *Development Plan Overlay* for land at 799 & 815 Hendy Main Road, Moriac.

The subject site is a 30.4 hectare land parcel identified in the Surf Coast Planning Scheme as Low Density Residential although currently agricultural farmland. A 54 lot rural subdivision has been proposed for the site.

The proposed subdivision will result in an increase in impervious surfaces resulting in an increase in stormwater runoff volumes and gross pollutants. The following site stormwater management plan (SSMP) provides guidance on future stormwater treatment systems required to deliver best practice outcomes to support such a development.

The objective of the stormwater management plan is to meet the conditions and requirements, set out by the Surf Coast Shire Council in *Schedule 14* for stormwater management. Stormwater mitigation systems are designed to ensure that stormwater quality and quantity targets are met. The targets for this study are:

1. Best Practice reductions for Water Quality

→ 80% reduction in Suspended solids (SS)

→ 45% reduction in total nitrogen (TN)

→ 45% reduction in total phosphorus (TP)

→ 70% reduction in gross pollutants (GP)

2. No-worsening stormwater peak discharges

→ During a 1% AEP design storm event.

(Lower events were not considered in this study as flows will be managed through staged discharge arrangements, designed at a later stage)

Two scenarios have been considered to analyse the effectiveness of the proposed design to manage stormwater runoff quality and quantity from the developed Hendy Main Street site.

Scenario 1: *Existing Conditions,* or 'base-case';

Scenario 2: *Developed Conditions (Includes sub-scenarios 2a and 2b)*.

An integrated systems approach was used to create the stormwater management plan for the development of the proposed subdivision site, by analysing the performance of the stormwater and water cycle management design options. This type of analysis is dependent on detailed inputs including topography, climate, geology, hydrology, stormwater quality and sound urban design principles.

The study shows that stormwater generated within the proposed subdivision site and contributing external catchments can be mitigated to meet 'best-practice' objectives with adoption of a vegetated swale treatment system and bio-retention or wetland facility. The reduction in pollutant loads achieved by the proposed treatment train systems can be seen in Table A.

Table A: Pollutant Reductions

Scenario 2 – Developed treatment train	Combined Site Reduction (%)	
Criteria	**Scenario 2a (Swales & Bio-retention)**	**Scenario 2b (Swales & Wetland)**
Total Suspended Solids	90.5	90.8
Total Phosphorus	65.6	70.9
Total Nitrogen	48	46.6
Gross Pollutants	100	100

Stormwater peak discharges generated by a 1% AEP storm event from the fully developed site can be managed to achieve a 'no-worsening' of pre-development discharges by integrating a detention basin into the proposed treatment train of each catchment. The detention basin system generates peak discharges as shown in Table B.

Table B: Peak Discharge

Catchment	1% AEP Discharge (L/s)	
	Existing	**Developed**
1	500	**499**
2	130	**130**

This study shows that the stormwater runoff can be effectively managed to ensure all stormwater targets and objectives can be met.

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1. INTRODUCTION

TGM Group Pty Ltd has been engaged by John & Wendy Earl to develop a Site Stormwater Management Plan (SSMP) in response to *Schedule 14* of the *Development Plan Overlay* for land located at 799 & 815 Hendy Main Road, Moriac.

The land is located within a broader rural context to the east of the Moriac Township approximately 18 km south west of Geelong. The land adjoins the Moriac Township and a Public Use - Education zone to the north and Farming zones to the north-east, east and south.

The study location is seen in Figure 1.1.

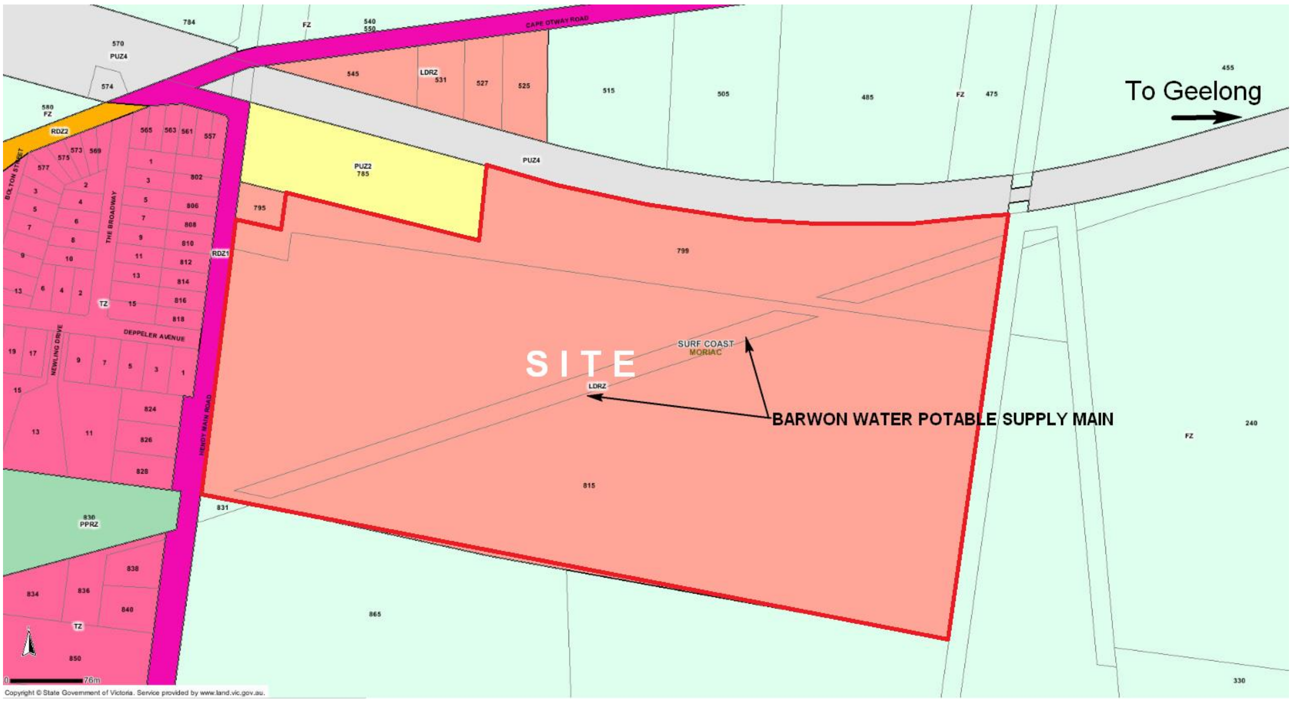


Figure 1.1: 799 & 815 Hendy Main Rd Study Area Location

The land is identified in the Moriac Structure Plan 2010 as Future Low Density Residential.

2. STUDY AREA

2.1 Site Description

The subject site is a 30.4 hectare land parcel located at 799 & 815 Hendy Main Road, Moriac. The site can be seen in Figure 2.1 below.

Figure 2.1: Subject Site – 799 & 815 Hendy Main Road, Moriac

Two dwellings and an outbuilding are located off Hendy Main Road, along the western site boundary. The remainder of the site has been cleared for agricultural purposes and is largely devoid of any development and significant areas of flora.

Existing vegetation within the site consists predominantly of pastoral grasses with a stand of trees through the centre of the site.

The site is relatively flat with minor depressions formed by stormwater flow across the site.

There is an existing Barwon Water potable supply trunk main easement that dissects the subject site from the north-east to the south-west.

2.2 Stormwater Catchment

The subject site is nestled between two tributaries of the Thompson Creek, to the south-east. The site is located along the south side of a ridgeline dividing the two catchments the result is that there is a small external catchment area that originates within the Moriac town centre, west of the site.

Analysis of the topography indicates that the site is not subject to flooding during regional storm events. Therefore, it is expected that external flows will be conveyed around the perimeter of the site in spoon drains, where possible, or through the site within the proposed drainage system. Therefore, the external catchments will not be reviewed further in this study.

The site location in relation to the Thompson Creek tributaries is depicted in Figure 2.2.

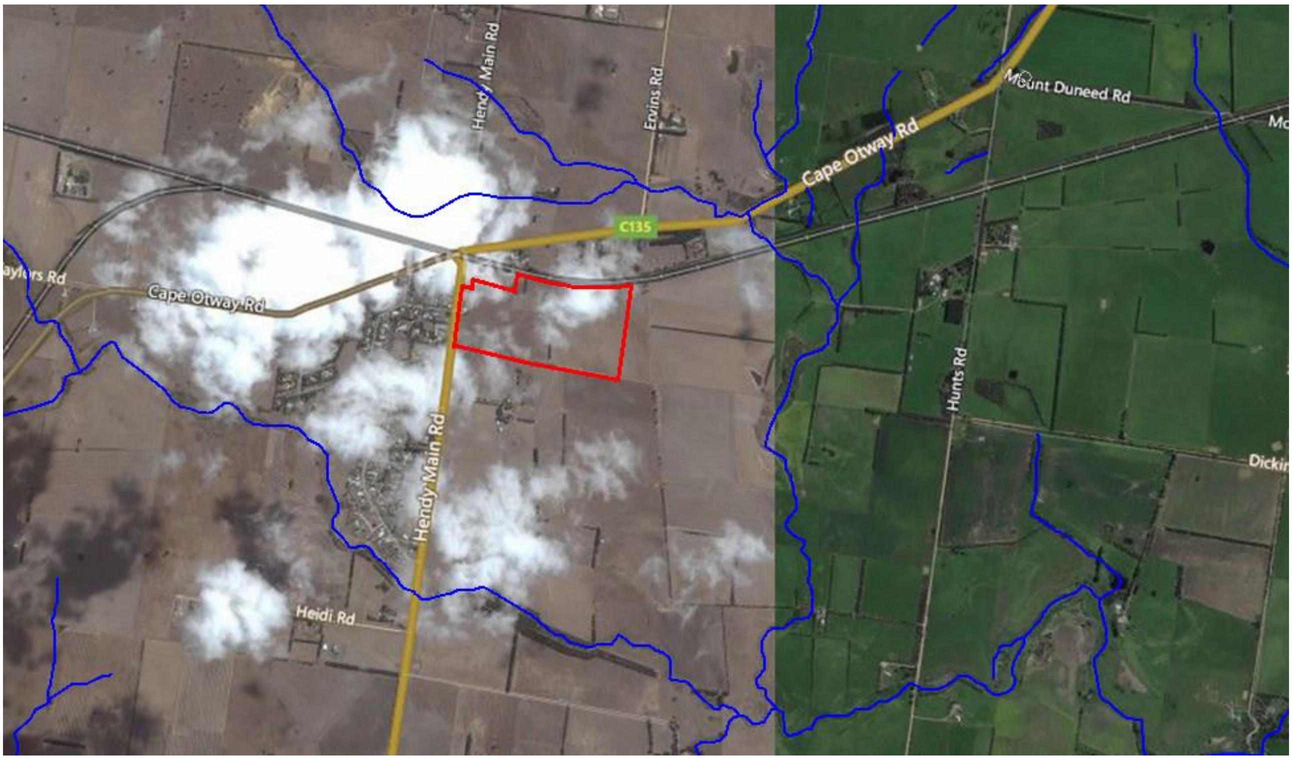


Figure 2.2: Site External Catchment and Waterway Proximity

2.3 Internal Stormwater Catchments

2.3.1 Existing Site

The overall existing site can be broken up into 2 sub-catchments according to topography and site discharge locations. The internal catchments are shown in Figure 2.3 and detailed in Table 2-1.

