135-225 Austin Street, Winchelsea

Site Stormwater Management Plan

Ref No: 17797-03

Prepared For: Surf Coast Shire

Due: September 2019

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Front Cover Image – Site Photo (Courtesy of CardnoTGM)

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Executive Summary

Study objectives

CardnoTGM has been engaged by Anthony Robert Grant Pty Ltd to prepare a Stormwater Management Plan (SMP) for the proposed development of 126-lot subdivision that is situated between Austin, Witcombe and Stephenson Street, Winchelsea.

The proposed subdivision will create an increase in impervious surfaces resulting in an increase in stormwater runoff volumes and contaminant loading. The SMP demonstrates that the site can be developed using best practice stormwater management principles and techniques. The objectives will inform stormwater designs and ensure that stormwater quality and quantity targets are achieved and maintained.

Study Methodology

Site pre-developed discharges were calculated using methods and data from the latest revision of Australian Rainfall and Runoff (ARR2016) and measures undertaken to ensure best practice guidelines were achieved namely;

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. Stormwater conveyance
   - Conveyance of flows up to and including the 1% AEP flows to the LPOD.
3. Stormwater Quantity
   - Ensuring no increase in stormwater rates discharging from the LPOD for events up to and including the 1% AEP flows.

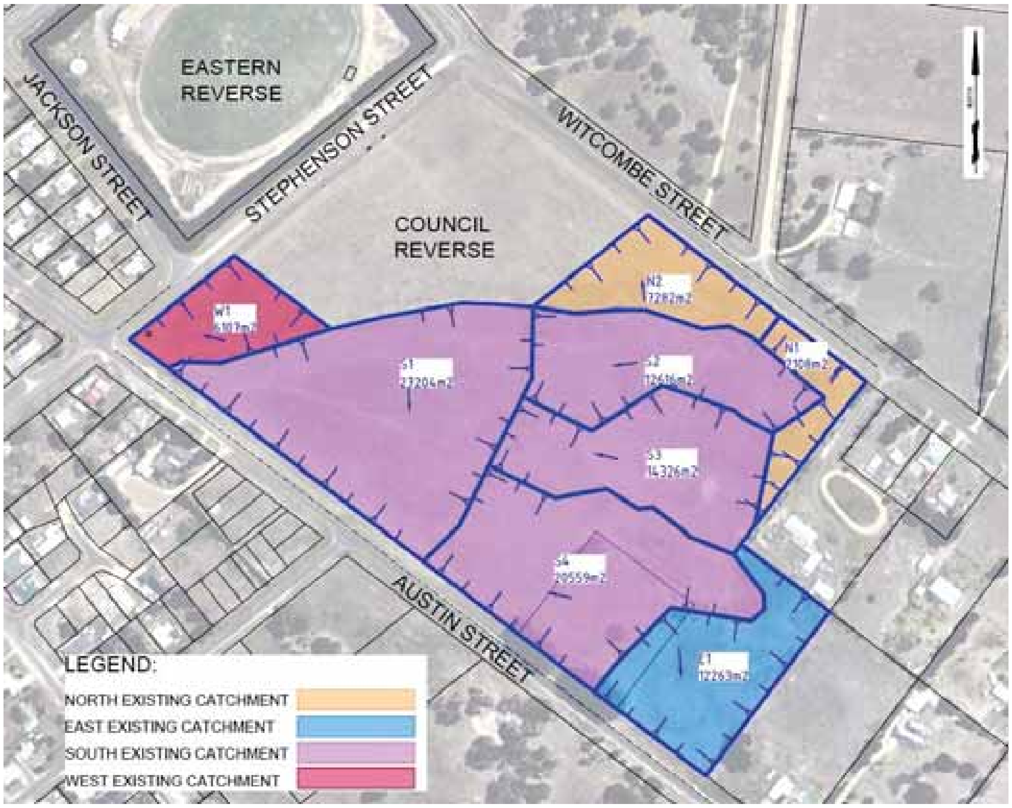

Figure 1-1 Existing Catchments – Austin Street.

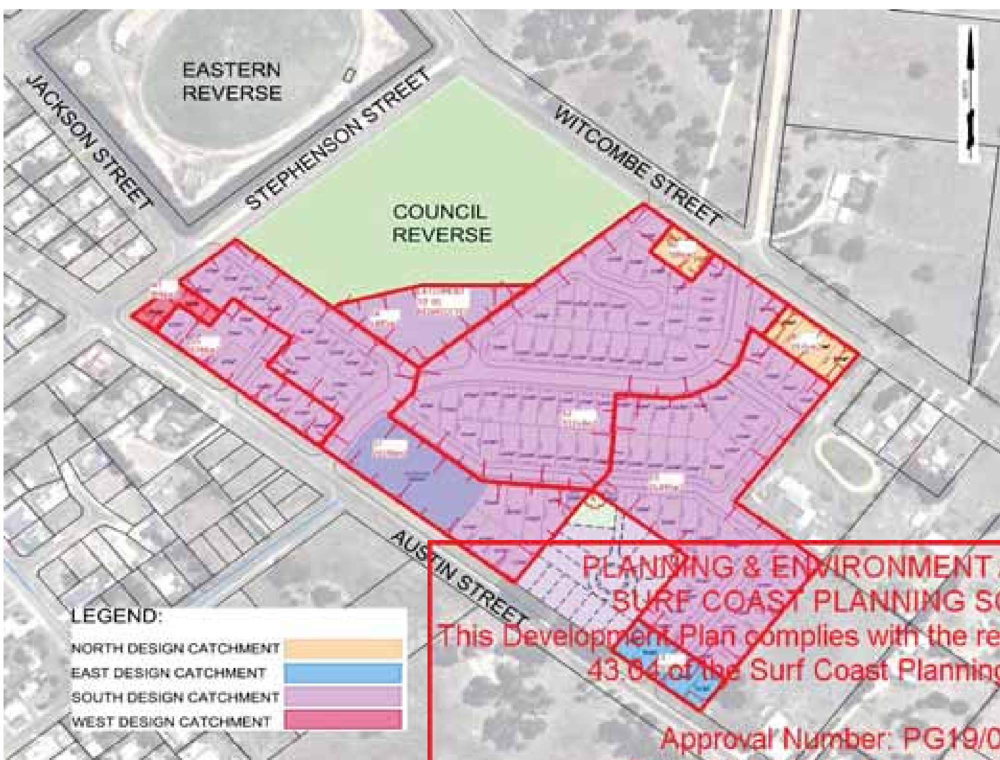

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Figure 1-2 Developed Catchments – Austin Street

Key Results

Stormwater peak discharge generated by the 1% AEP storm event from the fully developed site will be managed utilising a combination of underground stormwater drainage, incorporation of stormwater detention/ treatment systems within the south catchment.