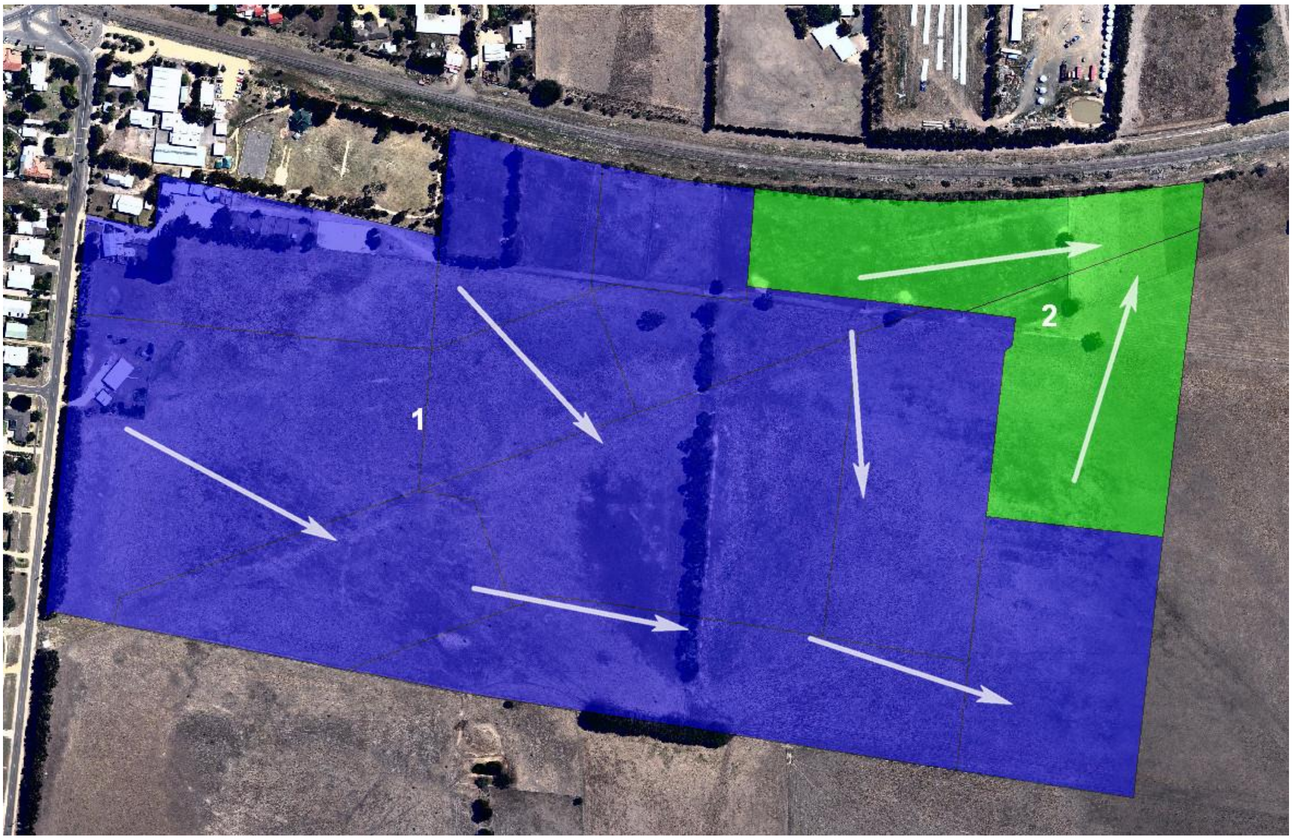


Figure 2.3: Internal Stormwater Catchments

Table 2-1: Catchment details

Catchment	Area (ha)	Point of Discharge
1	25.8	South-East Corner of site
2	4.6	North-East Corner of site

TGM Group has analysed LiDAR data of the site, allowing creation of a digital elevation model (DEM) and confirmation of catchment areas. The DEM is depicted in Figure 2.4.

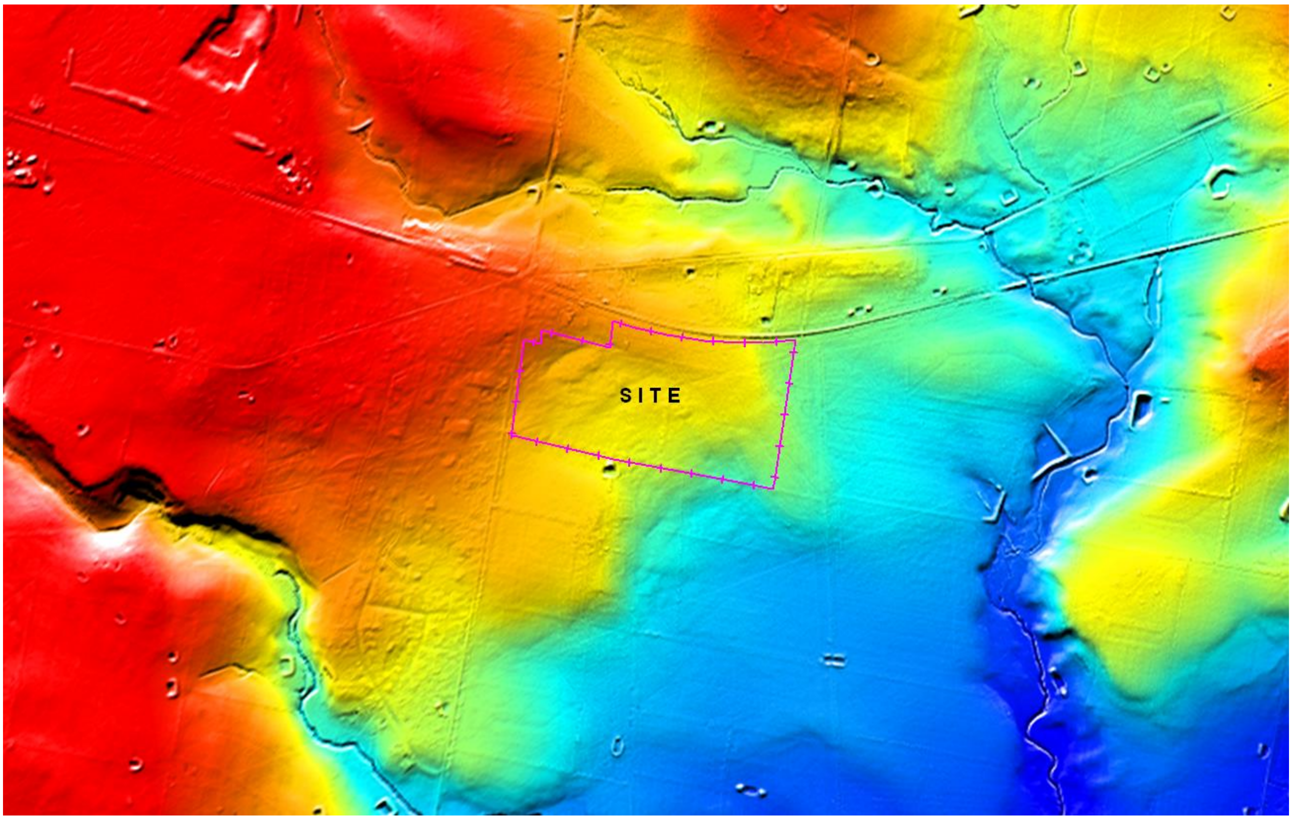


Figure 2.4: Study DEM – Topographic model

2.3.2 Developed Site

It is anticipated that the developed site will maintain grades to the two (2) existing discharge locations. These will be referred to as the Legal Point of Discharge (LPOD) for the development site. The developed layout is depicted in Figure 2.5.

The proposed rezoning of the site to Low Density Residential Zone (LDRZ) will result in proposed lot sizes ranging from 4,000 m² to just over 15,000 m². The proposed developed catchment is detailed in Table 2-2

Table 2-2: Developed Catchments

Catchment	Area (ha)	Land Use	Expected No. of Lots	Percentage Impervious (%)
Developed	30.4	Residential (LDRZ)	54	0.2

The proposed subdivision will increase the amount of impervious surfaces within the 30.4 ha hectare site by approximately 20%. The total impervious area in the developed situation will be approximately 6.08 hectares.

The increase in impervious surfaces results in a decrease in stormwater infiltration; this creates an increase in the stormwater runoff volumes and velocities generated within the site.

Stormwater mitigation measures will be required to counteract the impact of the proposed development upon the downstream catchments and receiving environment and achieve best practice objectives as defined in the Best Practice Environmental Management Guidelines.

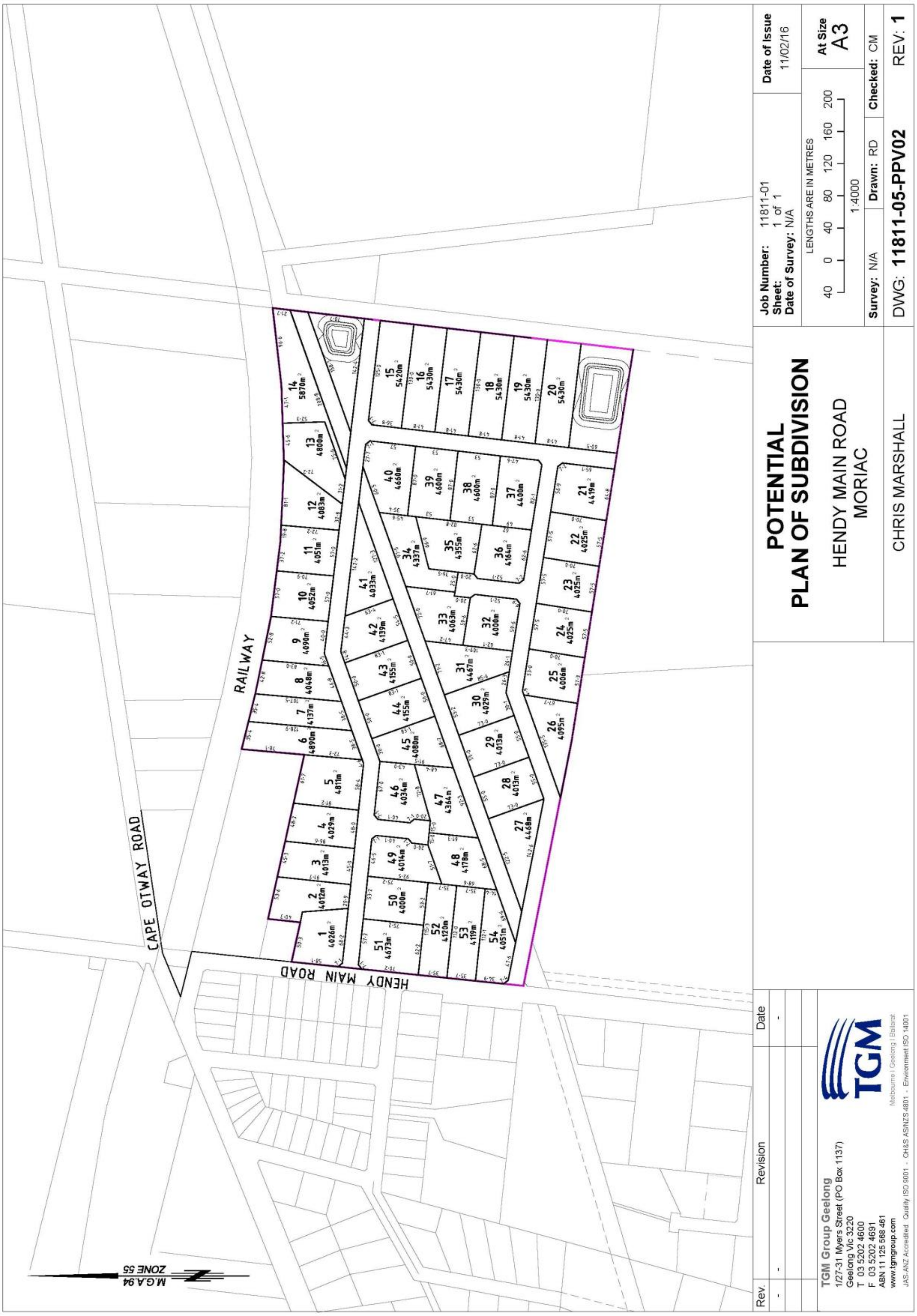


Figure 2.5: Proposed layout - Developed site

3. STORMWATER OBJECTIVES

The objective of the SSMP is to meet the conditions and requirements set in the planning application for stormwater management. These requirements ensure that appropriate design and stormwater mitigation is applied to ensure that stormwater quality and quantity targets are achieved and maintained.

3.1 Site Stormwater Objectives

1. Best Practice reductions for Water Quality

- i. 80% reduction in Suspended solids (SS)
- ii. 45% reduction in total nitrogen (TN)
- iii. 45% reduction in total phosphorus (TP)
- iv. 70% reduction in gross pollutants (GP)

2. No-worsening stormwater peak discharges

- i. Ensure pre-development discharges are maintained for the 1% AEP rainfall event.

Note: For this study only the discharge for the 1% AEP event was assessed; smaller AEP events can be mitigated through a staged outlet condition determined during detailed design.

4. MITIGATION OPTIONS

Stormwater mitigation systems will be assessed to understand how the developed site can achieve best practice reductions and stormwater objectives identified in Section 3.