Table 1-1 Overall Site Pre-Development Discharge Summary

	1%	20%
North Pre-Developed Peak Flow (m^3/s)	0.11	0.04
North Pre-Developed Storm Duration (min)	120	15
East Pre-Developed Peak Flow (m3/s)	0.062	0.022
East Pre-Developed Storm Duration (min)	120	20
South Pre-Developed Peak Flow (m3/s)	0.235	0.059
South Pre-Developed Storm Duration (min)	120	10
West Pre-Developed Peak Flow (m3/s)	0.048	0.012
West Pre-Developed Storm Duration (min)	120	15

Table 1-2 Overall Site Developed Discharge Summary

	1%	20%
North Developed Peak Flow (m^3/s)	0.062	0.033
North Developed Storm Duration (min)	15	15
East Developed Peak Flow (m3/s)	0.042	0.017
East Developed Storm Duration (min)	10	10
South Developed Peak Flow (m3/s)	0.235	0.057
South Developed Storm Duration (min)	540	540
West Developed Peak Flow (m3/s)	0.031	0.013
West Developed Storm Duration (min)	15	15

The study also shows that stormwater generated within the proposed subdivision site can be mitigated to meet 'best practice' water quality objectives. The subdivision is to comprise of a Gross Pollutant Trap, Sedimentation Basin and Bioretention Basin.

Table 1-3 Stormwater Quality Treatment Efficiencies – Overall Site

Criteria	Reduction (%)	
	Results	Target
Total Suspended Solids (kg/yr)	80.1	80
Total Phosphorus (kg/yr)	51	45
Total Nitrogen (kg/yr)	45.2	45
Gross Pollutants (kg/yr)	87.9	70

This report provides a prima facie assessment of the proposed development of Austin Street to show that the stormwater runoff can be effectively managed to ensure all stormwater targets and objectives can be meet.

Table of Contents

Tables

Figures

1 Background

CardnoTGM has been engaged by Anthony Robert Grant Pty Ltd to undertake an assessment of a proposed 126-lot subdivision and preparation of a Stormwater Management plan and report to accompany a planning permit submission to Surf Coast Shire (SCS). The site is approximately 9.9 Hectares in size and is located approximately 33km South West of Geelong CBD within a parcel of land recently rezoned General Residential Zone 1.

Council has previously purchased the northern corner of the site for the construction of a second sporting field adjacent to Eastern Reserve with the proposal to close off Stephenson Street in between.

Figure 1-1 Planning Zone

2 Study Objectives

The objective of this Stormwater Management Plan (SMP) is to demonstrate that the stormwater treatment infrastructure proposed as part of the development can be designed in accordance with best practice engineering guidelines and the Infrastructure Design Manual (IDM) (where appropriate). This will provide guidance to the developer and SCS as to the scale of stormwater infrastructure required in order to achieve and maintain stormwater quality and quantity targets.

Stormwater runoff from the site will be captured and conveyed, via an underground drainage network for the minor storm events (20% Annual Exceedance Probability (AEP)) and predominantly via the road network for the major storm event (up to and including the 1% AEP) to the site's Legal Point of Discharge (LPOD). Within the southern portion of the site a stormwater treatment facility is proposed to treat runoff before leaving the site at one of the proposed LPOD. Small catchments located in the East, West and North of the site are proposed to discharge the site without undergoing stormwater treatment, these catchments are proposed to outlet to their respective existing downstream drainage infrastructure at, or near to, pre-development flows.

The objectives of the Site Stormwater Management Plan (SSMP) are detailed below.

2.1 Site Stormwater Objectives

The site stormwater objectives 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. Stormwater conveyance
   - Conveyance of flows up to and including the 1% AEP flows to the LPOD.
3. Stormwater Quantity
   - Ensuring no increase in stormwater rates discharging from the LPOD for events up to and including the 1% AEP flows.

Throughout this report the '20% AEP' and '1% AEP' will be referred to as the minor and major storm events respectively.

The following stormwater management plan will provide details on the proposed stormwater treatment facility and associated structures physical requirements for the mitigation of runoff from the development to ensure stormwater discharge targets are achieved for the entire site.

3 Study Area

3.1 Existing Site Conditions

The site is an approximately 9.9 Hectare land parcel that is bounded by three streets, Austin Street to the south, Witcombe Street to the west and Stephenson Street to the west. Along the eastern side of the property are existing Farm zone properties. Council have recently acquired a parcel of land that was originally in the northern corner of the site earmarked for the future extension of Eastern Reserve.

The surrounding existing drainage infrastructure is mostly open drains with some kerb and channel in Austin Street. Along Austin Street an existing open drain fronts the site, there is also a small portion of kerb and channel on the southern side of Austin Street and extending into Batson Street discharging to the east. There is an existing underground drain located on the eastern side of Baston Street that drains south towards Harding Street.

Witcombe Street has an open drain running on both sides of the road which fall both to the west, and, to the north down Gladman Street. Stephenson Street has an open drain along the site frontage, on the corner of Stephenson street and Austin Street a culvert runs underneath Stephenson Street and connects to the existing kerb and channel in Austin Street that continues to the north west. At the eastern end of Austin Street and existing open drain captures runoff and discharges to the east.

Figure 3-1 Austin Street Locality Plan

Figure 3-2 Proposed Council Reserve Plans[1]

A ridgeline runs from west to east creating a high point within the site resulting in two distinct catchments that fall to the north and south. Within the south there are two minor catchments that fall out to the west and east.

[1] Winchelsea Eastern Recreation Reserve Master Plan prepared by GHD

Figure 3-3 Existing Catchments

An existing dumping site exists along the eastern section of Austin Street originally used for night fill from the district, this portion of site has existing tall trees mostly along the portion fronting Austin Street. This area is identified as a medium density site is to be constructed under a different planning permit, it has been included within this report.

Figure 3-4 Existing Night Fill Site

3.2 Developed Site Conditions

Development of the site is to include construction of 126 lots, a medium density residential area (to be built with a sperate planning permit at a later date), drainage reserve and associated road networks. Residential lots range in size from approximately 340m^2 to 900m2. The overall Development Plan (ODP) is presented in Figure 3-5.

Figure 3-5 Overall Development Plan

The majority of the site (approx. 8.87ha) will be directed to a drainage reserve containing a stormwater treatment facility located along Austin Street. Flow conveyance will consist of an underground drainage system designed for the 20% AEP and overland contained flows within the road reserve for flows up to and including the 1% AEP.

Three catchments are proposed to discharge offsite without treatment;

> a 0.32ha northern catchment consisting of the lots fronting Witcombe Street. The size of the catchment is smaller than the existing catchment of 1.13ha.

  The catchment is to discharge into the existing open drain located on the eastern side of Gladman Street via a new underground drainage system constructed during reconstruction of Witcombe Street.

> An eastern catchment of 0.26ha will discharge via an underground drainage system connecting to an existing open drain located in Austin Street to the east of the site.

> The western catchment is 0.11ha, this catchment is to drain to the existing culvert located on the corner or Stephenson Street and Austin Street.

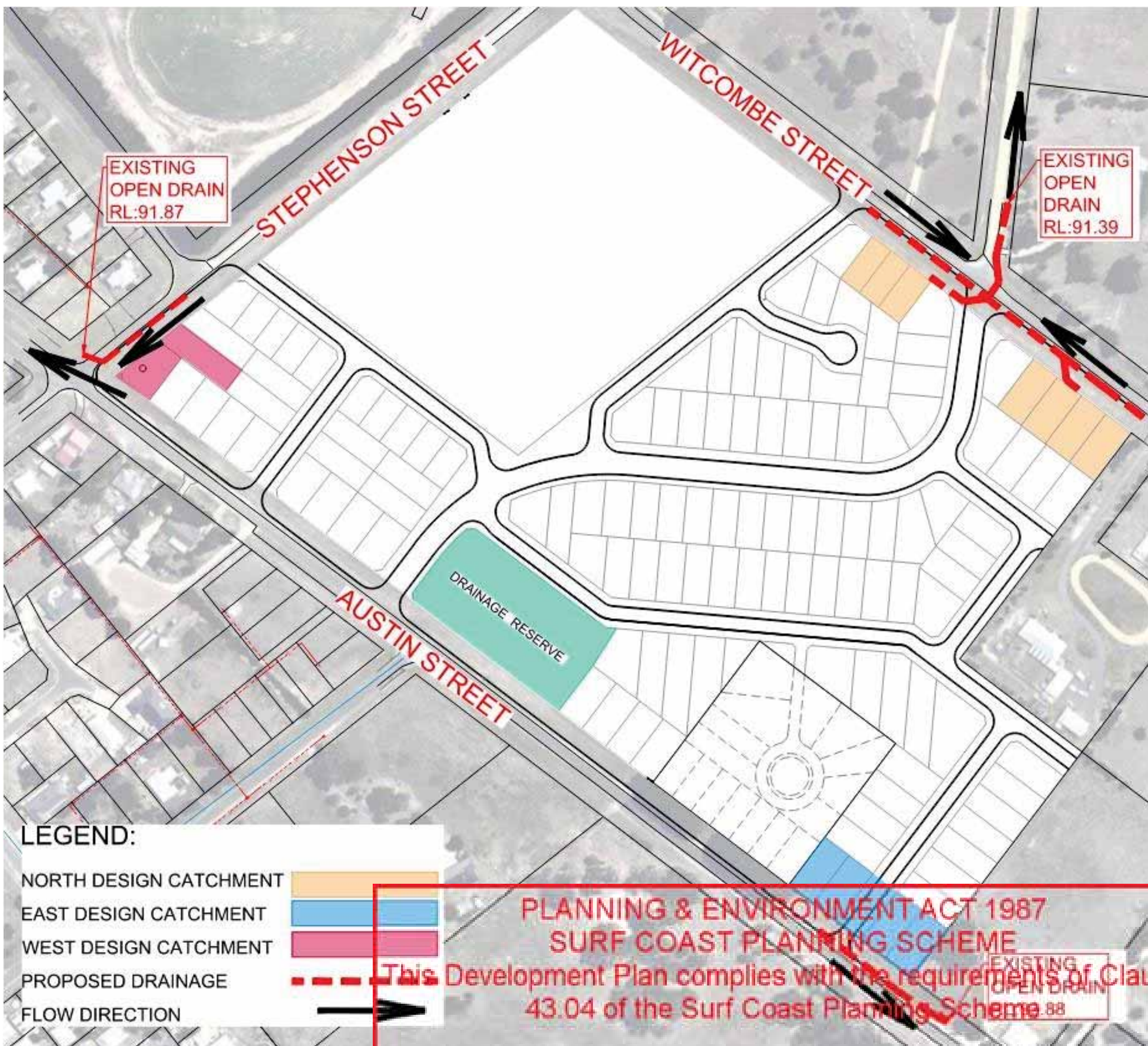

Figure 3-6 Undetained Catchment Outfalls

In order to limit flows from the site to the pre-developed flowrates a stormwater treatment facility consisting of a gross pollutant trap (GPT), a sedimentation basin, detention and bioretention basin (or approved proprietary product) is to be constructed in the south of the site fronting Austin Street prior to LPOD in Baston Street.

Figure 3-7 Concept Drainage Reserve Outfall

Commented [TD1]: Note bioretention as 'bioretention or approved proprietary product

From the LPOD the treated stormwater runoff will discharge to the Barwon River via the existing underground network outlined in Figure 3-8 and associated appendices. Provided drainage longitudinal sections states that 235L/s was the allowable discharge rate allowed into the existing drainage system.

From the analysis completed in the following sections of the report it was shown that for the southern catchment a pre-developed rate of 235L/s was established for the site, which can be detained and discharged to 235L/s through the analysis of the conceptual drainage reserve configuration shown above and within following sections of the report.

Figure 3-8 Existing Drainage Alignment to Barwon River

4 Stormwater Mitigation Systems

To ensure the reductions in flow rate and contaminant loading are achieved a stormwater detention and treatment facility is to be constructed prior to discharge point in Austin Street, the facility is to be made up of a GTP, Sedimentation Basin and Bioretention Basin with stormwater detention.

Details on the different aspects of the stormwater mitigation system are detailed below;

4.1 Gross Pollutant Trap

At the start of the stormwater treatment facility a gross pollutant trap (GTP) is to be constructed. The GTP is to trap gross pollutant's i.e. rubbish from entering the system and also provides another form of sediment removal. As bioretention basins are sensitive to sediment build up, by providing strong sediment removal at the front of the system further protects the basin.

Within this treatment facility, the GTP has not been modelled for nutrient removal, it is solely been modelled for gross pollutants and sediments.

4.2 Sedimentation Basin

Stormwater runoff generated within the development will be captured and conveyed in a traditional underground drainage system to end of line sedimentation basins. Stormwater runoff will undergo initial treatment via sedimentation to remove coarse sediments, reducing scour velocities due to increased surface area for flow and limiting nutrient build-up prior to entering the bioretention system proper. Treated flows then enter the centrally located wetland system through an outlet orifice up to the extended detention depth.