4.1 Scenario 1 – Existing Conditions 'base-case'

The stormwater runoff characteristics from the existing site will be evaluated for the 1% AEP storm event. These values will form the basis for comparison and design for the developed treatment system.

4.2 Scenario 2 – Developed Conditions

Stormwater runoff generated within the developed site will be conveyed, as overland flows, via a grassed or vegetated swale system, through a series of water quality control measures and discharging into an end-of-line detention basin located at the existing legal point of discharge.

The grassed swales will be used to convey stormwater in lieu of pipes to provide water quality treatment for low events by removing coarse and medium sediment and allow additional infiltration into the soil.

The swales provide a disconnection of impervious areas from hydraulically efficient pipe drainage systems, resulting in slower travel times (times of concentration), reducing the impact of increased imperviousness within the catchment on peak flow rates.

During the low events the detention basins will act as sedimentation basins, providing further stormwater quality improvements, whilst mitigating stormwater discharge volumes during the larger events.

However, to achieve best practice reductions in stormwater contaminant loads a secondary treatment measure will need to be adopted in conjunction with the swale system. TGM has analysed the use fo bio-retention basins and wetlands to complete the treatment train. The combination of the swale systems with secondary treatment facilities are described in the following design scenario variations –

4.2.1 Scenario 2a – Bio-retention

Scenario 2a adopts bio-retention basins within the base of the detention basins to treat stormwater runoff up to and including the 1.5 year ARI event. The internal swale drainage system conveys the stormwater runoff within the development to the end-of-line bio-retention basin, where it undergoes final treatment prior to discharge.

4.2.2 Scenario 2b – Wetland

Scenario 2b adopts an end-of-line wetland within the base of the detention basin. Operation is similar to the Bio-retention system, however, required facility footprint is much larger.

5. METHODOLOGY

To create the stormwater management plan for the site, an integrated systems approach was used to analyse the performance of a range of stormwater and water cycle management options. This type of analysis is dependent on detailed inputs including topography, climate, geology, hydrology, stormwater quality and sound urban design principles.

5.1 Topography and existing infrastructure

The topographical data set used in this study to define the site and contributing catchment was composed of detailed LiDAR data and compiled into a digital elevation model by TGM group.

There is notable infrastructure present within the site, this being the Barwon Water potable water supply main, connecting the Moriac township to the water grid. The depth (R.L) and cover of water main is unknown. Identification of this will be required during detailed design to confirm design constraints.

The sites northern boundary is bounded by a V-Track railway line. No works are proposed to be undertaken within the V-Track reserve.

5.2 Climate Processes

Australia experiences one of the most variable climatic regimes on the planet. The extreme natural variation of the continent's climate includes cyclic patterns of droughts and floods throughout recorded history.

An understanding of the temporal and spatial variation of climate processes is essential for an accurate analysis of hydrology. Further, the need to effectively include climate processes in analysis of water resources and flooding has increased with the onset of climate change. It is now widely understood that the earth's climate system has been subject to significant warming that will increase the variability of climate processes. [1,2]

Three (3) long term rainfall sequences were analysed for the development at 799 & 815 Hendy Main Road in Moriac. Pluviograph rain gauges situated at Geelong North (BOM station number 87133); Buckley (BOM station number 87124); and Gnarwarre (BOM station number 87162), located 21.2 km, 8.3 km and 10.7 km from the development site respectively. The long term rainfall sequences for the three sites are shown in Figure 5.1.

[1] DSE (2008). Climate change in Victoria: 2008 summary – the Victorian climate change adaptation program.
[2] Coombes P., Colegate M., et. al(2012). Greater Melbourne systems model – modelling in support of the Living Victoria Ministerial Advisory Council.

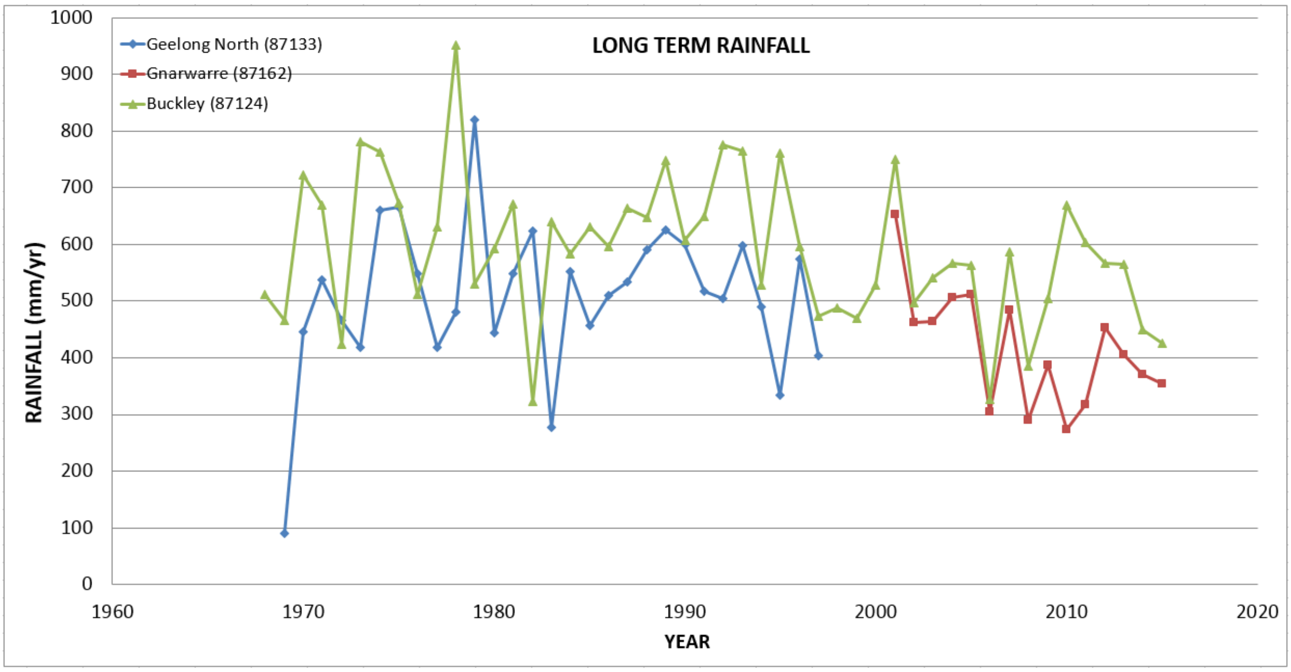


Figure 5.1: Long term annual rainfall records around the Hopes Plain Road site

A summary of the rainfall gauges is shown in Table 5-1, below.

Table 5-1: Rainfall gauges reviewed in this analysis

Record	Start Date	End Date	Average Annual Rainfall (mm/yr)	Length (years)	Distance from Subdivision (km)
Geelong North rainfall gauge (87133)	1969	2015	508.0	29	21.2
Buckley rainfall gauge (87124)	1968	2015	591.1	48	8.3
Gnarwarre rainfall gauge (87162)	2001	2015	415.9	15	10.7

To conduct a robust hydrological analysis, evapotranspiration must also be analysed in the assessment. Evapotranspiration is the term used for the transfer of water from the land surface to the atmosphere as water vapour. It is based on different climate zones, which vary throughout the Australian continent due to the availability of water and vegetation.

Data presented by the Bureau of Meteorology[3] enabled the extraction of average monthly areal potential evapotranspiration (PET) that correlated to the development catchment. This data was employed for the hydrologic and water quality analyses. A map of the average annual areal PET is shown below in Figure 5.2.

[3] Bureau of Meteorology (http://www.bom.gov.au/jsp/ncc/climate_averages/evapotranspiration/index.jsp?maptype=3&period=an)

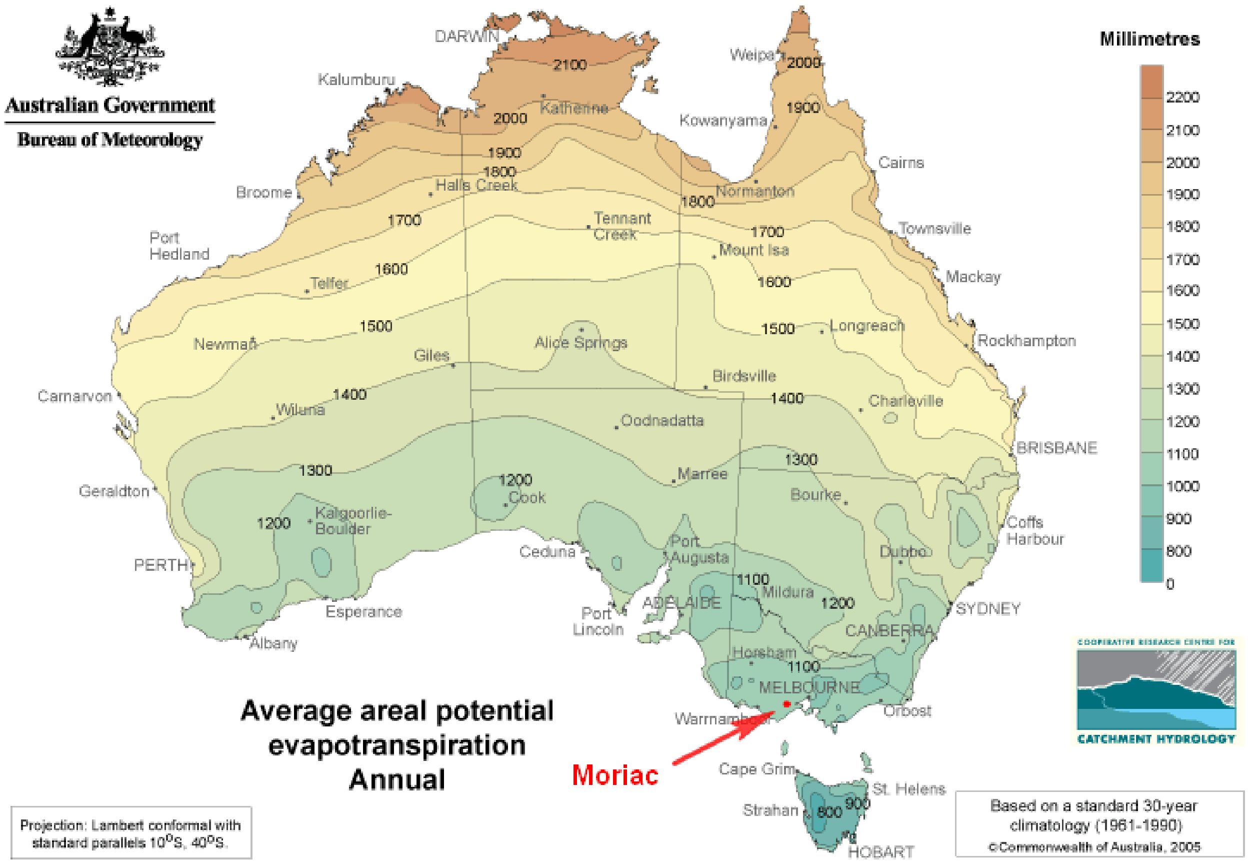


Figure 5.2: Average annual areal potential evapotranspiration

5.3 Geology

The development site resides on two (2) distinct geological formations identified as consisting of 'Newer Volcanic Basalt' and 'Moorarbool Viaduct Sand' deposits.

The ground surface gradients have an average of 1%. Site surface drainage is moderate.

The soil parameters adopted for the MUSIC model in this study are shown below in Table 5-2.

Table 5-2: Soil characteristics for site

Parameter	Urban Residential
Rainfall Threshold (mm)	1
Soil Capacity (mm)	30
Initial Storage (%)	30
Field Capacity	20
Infiltration Capacity coefficient a	200
Infiltration Capacity coefficient b	1
Initial Depth (mm)	10
Daily Recharge Rate (%)	25
Daily Baseflow Rate (%)	5
Daily Deep Seepage Rate (%)	0

5.4 Hydrology

It is an objective for this study that the proposed development should maintain the stormwater runoff volume and quantity discharge characteristics of the existing pre-developed site for the 1% AEP storm event.

The assessment of the stormwater runoff characteristics of the site under existing and developed conditions was undertaken using the XP-RAFTS runoff-routing hydrology model.