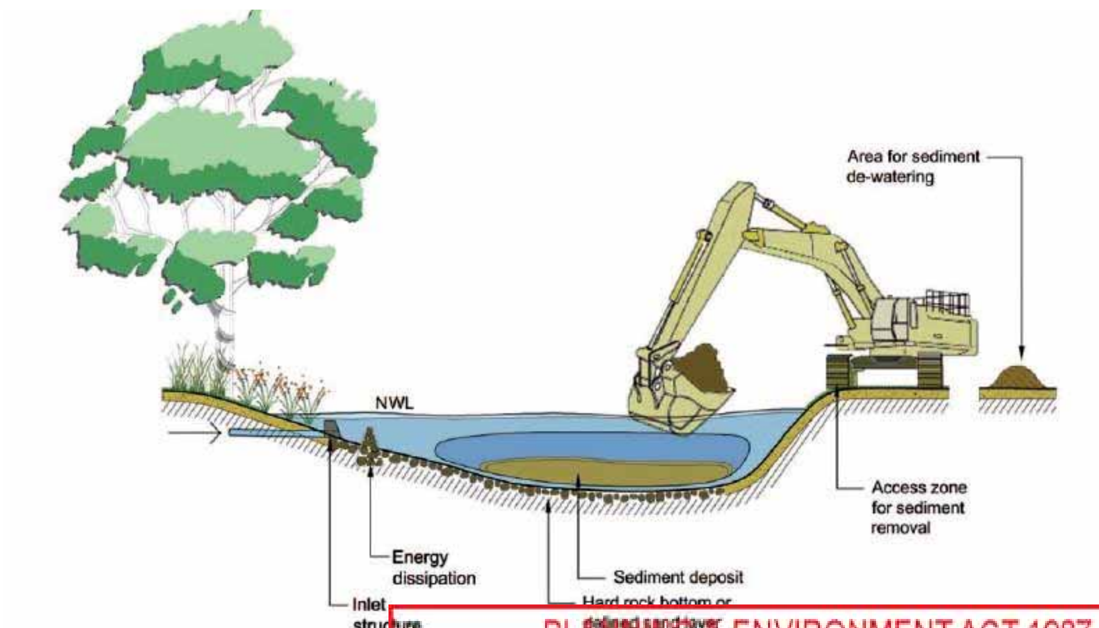

Figure 4-1 Typical Sedimentation Basin[2]

[2] Melbourne Water (2005), WSUD Engineering Procedures Stormwater, CSIRO Publishing, pp. 26

4.3 Bioretention Basin

After flows have received pre-treatment from the sedimentation basin the flows enter a bioretention basin. Bioretention basins generally have a smaller footprint than constructed wetlands whilst providing high treatment efficiencies. Runoff is directed into the basin via an inlet pipe once runoff is discharged into the basin area the runoff is treated by a filter media. The runoff filters through the filter media which enables the finer sediments to settle, the filter media is highly vegetated with plant life which allows the biological uptake of nutrients from the stormwater runoff.

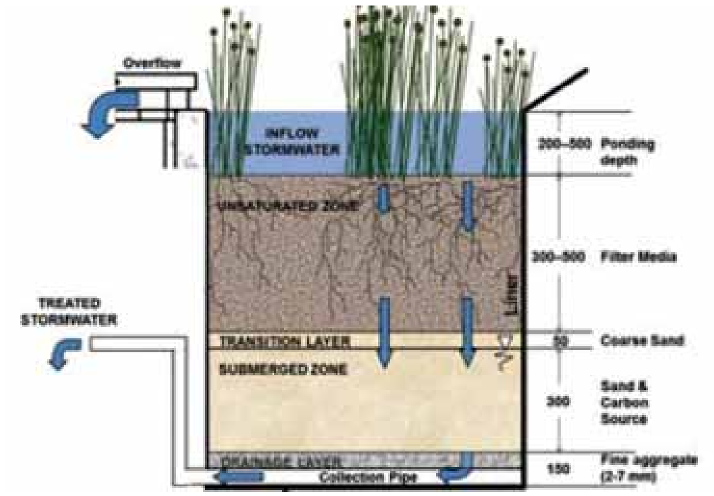

Figure 4-2 Typical Filter Media Section with Submerged Zone.

Bioretention basin are sensitive system that require regular maintenance to ensure that the basin works as intended. During construction of the subdivision the bioretention basin needs to be staged to ensure that it is protected and not damaged, it is typical for bioretention basin to be constructed once 80% of the overall catchment has been constructed and online. Further ongoing maintenance needs to be provided to ensure that plant life within the basins filter media is kept alive, also maintenance to remove any fine sediment build up to ensure filter media remains free draining through its life span.

4.3.2 Proprietary Systems

A number of proprietary product also exist that can take place of a cast in situ bioretention system. Some proprietary products that have the capacity to treat urban stormwater to the current best practice standards include the 'Hydrosystem' by SPEL P/L and the 'Filterra' by Ocean Protect P/L. When designed and installed correctly these systems often require less land take and less maintenances. In the case of the 'Filterra' system maintenance in some situations can be at up to 5-year intervals.

4.4 Detention

Existing conditions are changed when developing sites, due to reducing the amount of pervious surface that original existed. Relevant guidelines and best practices outline that when developing a site there should be no worsen or increase in discharge to any downstream catchments, as an increase in impervious surfaces increases the amount of runoff produce onsite. To ensure no increasing downstream discharge a basin is proposed to be constructed and retain flows to ensure site discharge generated is mitigated to pre-developed conditions. Within the proposed development the detention volume, was calculated and a basin area footprint established. The volume of detention required is measured from the top of the extended detention depth (ED) to the 1% AEP level.

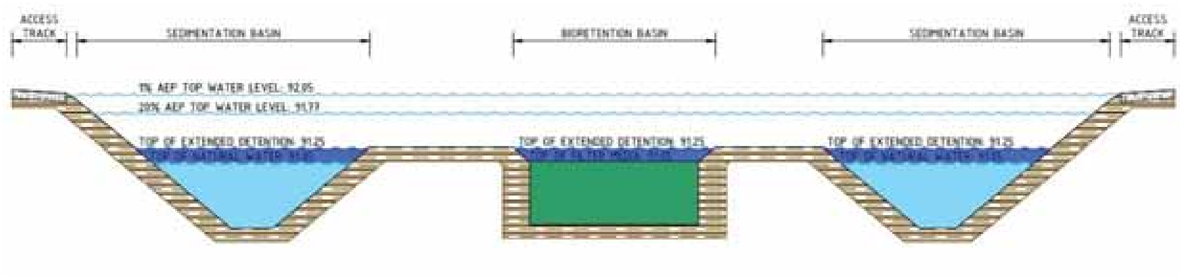

Figure 4-3 Basin Typical Section

5 Hydraulics and Hydrology

The hydrological analysis was simulated with XP-STORM by applying the rainfall and runoff, Laurenson routing and one-dimensional (1D) hydraulic channel techniques based on Australian Rainfall & Runoff 2016 (ARR2016) Methodology. XP-STORM provides features to allow interface with the ARR Data Hub and Bureau of Meteorology (BOM) to obtain IFD and rainfall data to generate temporal patterns for a range of event probabilities.

A distributed hydrological model of the study catchment was used to compute the stormwater hydrographs to determine the discharge infrastructure sizing and distributed detention basin sizing for input into 12D to further determine land area requirements.

5.1 Sub-areas Delineation.

5.1.1 Existing Catchments Delineation

Delineation of existing catchments was based on existing contours provided by a feature survey of the site completed in November 2018. The feature survey allowed the catchments to be determined, and was further enhanced by a site inspection which confirmed what was previously thought.

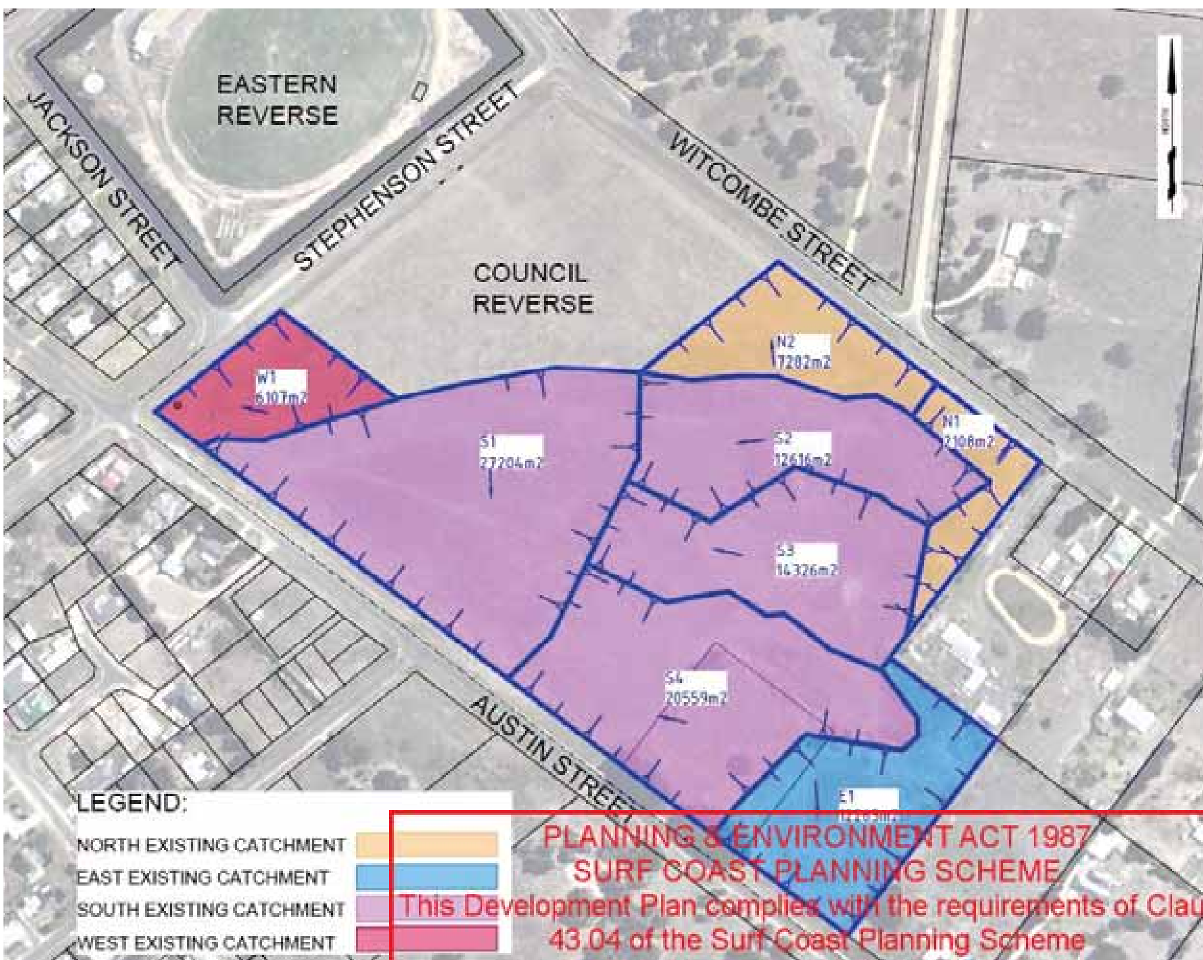

Figure 5-1 Overall Existing Catchments.

5.1.2 Design Catchment Delineation

Determination of design catchments was devised by undertaking a conceptual design which indicated the proposed developed flow paths within the site. Developed catchments can be seen in Figure 5-2.

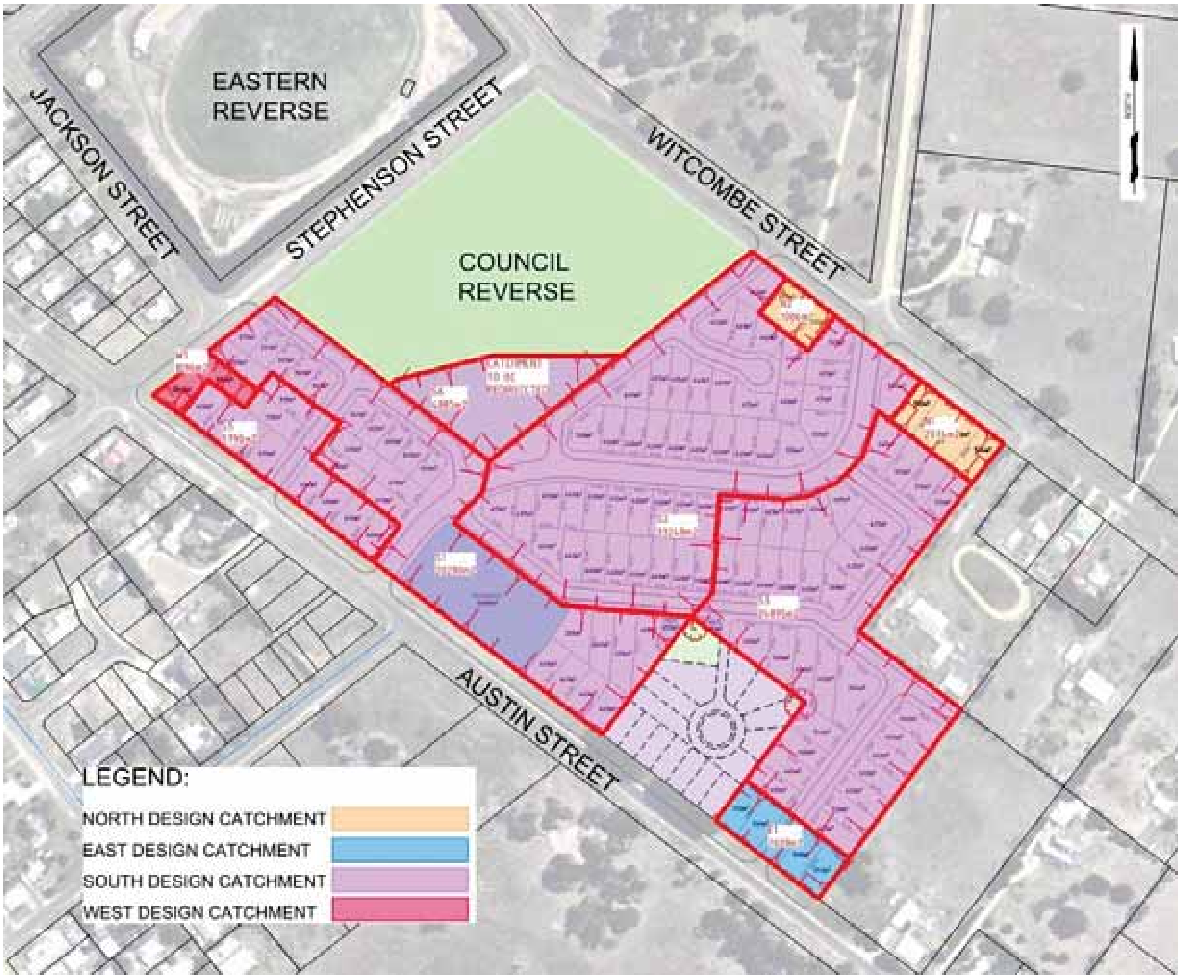

Figure 5-2 Overall Design Catchments

5.2 Model Parameters

5.2.1 Permeability and Fraction Impervious

A fraction impervious percentage was assigned to the catchments to reflect the excepted permeability based on planning context and actual land use.