5.4.1 Validation of Hydrology Model

The existing hydrology model was validated using the Probabilistic Rational Method (PRM). The PRM input parameters are shown in Table 5-3.

Table 5-3: Rational Method parameters for development site

Parameter	Value	Notes
Area of internal catchments	30.4 ha	799 & 815 Hendy Main Road
C_{10}	0.07	Runoff coefficient
T_c	29 minutes (total site)	Time of concentration
F2	4.29	Geographical factor for a 6 minute, 2 year ARI
F50	14.81	Geographical factor for a 6 minute, 50 year ARI

Design storms were generated for all storm durations using a skew (G) of **0.43** and temporal pattern region **1** as defined from Australian Rainfall and Runoff.

5.4.2 Intensity-Frequency-Discharge (IFD) Data

ARR87 intensity frequency duration (IFD) data was used in the hydrology model. The parameters are shown in Table 5-4.

Table 5-4: IFD data for the Site

ARI (years)	Rainfall intensity (mm/hour) for a given duration (hours)		
	I_1	I_{12}	I_{72}
2	17.90	3.46	0.91
50	34.51	6.01	1.78

The resulting IFD table is shown in Figure 5.3.

Intensity-Frequency-Duration Table

Location: 38.225S 144.175E NEAR.. 799 & 815 Hendy Main Road Issued: 14/1/2016

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	44.7	59.6	81.6	96.8	117	147	172
6Mins	41.7	55.6	76.0	90.0	109	137	160
10Mins	33.9	45.1	61.0	71.9	86.8	108	126
20Mins	24.6	32.5	43.2	50.6	60.4	74.6	86.5
30Mins	19.9	26.1	34.5	40.1	47.8	58.7	67.8
1Hr	13.2	17.3	22.6	26.1	30.8	37.6	43.2
2Hrs	8.55	11.1	14.3	16.4	19.2	23.2	26.5
3Hrs	6.57	8.52	10.8	12.3	14.4	17.4	19.7
6Hrs	4.16	5.36	6.73	7.61	8.83	10.5	11.9
12Hrs	2.61	3.36	4.20	4.73	5.48	6.52	7.35
24Hrs	1.61	2.08	2.62	2.97	3.46	4.14	4.70
48Hrs	.949	1.24	1.60	1.83	2.17	2.63	3.01
72Hrs	.681	.891	1.16	1.34	1.59	1.94	2.23

(Raw data: 17.9, 3.46, 0.91, 34.51, 6.01, 1.78, skew=0.43, F2=4.29, F50=14.81)

Figure 5.3: IFD Table – Hopes Plain Road site (Bureau of Meteorology)

The Rational method produced a 1% AEP (Q_{100}) peak discharge which was used to validate the RAFTS validation process. When validating a hydrology model to the Rational Method, the City of Greater Geelong state a band of ±30% is within acceptable tolerances (often erring on the side of caution and adopting a +30% threshold). TGM applied this bandwidth in the validation of the RAFTS model for the Moriac study area. TGM note that this will produce a conservative discharge and recommend that the validation process is reviewed and refined during detailed design.

The sub-catchment validated peak discharges are depicted in Table 5-5.

Table 5-5: Validated Peak Discharges

Catchment Location	Area (ha)	1% AEP PRM (m³/s)	1% AEP PRM CoGG +30% (m³/s)	RAFTS Model (m³/s)	
				Validation	Existing
1	25.79	0.476	0.619	0.500	0.500
2	4.64	0.129	0.167	0.130	0.130

Table 5-5 shows the existing conditions peak discharges, which were derived by updating the RAFTS model to reflect actual catchment conditions, including grade and land use other than what is assumed by the PRM computations.

The RAFTS layout schematic is shown in Figure 5.4

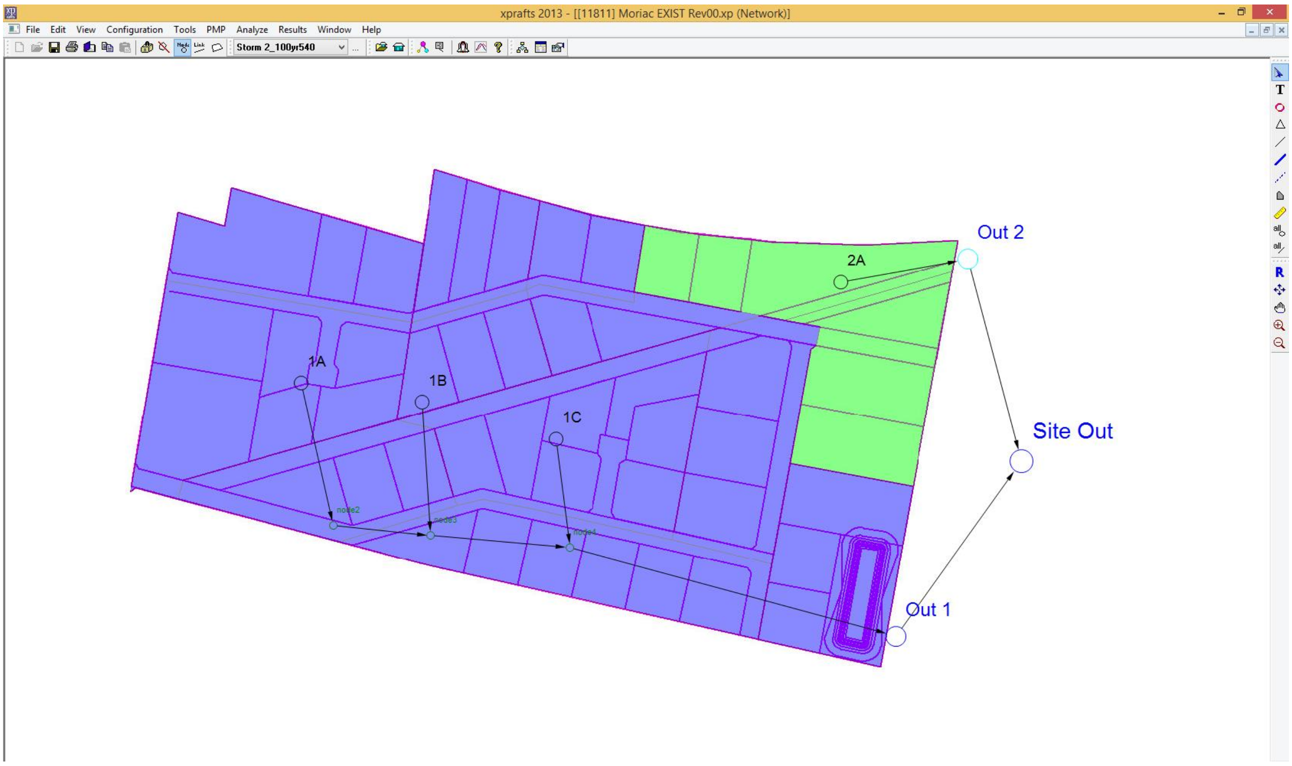


Figure 5.4: RAFTS model schematic layout (Existing Conditions)*

**Note: lot layout plan shown in above schematic does not represent latest development plan.*

The hydrological model was used to identify the requirement for stormwater flow mitigation and enable design detention facilities to manage peak discharges for the 1% AEP storm event. Detention systems are designed to ensure pre-development flows are not exceeded.

5.5 Discharge Target Objectives

The stormwater discharge targets for the developed site are detailed in Table 5-6.

Table 5-6: Peak discharge targets for developed site

Catchment	Discharge (L/s)
	1% AEP
1	500
2	130

The runoff hydrograph for the 1% AEP with a range of storm event durations can be seen in Figure 5.5, Figure 5.6 for each catchment.

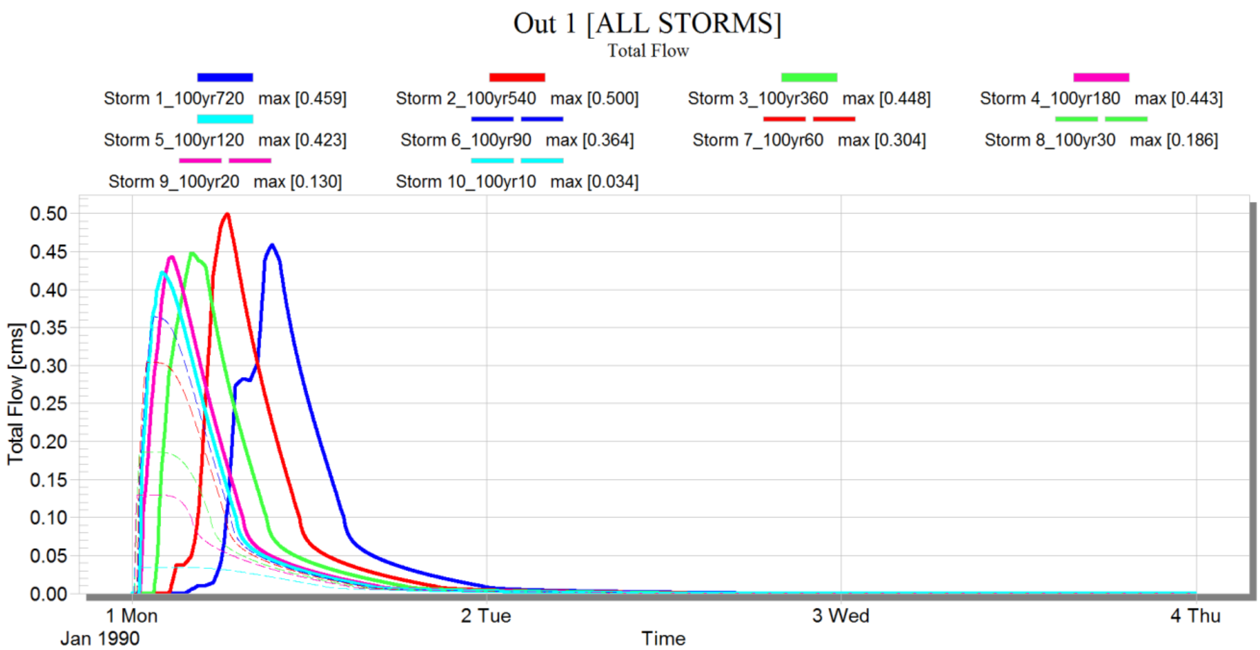


Figure 5.5: Catchment 1 - 1% AEP hydrographs (Existing)

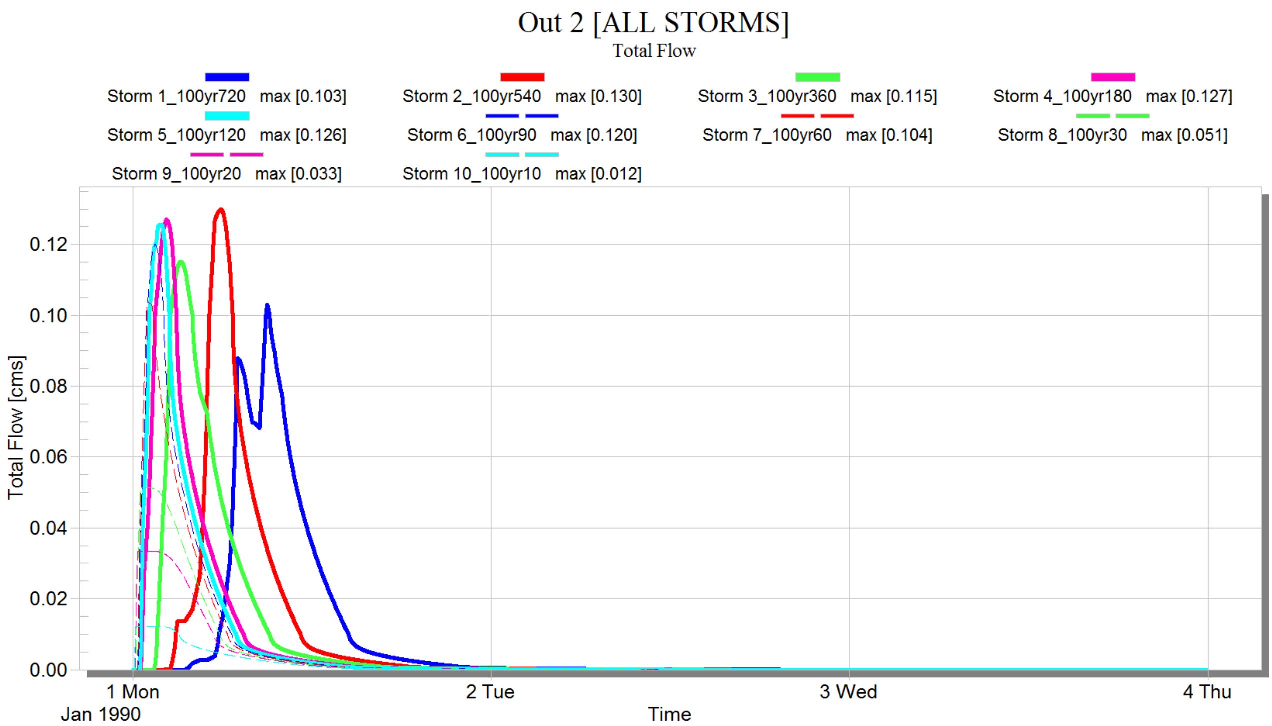


Figure 5.6: Catchment 2 - 1% AEP hydrographs (Existing)

5.6 Stormwater Quality

The ability of the proposed development to meet stormwater quality 'best practice' requirements and design of the mitigation treatment train simulated using MUSIC.