To assign fraction impervious values for the developed conditions, lot sizes indicated within the Overall Development Plan (ODP) were related to the relevant impervious fractions noted in Melbourne Water[3].

[3] Melbourne Water (2018). MUSIC Guidelines – Input parameters and modelling approaches for MUSIC users in Melbourne Water's service area 2018.

Table 5-1 Fraction Impervious

Land Use	Impervious Surface (%)
Pre-developed Agricultural	0.10
Residential Growth Zone – Standard Densities	0.75

Details of the existing conditions input values are shown in Table 5.2 below.

Table 5-2 Existing Catchment Parameters

Catchment	Area (ha)	Fraction Impervious	Impervious Area (ha)	Slope (%)
S1	2.7204	0.100	0.272	2.35
S2	2.0559	0.100	0.206	2.06
S3	1.4326	0.100	0.143	1.08
S4	1.2616	0.100	0.126	1.38
E1	1.2263	0.100	0.123	1.22
W1	0.6107	0.100	0.061	2.81
N1	0.4053	0.100	0.041	2.46
N2	0.7282	0.100	0.073	3.14

Details of the proposed conditions input values are shown in Table 5.3 below.

Table 5-3 Design Catchment Parameters

Catchment	Area (ha)	Fraction Impervious	Impervious Area (ha)	Slope (%)
S1	2.03	0.750	1.52	1.00
S2	3.32	0.750	2.49	0.52
S3	2.49	0.750	1.87	0.94
S5	0.57	0.750	0.43	2.71
S6	0.49	0.100	0.049	3.69
W1	0.11	0.750	0.08	3.38
E1	0.26	0.750	0.19	2.55
N1	0.21	0.750	0.16	2.86
N2	0.11	0.750	0.08	2.46

5.2.2 Loss Parameters

XP-STORM was run as an Initial Loss (IL) and Continuing Loss (CL) model using parameters provided from the ARR Data Hub[4].

The hydrologic losses adopted in this study are summarised in Table 5-4 below.

Table 5-4 Adopted Hydrological Loss Parameters

Surface	Storm Initial Loss (mm)	Pre-Burst Depth (mm)	Adopted Losses	
			Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Pervious	20.00	1.80	19	3.5
Impervious	0		0	0

5.2.3 Manning's Roughness Coefficients

In the hydrological model, all sub-areas are also characterised by Manning's 'n' coefficients, which describe the hydraulic roughness properties of the soil surfaces.

The Manning's coefficients adopted in this study are summarised in Table 5.5.

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[4] ARR Data Hub, http://data.arr-software.org/ . Accessed on 5th September 2019

Table 5-5 Manning's Coefficients 'n' Adopted in the Hydrological Model.

Surface	Manning's Coefficients 'n'
Pervious	0.050
Impervious	0.018

The Manning's coefficients for the pervious surface have been further analysed through the sensitivity analysis process (Section 5.2.4)

5.2.4 Sensitivity Analysis

The hydrological models' parameters are determined through a calibration procedure to optimise the model performances in relation to a specific site. Indeed, the choice of the hydrological model parameters usually reflect the characteristics of the site and the soil properties.

The most used calibration procedure in rainfall-runoff hydrological models involves the comparison between observed and computed data. In the calibration phase, hydrological model parameters are adjusted to attain am output that matches the observed data. However, absence of gauged data, both calibration and validation are not possible.

The Regional Flood Frequency Estimation (RFFE) tool provided by ARR2016 indicated peak flood estimates for rural catchments and cannot be applied directly to urban catchments (where more than 10% of the catchment is affected by residential or urban development).[5] The RFFE cannot be used to define the expected runoff discharge from the 'existing catchment', however, it can be used to define excepted runoff discharges from a 'pre-urban development' catchment.

The RFFE tool was employed as a point of reference in the assessment of the suitability and sensitivity of the selected hydrological parameters.

The sensitivity analysis was setup to reflect rural or pre-development catchments to allow the comparison with flood estimation techniques. More in detail, all sub-areas have been considered as rural, and the pervious fraction has been set at 100%.

A key element of the sensitivity analysis process is the identification of the stormwater catchments that impact on the study area, the characteristics of those catchments and the configuration of waterways.

Factors such as availability of observed rainfall data, soil type, soil conditions, land use and local knowledge were considered in this investigation.

The sensitivity analysis has been undertaken to the 10% AEP event identified using the ARR2016 tool for the lumped catchment.

The 10% AEP event was selected – as the recorded data used to generate the RFFE discharges have a larger sample and more robust records of 10% AEP events. This provides more reliable flood frequency estimate.

The sensitivity analysis procedure has been performed by changing the Manning coefficient 'n' for the pervious surfaces to better represent the energy losses within the study area.

A further analysis of the model predictability has been carried out by comparing the XP-STORM model and the RFFE predicted peak discharges for the 20%, 10 % and 1% AEP events.

[5] ARR – Limits of Applicability – https://rffe.arr-software.org/limits.html , viewed on 6th September 2019

5.2.5 ARR2016 Regional Flood Frequency (RFFE) Model

The ARR2016 RFFE model[6][7] available online at http://arr.ga.gov.au, was used to provide peak flow estimates for the study catchments.

The RFFE model provide peak flood estimates for rural catchments, therefore, the lumped model was initially considered to be undeveloped (pre-development) to allow comparison.

5.2.6 Storm Burst Pattern Ensemble

The XP-STORM model applied ensemble rainfall patterns, storm burst loss factors and runoff estimation techniques from ARR2016[8] to the study catchment area to generate runoff hydrographs and predict the volume of stormwater generated.

As detailed in ARR2016[9] the majority of hydrograph estimation methods used for flood estimation require a temporal pattern that describes how rainfall falls over time as a design input.

The importance of temporal patterners has increased as the practice of flood estimation has evolved from peak flow estimation to full hydrology estimation.

An ensemble of 240 storms was analysed within the XP-STORM model for each storm event probability. For the sensitivity analysis process, the study catchment was set up as a rural catchment with no impervious surfaces.

Using the burst initial loss and continuing loss identified in Table 5-4, and known catchment characteristics, i.e. area, slope, overland flow path profiles, etc. the model was run using all 240 storm burst patterns for each AEP.

ARR2016 states that the temporal pattern that represents the worst (or best) case should not be used by itself for design. Testing has demonstrated that on most catchments large number of events in the ensemble patterns are clusters around the mean and median[10]. Based on this guidance the design has adopted the temporal pattern producing the median peak flow rate at the catchment outlet.

5.2.7 Results Summary

Based on the sensitivity analysis, the Manning's 'n' was refined. The comparison between the peak discharges generated with the XP-STORM model and the estimated RFFE model for the catchment is summarised in Table 5-6 and shown in Figure 5-3.

The hydrological parameters defined by the catchment characteristics, were capable of generating discharges within an acceptable range of the predicted RFFE discharge targets for all event probabilities.

[6] Rahman. A, et al (2013). New Regional Flood Frequency Estimation (RFFE) Method for the whole of Australia: Overview of progress. Paper. Flood plain conference 2013.
[7] Rahman. A, Haddad. K, Kuczera. G and Weinmann. E, 2016, Peak Flow Estimation, Chapter 3 Book 3 in Australian Rainfall and Runoff – A Guide to Flood Estimation, Commonwealth of Australia.
[8] Ball. J, Babister. M, Nathan. R, Weeks. W, Weinmann. E, Retallick. M, Testoni. I, (Editors), 2016, Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia.
[9] Babister. M, Retallicj. M, Loveridge. M, Testoni. I and Podger. S, 2016, Temporal Patterns. Chapter 5 Book 2 in Australian Rainfall and Runoff- A Guide to Flood Estimation, Commonwealth of Australia.
[10] Babister. M, Retallicj. M, Loveridge. M, Testoni. I and Podger. S, 2016, Temporal Patterns. Chapter 5 Book 2 in Australian Rainfall and Runoff- A Guide to Flood Estimation, Commonwealth of Australia.

Table 5-6 Manning's 'n' and Peak Discharges

Event AEP (%)	Area (ha)	Manning's 'n' Adopted	RFFE Discharge (m^3/s)	XP STORM Discharge (m^3/s)
20	10.441	0.050	0.09	0.07
10			0.12	0.10
1			0.28	0.33

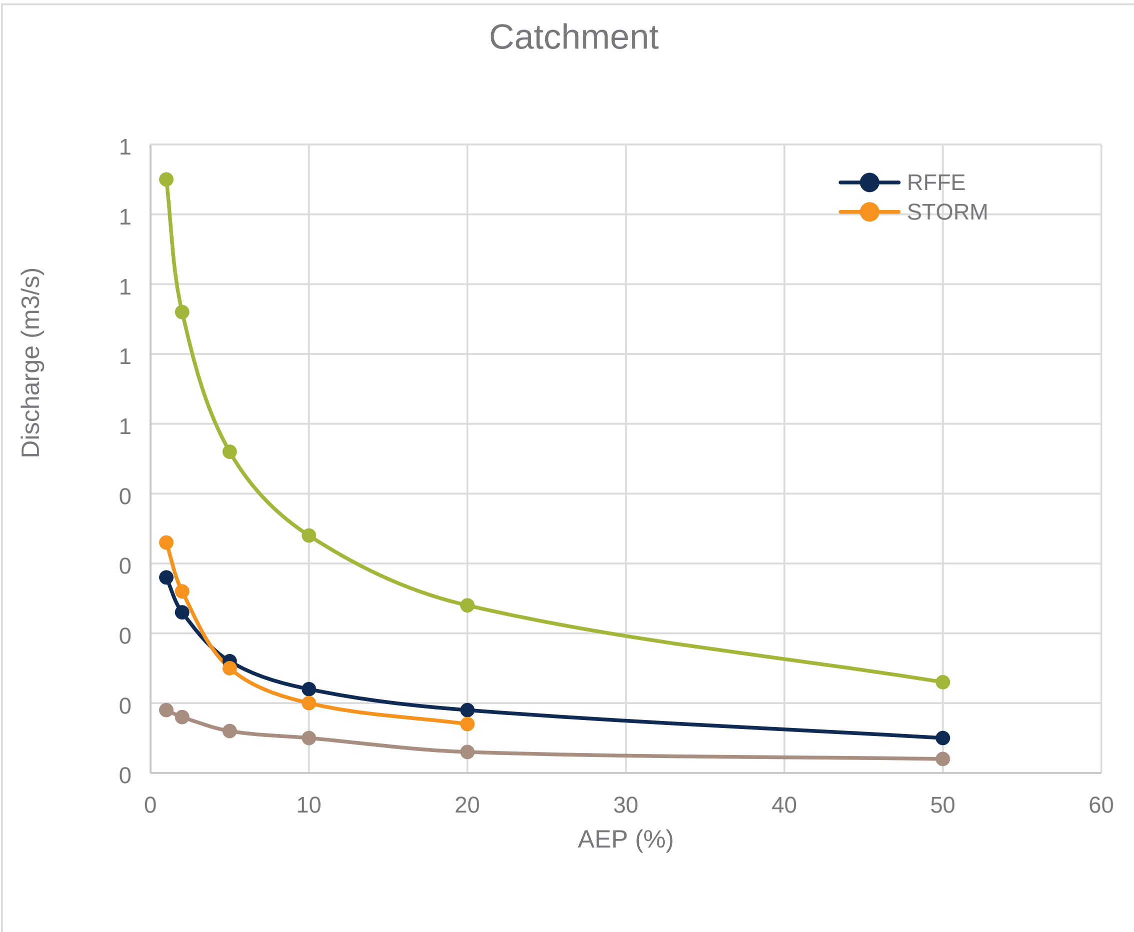


Figure 5-7 Estimated Peak Discharge – XP-STORM vs RFFE

5.2.8 RFFE Accuracy Considerations

A RFFE technique essentially represents a 'transfer function' that converts predictor variables to a flood quantile estimate. It is assumed that the use of a limited number of predictor variables (e.g. catchment area and design rainfall intensity) combined with an optimised transfer function captures the general nature of the rainfall-runoff relationship for flood events and hence provides flood quantile estimates of 'acceptable' accuracy.

ARR2016 identified ongoing concerns about estimation for parameter values (such as runoff coefficient and time of concentration) that are the basis of using the Probabilistic Rational Method[11].

The use or application of the Probabilistic Rational Method, including the VicRoads variant, is no longer supported or recognised in ARR2016 as being suitable RFFE technique for larger catchments and volume driven designs[12][13].

All RFFE techniques are subject to uncertainty, which, generally, is likely to be greater than for at-site Flood Frequency Analysis when a good quality and long record of streamflow data set is available at the location of interest.

The RFFE model estimates of regional flood frequency included substantial error bounds and are considered to be a best estimate or rarer events that cannot be described in the ungauged catchment. Recent studies[14] show how hydrological parameters from gauged catchments can be transferred to nearby ungauged catchments with similar natural characteristics.

5.3 Temporal Pattern Selection

5.3.1 Temporal Pattern Concept

In order to properly understand the concept of temporal pattern, it is necessary to understand the components of a storm event and how they relate to Intensity Frequency Duration Data (IFD) and catchment response.