The MUSIC model was used to analyse the efficiency of the proposed stormwater system:

- → Stormwater quality,
- → Total Suspended Solids
- → Total Phosphorus
- → Total Nitrogen
- → Gross Pollutants
- → Average annual runoff volumes, and
- → Frequency of stormwater runoff as indicated by average annual runoff days.

Stormwater quality measures were designed using MUSIC to meet "best practice" targets as described in Section 3.

6. RESULTS

The results of the stormwater hydrology and water quality analysis are shown in this section. The proposed treatment system was designed to meet stormwater quality 'best practice' standards and to ensure that peak discharges from the 1% AEP storm event do not exceed existing conditions and/or create a negative impact to neighbouring properties and receiving ecosystems.

6.1 Water Quality

The ability of the development to meet stormwater quality 'best practice' standards and the performance of the treatment train was continuously simulated using MUSIC.

6.1.1 Scenario 2 – Developed Conditions

Stormwater runoff from the developed site is mitigated through a series of grassed/vegetated swale systems design to convey stormwater runoff, as overland flow, to a detention basin at each legal point of discharge from the site.

The developed treatment train proposes use of grassed swales to convey and treat stormwater runoff, variation of this scenario resides with the secondary treatment facility – bio-retention or wetland.

The developed site applies two (2) swale systems, one for each LPOD. The swale lengths required for each treatment system are shown in Table 6-1

Table 6-1: Swale dimensions – developed conditions

Catchment	Swale
	Length (m)
1	2,815
2	390

A typical swale cross-sectional profile, similar to the one adopted in MUSIC, is shown in relative to the available road reserve in Figure 6.1.

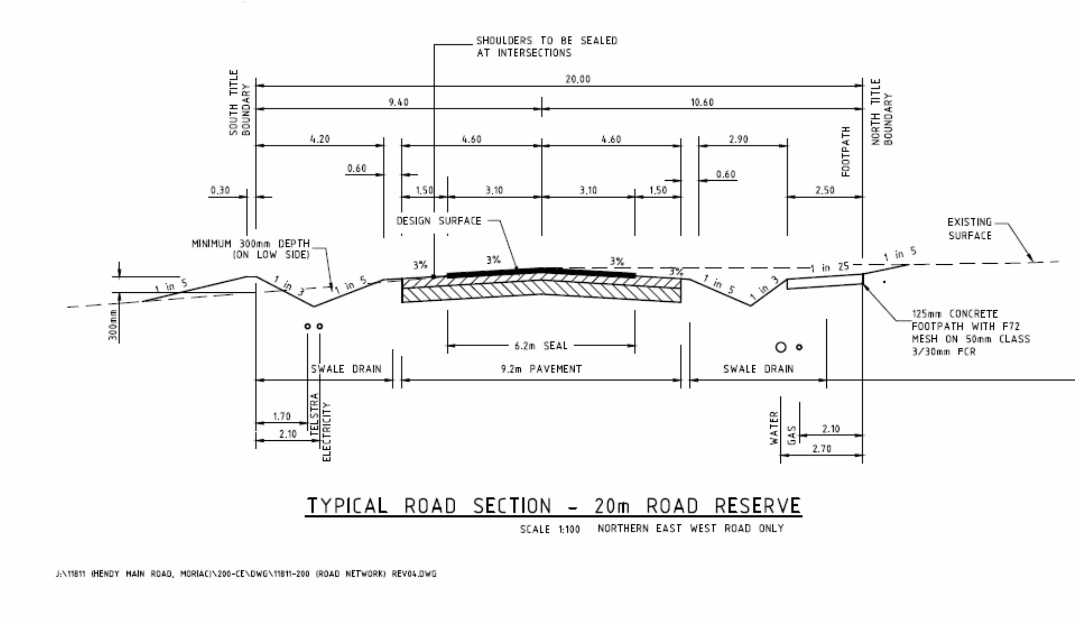
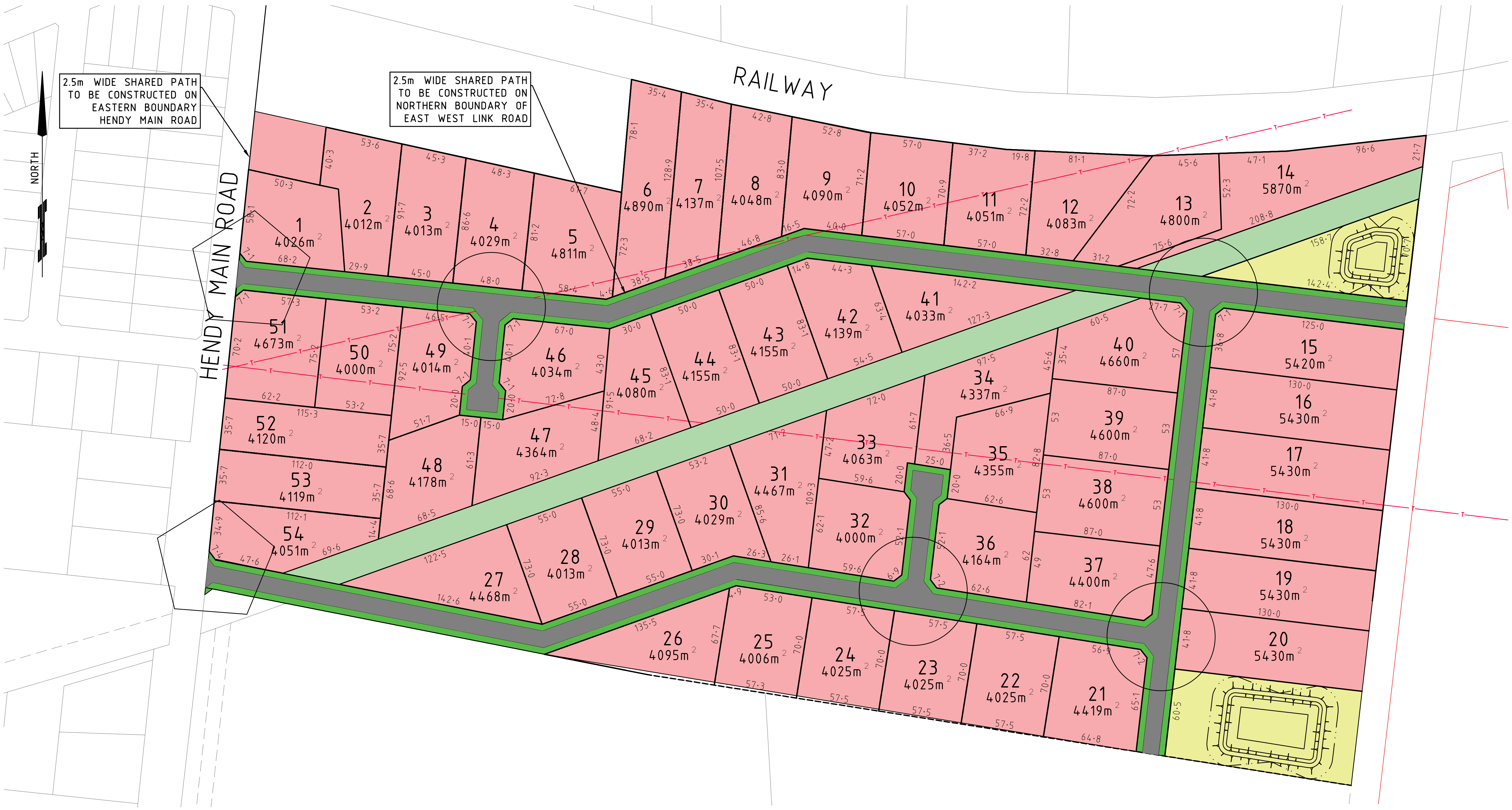


Figure 6.1: Typical road cross-section with swales *(conceptual – may not represent final design)*

The road network is depicted in Figure 6.2.

ROAD NETWORK PLAN
HENDY MAIN ROAD
MORIAC

1:1000 (□ A1)
11/05/2016
Rev. 04



NOTE:
EXTERNAL BASIN BATTERS (GRADE 1 IN 8 APPROXIMATE ONLY)

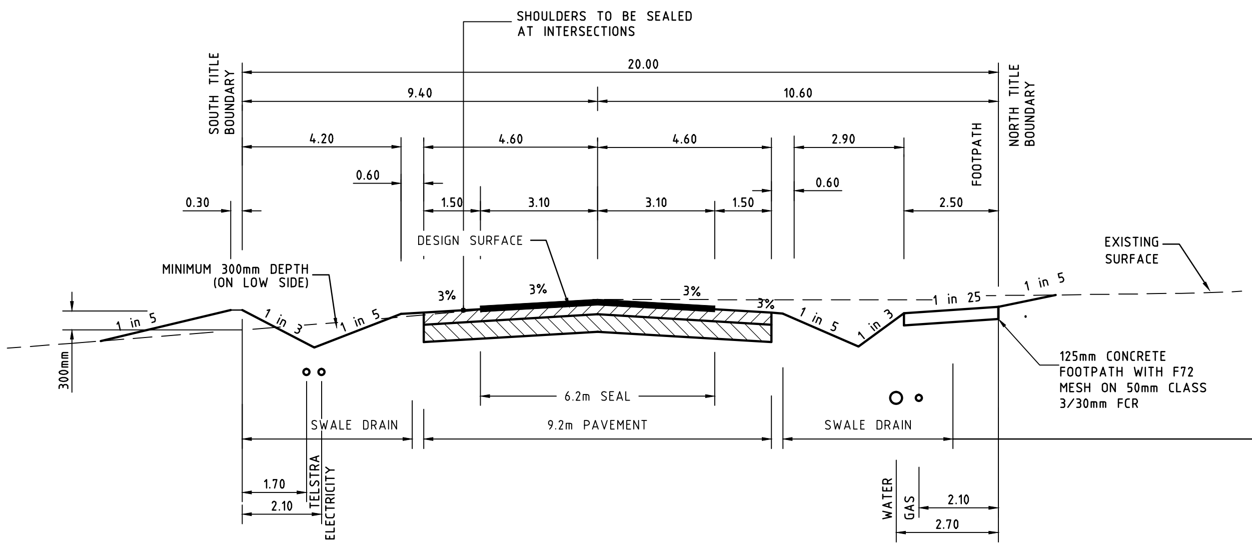
○ INTERSECTIONS TO BE CONSTRUCTED AS STANDARD 'T' INTERSECTION

⬠ MINOR INTERSECTIONS WORKS

NOTE:
PROPOSED INTERSECTIONS TO HENDY MAIN ROAD TO BE AS PER AUSTROADS GUIDE TO ROAD DESIGN-PART 4A. REFER SECTION 8.2.1 RURAL BASIC LEFT TURN TREATMENT.

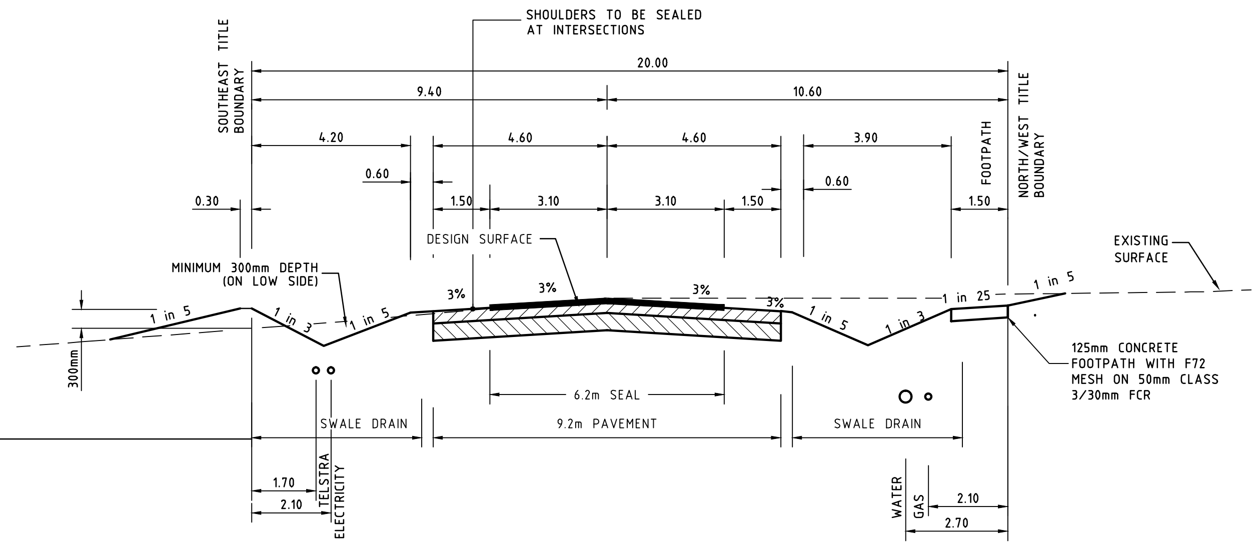
- LOW DENSITY RESIDENTIAL LOTS
- MUNICIPAL DRAINAGE RESERVE
- BARWON WATER RESERVE
- 20m WIDE ROAD

NOTE:
PROPOSED COURTBOWLS AND MINIMUM SEAL WIDTH ACCORDANCE WITH IDM STANDARDS 12.4.2 (RURAL ROAD CHARACTERISTICS).



TYPICAL ROAD SECTION - 20m ROAD RESERVE

SCALE 1:100 NORTHERN EAST WEST ROAD ONLY



TYPICAL ROAD SECTION - 20m ROAD RESERVE

SCALE 1:100

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It is noted that the width of the available road reserve in the proposed development is 20 metres. Swale parameters modelled in MUSIC can be contained within a swale system using only a single side (50%) of the available road reserve, any variation to the vegetated swale parameters during final design should easily be accommodated.

Swale parameters are depicted in Figure 6.3, below.

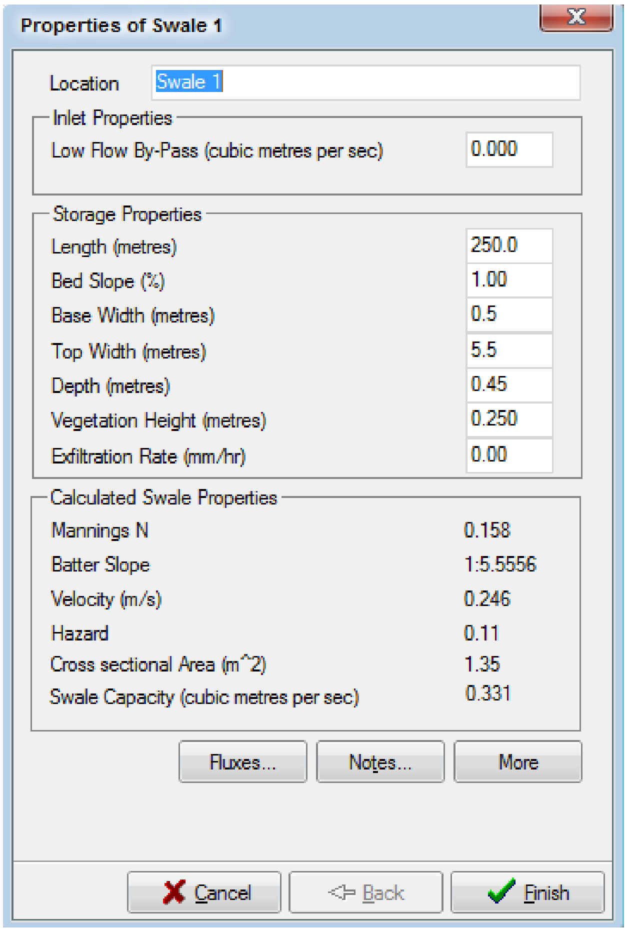

Figure 6.3: Scenario 2 – Swale properties MUSIC

As noted, the swale system only contributes part of the water quality treatment train, with the residual treatment requirement undertaken by bio-retention or wetland facilities.

The treatment train effectiveness for the bio-retention option and the wetland option are shown below.

6.1.2 Scenario 2a – Developed Conditions (Bio-retention)

Stormwater runoff from the developed site is mitigated through a series of grassed/vegetated swale systems design to convey stormwater runoff, as overland flow, to a bio-retention basin located within the base footprint of each detention basin.

The MUSIC model network is shown in Figure 6.4.

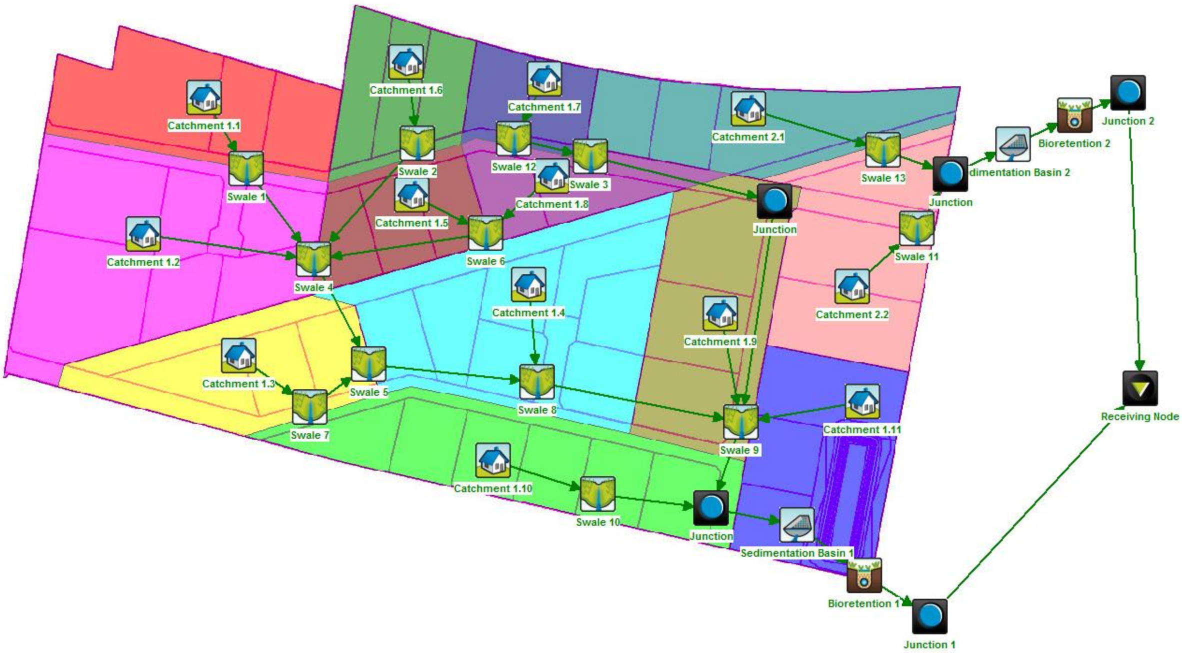

Figure 6.4: Scenario 2a – MUSIC network schematic*

**Note: lot layout shown in above schematic does not represent latest development plan.*

The treatment train for the developed site consists of two separate systems, one for each outlet point, made up of vegetated/grassed swales, a bio-retention basin and an end-of-line sedimentation basin. The bio-retention characteristics are shown in Figure 6.5, for Catchment 1.

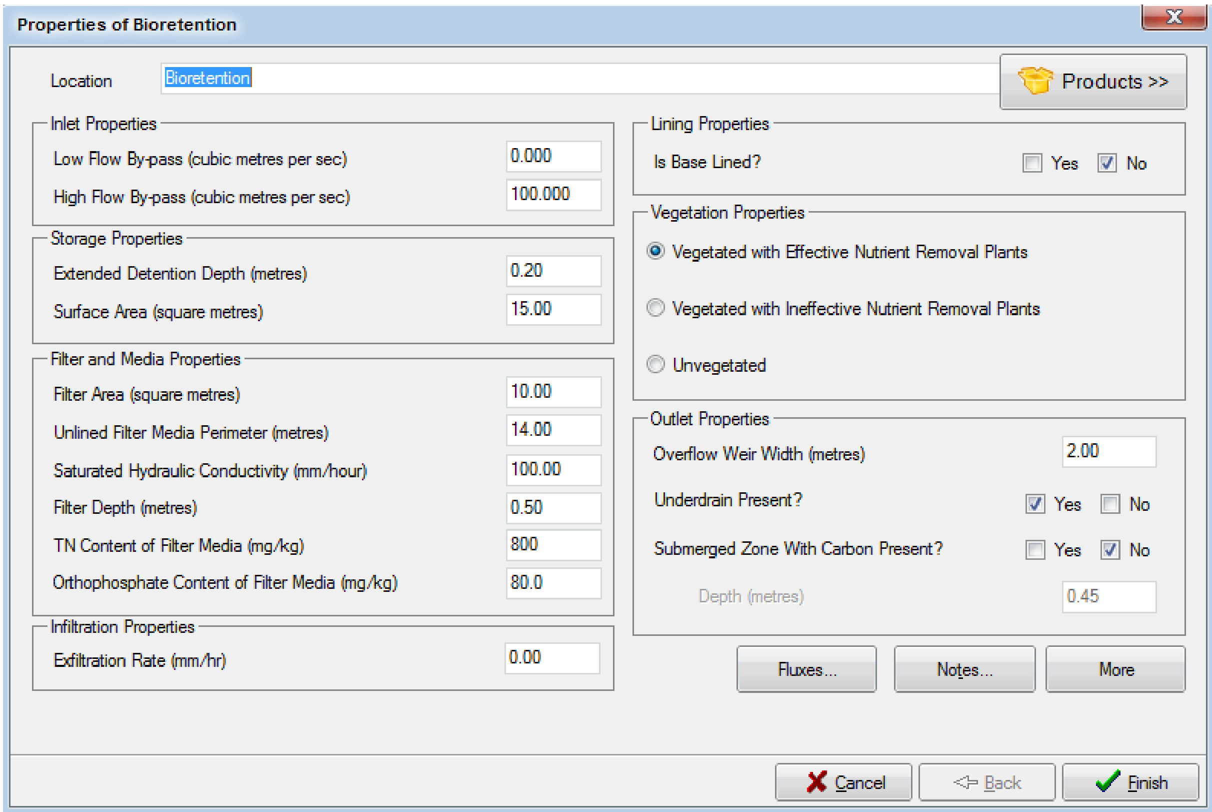

Figure 6.5: Scenario 2 – Bio-retention properties MUSIC

The variable bio-retention values for both catchments are shown in Table 6-2

Table 6-2: Developed conditions stormwater quality treatment efficiency

Catchment	Surface Area (m²)	Filter Area (m²)	Unlined Filter Media Perimeter (m)
1	12	6	14
2	8	4	8

The treatment efficiency of the water quality treatment train with bio-retention is shown in Table 6-3.

Table 6-3: Developed conditions stormwater quality treatment efficiency

Scenario 2a – Developed conditions (Bio-retention)	Catchment 1	Catchment 2
Criteria	**Reduction (%)**	**Reduction (%)**
Flow (ML/yr)	4.5	3.1
Total Suspended Solids (kg/yr)	90.4	91.2
Total Phosphorus (kg/yr)	65.9	64
Total Nitrogen (kg/yr)	48.3	46.5
Gross Pollutants (kg/yr)	100	100

6.1.3 Scenario 2b – Developed Conditions (Wetland)

Use of a wetland for the secondary water quality treatment facility lowers the maintenance requirement for the treatment train in a low density rural residential development.

The MUSIC model network for Scenario 2b is shown in Figure 6.6.

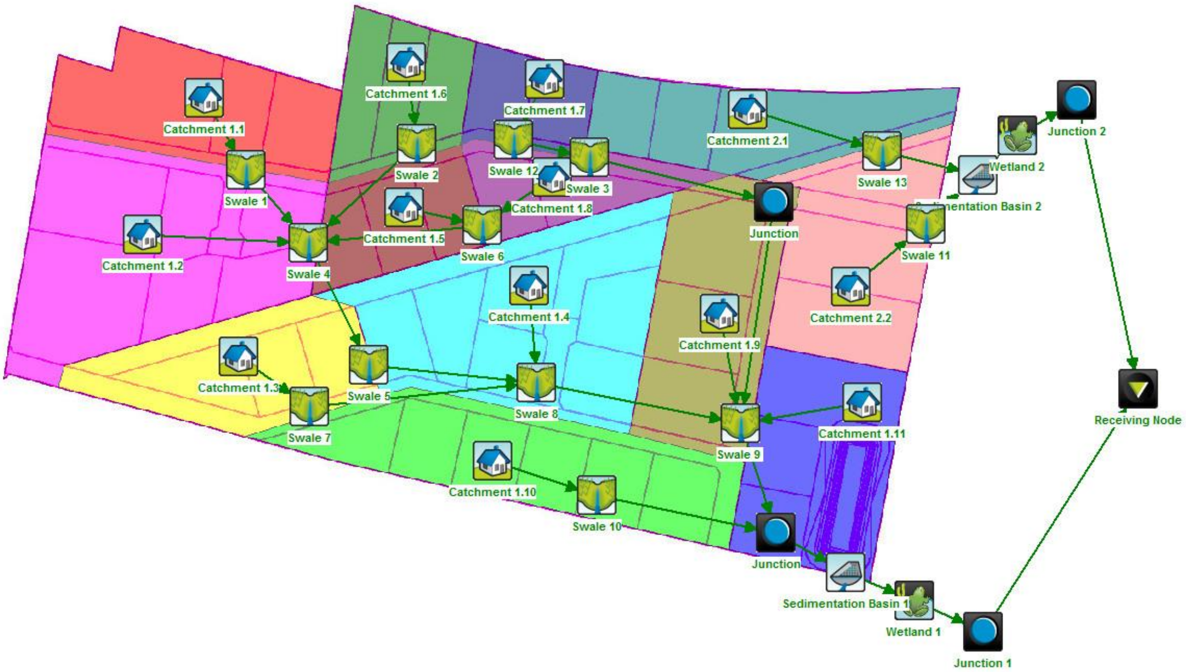

Figure 6.6: Scenario 2b – MUSIC network schematic*

**Note: lot layout plan shown in above schematic does not represent latest development plan.*

The wetland MUSIC input parameters are depicted in Figure 6.3 a are reflective of both catchments.

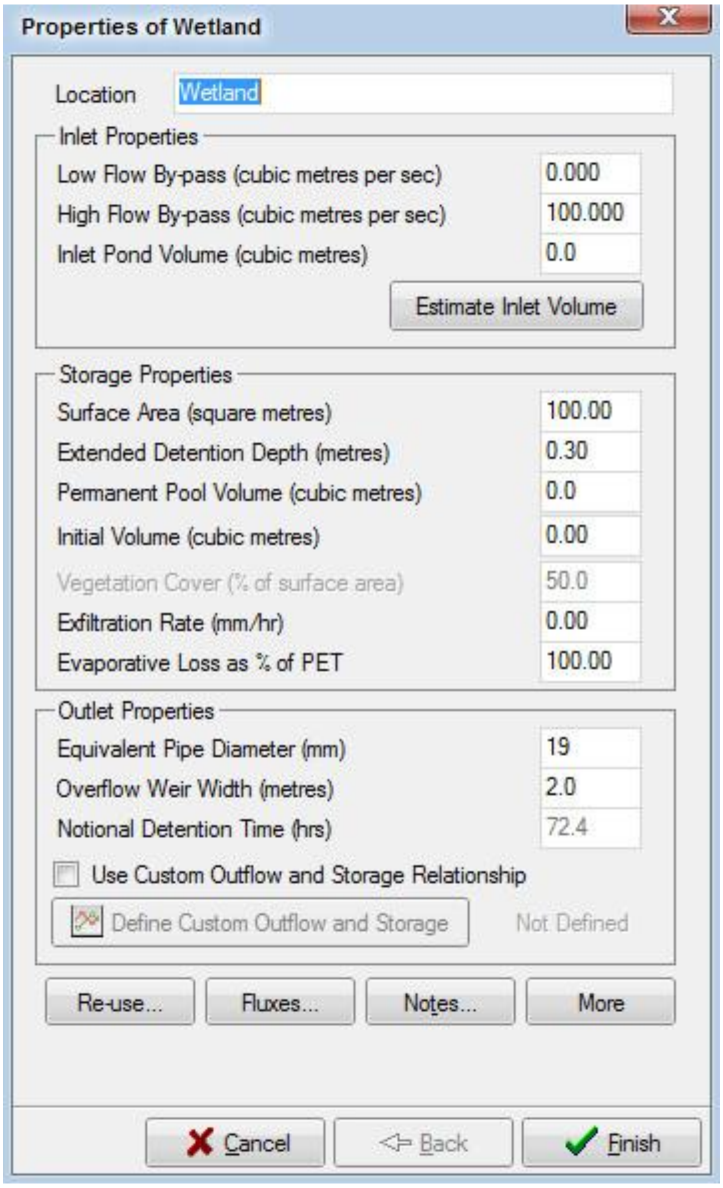

Figure 6.7: Scenario 2 – Wetland properties MUSIC

The treatment efficiency of the vegetated swale system with wetlands is shown in Table 6-4.

Table 6-4: Scenario 2b stormwater quality treatment efficiency

Scenario 2b – Developed conditions (Wetland)	Catchment 1	Catchment 2
Criteria	**Reduction (%)**	**Reduction (%)**
Flow (ML/yr)	4.5	3.5
Total Suspended Solids (kg/yr)	90.5	92.5
Total Phosphorus (kg/yr)	70.1	74.8
Total Nitrogen (kg/yr)	46.7	45.8
Gross Pollutants (kg/yr)	100	100

6.2 Hydrology

Design storm events were used to simulate the 1% AEP event and evaluate the stormwater peak discharges generated by the contributing catchment area. The critical duration for each design event probability and each sub-catchment may vary depending on a number of conditions. Therefore, consideration of a number of storm durations was important to ascertain the critical storm duration impacting the site. The critical event duration was the 9 hour for both existing and developed conditions.

The peak discharges from the site for Scenario 1 (existing) and Scenario 2 (developed) are discussed in the following section.

6.2.1 Scenario 1 – Existing Conditions

Existing conditions create the peak discharges used to set the *Discharge Target Objectives* nominated above. The 1% AEP targets are **500 L/s and 130 L/s** for catchment area 1 and 2, respectively. The peak discharge occurs during the 9 hour event duration for both catchments.

6.2.2 Scenario 2 – Developed Conditions

To ensure the developed site can achieve peak discharge objectives, in terms of 'no-worsening' impact over existing condition during the 1% AEP event, detention basins were designed using a stage-storage relationship along with the basin function in RAFTS.

The developed RAFTS model can be seen in Figure 6.8.

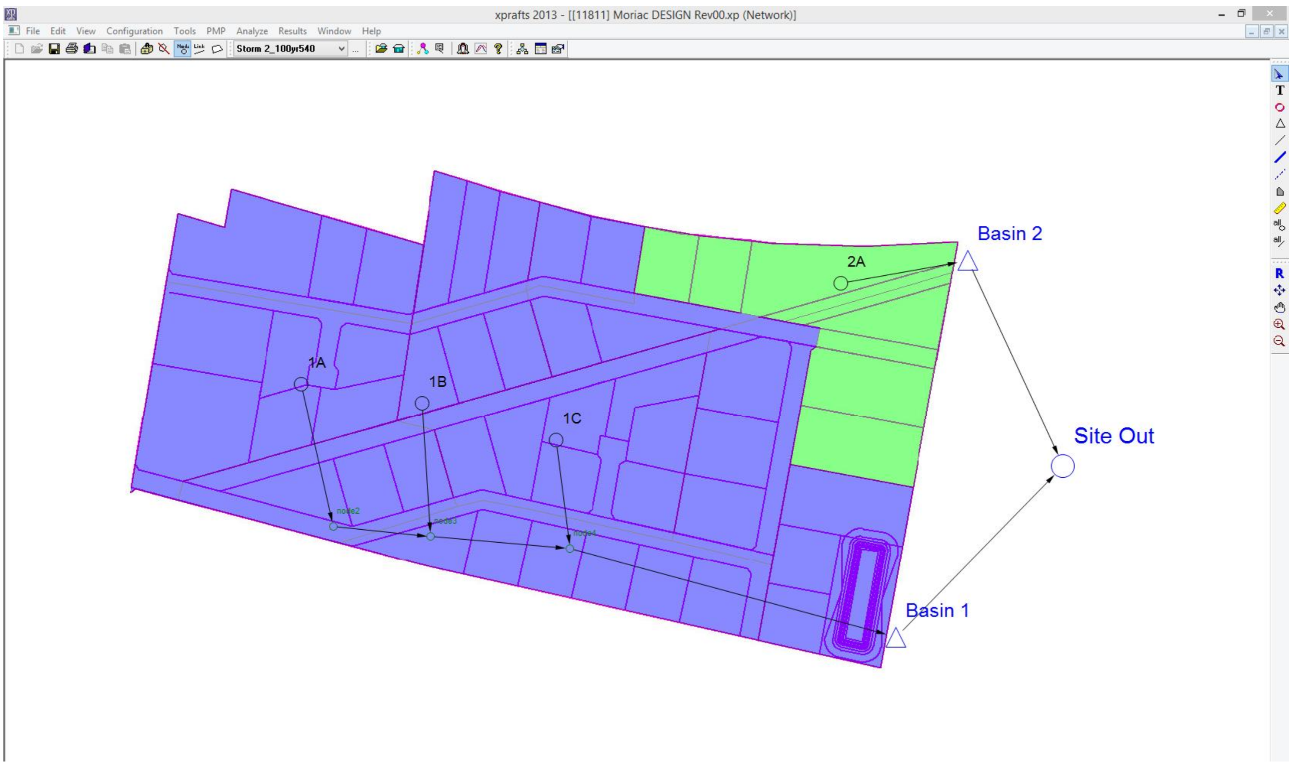

Figure 6.8: Developed conditions RAFTS model schematic*

**Note: lot layout plan shown in above schematic does not represent latest development plan.*

The detention basin volumes required for each legal point of discharge are shown in Table 6-5.

Table 6-5: Detention basin volumes

Catchment	Volume (m³)	Depth (m)
1	1350	0.87
2	300	0.6

The design basin outflow hydrographs are shown in Figure 6.9 and Figure 6.10, below.

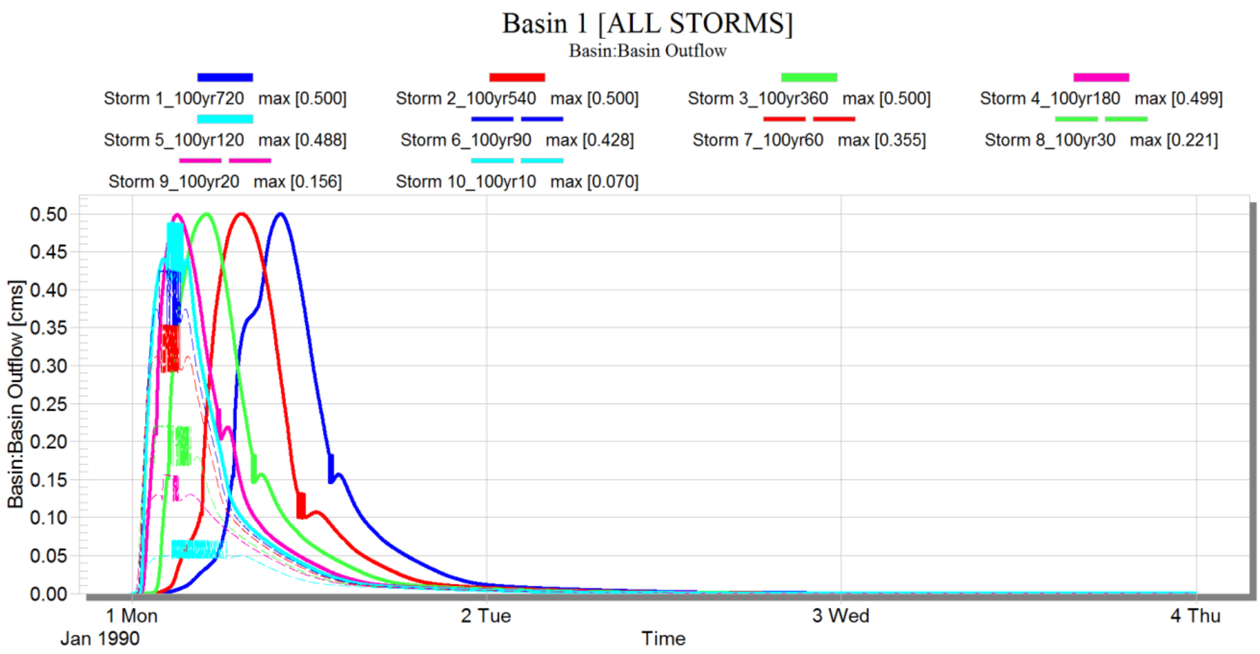

Figure 6.9: Catchment 1 - 1% AEP hydrographs (Developed)

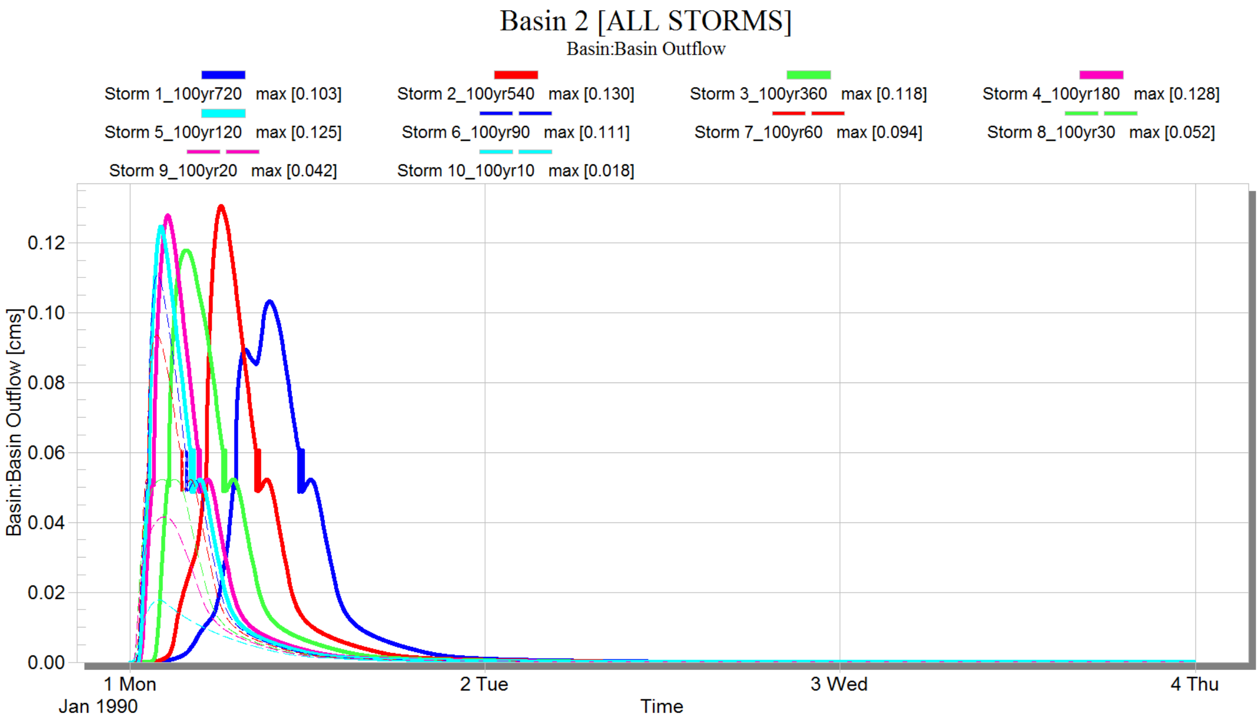

Figure 6.10: Catchment 2 - 1% AEP hydrographs (Developed)

Note: Detention basin footprints, volumes and stage-storage relationship may change during detailed design to accommodate drainage requirements and outfall conditions.

7. CONCLUSION

The report shows that stormwater generated within the proposed Hendy Main Road development site in Moriac can be treated to meet 'best-practice' design objectives. This can be achieved with adoption of a stormwater treatment train consisting of vegetated/grassed swales, sedimentation basins and either a bio-retention facility or wetland facility.

The stormwater runoff conveyed within the water quality treatment train will achieve the required reductions in pollutant loads using the facilities detailed in Table 7-1.

Table 7-1: Proposed treatment train pollutant reduction capability

Treatment Facility	Site Requirement	
Swales	Length	3,205 m
Bio-retention (2a)	Filter area	10 m^2
Wetland (2b)	Surface area	200 m^2

The use of natural systems, such as swales, in low density residential developments enables the treatment infrastructure to be integrated into the urban landscape and provide a visual amenity in tandem with a practical engineering solution.

Stormwater peak discharges generated by 1% AEP storm event from the fully developed site can be managed to achieve a 'no-worsening' outcome when compared to existing conditions. The ability to mitigate peak discharge from the developed site to maintain pre-development flows can be seen in Table 7-2.

Table 7-2: Peak discharges from site - existing vs developed

Catchment	1% AEP Discharge (L/s)	
	Existing	Developed
1	500	**499**
2	130	**130**

Mitigation involves the use of a detention basin situated at each legal point of discharge to control the rate of flow from the developed site.

Treatment facilities have been analysed using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) and the hydrology model XP-RAFTS. Both models have proven to achieve the stipulated flow and pollutant reductions.

Both models demonstrated that the proposed development can be constructed to meet the requirements for best practice reductions in stormwater contaminant loadings and ensure a no-worsening of peak discharges entering the receiving environment.

The area required for the detention basins is detailed in Table 7-3, below.

Table 7-3: Detention basin requirements

Catchment	Detention Basin Area	
	Detention Volume (m^3)	Total Footprint Area (m^2)
1	1,350	~ 1,552
2	300	730

The total footprint refers to the land take required for the proposed detention basins including batters and buffers. An indication of the land take required for the detention basins can be seen in Figure 7.1.

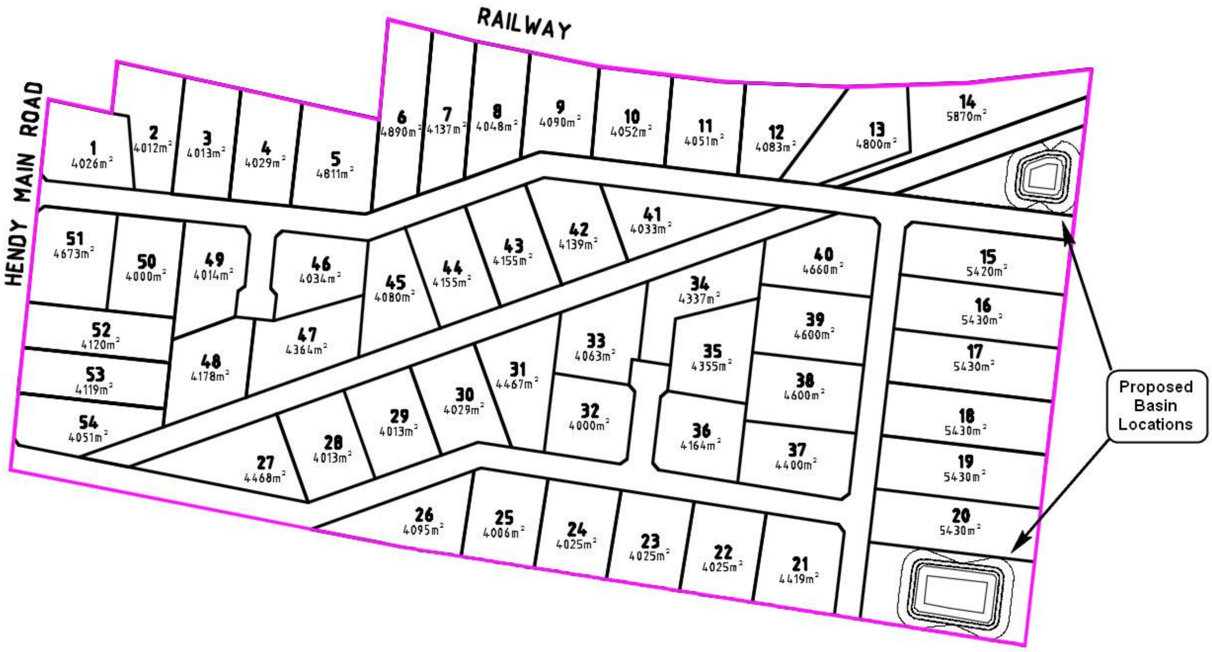

Figure 7.1: Indicative detention basin footprint

7.1 Design Drainage Schematic (Conceptual)

The developed treatment train will allow conveyance and treatment of stormwater generated with the development site. The conceptual treatment train schematic is shown in Figure 7.2.

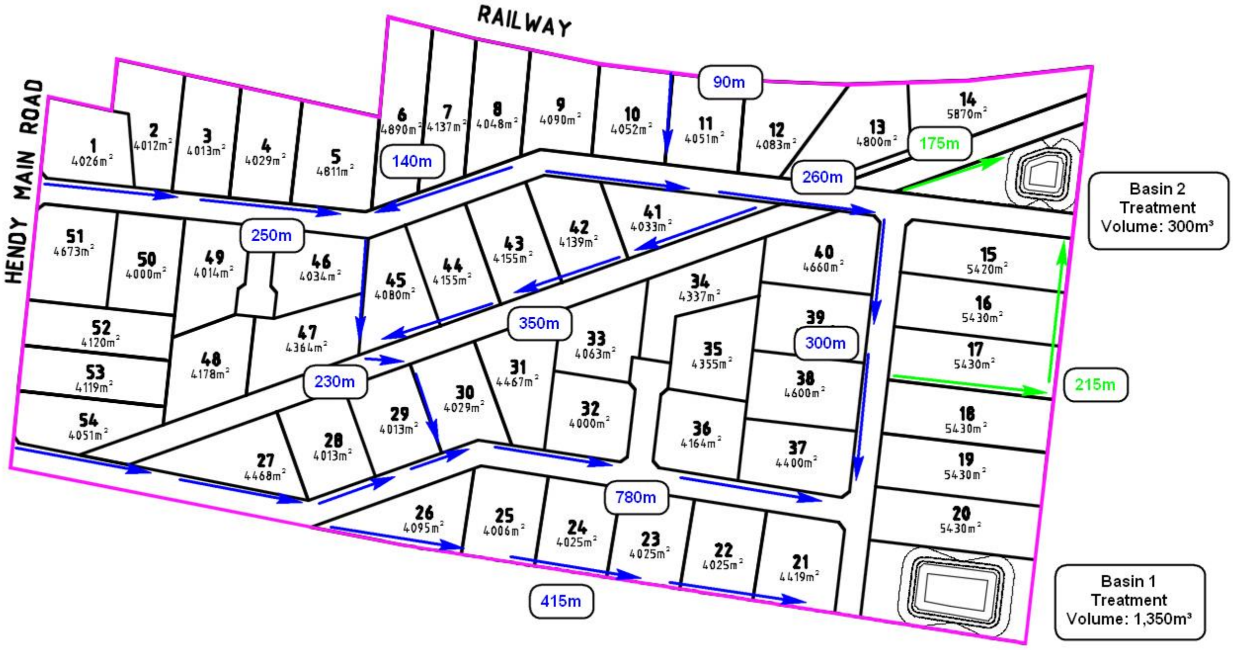

Figure 7.2: Conceptual Treatment Train Schematic

The concept plan adopts a swale system along one side of the proposed road reserve. The available road network accommodates the swale lengths required to enable best practice reduction in stormwater pollutant loadings to be achieved.