Components of a typical storm patterns have been characterised in Figure 5-4. It is important to note the components can be characterised either by IFD relationships or by catchment response and are highly dependent on the definitions used. The components of a storm include:

1. Antecedent rainfall (Pre-Storm Rainfall) – Is rainfall that had fallen before the storm event and is not considered part of the storm but can affect catchment response. This is important to understand when calibrating to or modelling historic events.
2. Pre-burst rainfall – Is storm rainfall that occurs before the main burst. With the exception of relatively frequent events, it generally does not have a significant influence on catchment response but is very important for understanding catchment and storage conditions before the main rainfall burst. Pre-burst rainfall often accounts for a proportion of the initial losses within a catchment. Pre-burst depths need to be quantified when only modelling storm burst patterns.
3. The burst – represents the main part of the storm but is very dependent on the definition used. Bursts have typically been characterised by duration. The burst could be defined as the critical rainfall burst, the rainfall period within the storm that has the lowest probability, or the critical response burst that corresponds to the duration which produces the largest catchment response for a given rainfall Annual Exceedance Probability (AEP).
4. Post-burst rainfall – Is rainfall that occurs after the main burst and is generally only considered when aspects of hydrograph recession are important. This could be for draining down a dam after a flood effect or understanding how inundation times affect flood recovery, road closures, agricultural land or detention basin design.

[11] Coombes P.J., Babister M., and McAlister A., 2015, Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments. 36th Hydrology and Water Resources Symposium, Engineers Australia, Hobart.
[12] Rahman. A, Haddad. K, Kuczera. G and Weinmann. E, 2016, Peak Flow Estimation, Chapter 3 Book 3 in Australian Rainfall and Runoff – A Guide to Flood Estimation, Commonwealth of Australia
[13] Coombes P.J., Babister M., and McAlister A., 2015, Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments. 36th Hydrology and Water Resources Symposium, Engineers Australia, Hobart.
[14] Coombes, P. Colegate, M. Barber, L. Babister, M. 2016, Modern perspective on hydrology processes of two catchments in Regional Victoria. 37th Hydrologic and Water Resource Symposium 2016.

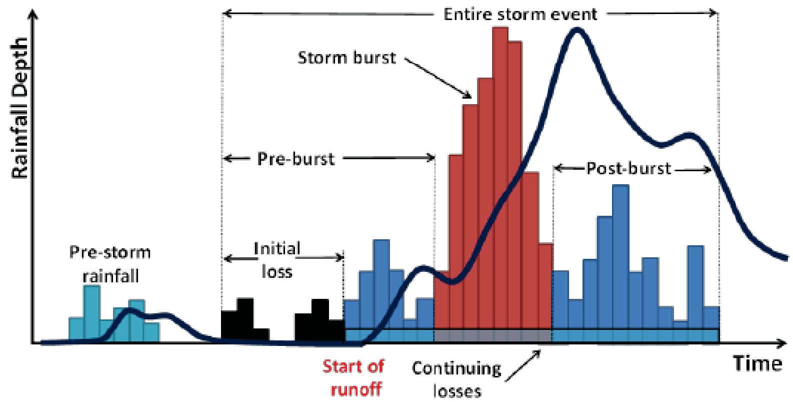


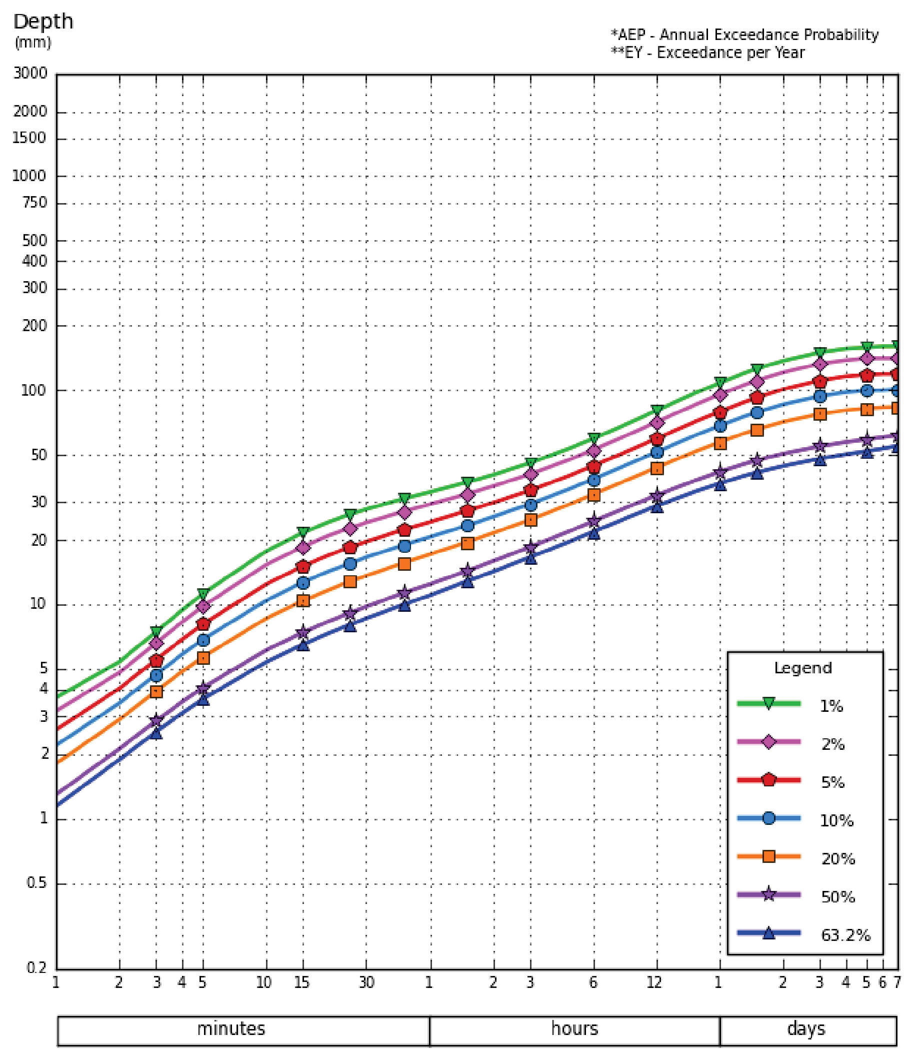
Figure 5-8 Elements of a Complete Storm Event and Hydrological Practice[15]

For this study, the Bureau of Meteorology's 2016 IFD data and ARR2016 temporal patterns were used to produce an ensemble of storm burst patterns which were analysed for a whole catchment response.

[15] Coombes, P. Colegate, M. Barber, L. Babister, M. 2016, A modern perspective on the hydrology processes of Armstrong Creek and Canadian Creek catchments in Regional Victoria. 4th National Conference on Urban Stormwater Management 2016.

5.3.2 Intensity Frequency Duration (IFD) Data

The 2016 rainfall intensity frequency duration (IFD) climatic data used in the hydrological model was extracted from the Bureau of Meteorology (BOM) website[16]. The IFD curves are shown in Figure 5-5.



Note:
The 50% AEP IFD **does not** correspond to the 2 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 1.44 ARI.
* The 20% AEP IFD **does not** correspond to the 5 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 4.48 ARI.

Figure 5-9 2016 IFD Curves – Bureau of Meteorology 2019

5.3.3 Critical Storm Burst Pattern Selection

The historical processes of using peak flows derived from a single critical storm burst does not account for the hydrological processes generated by the reality of complete (full volume) storms as

[16] Bureau of Meteorology, http://www.bom.gov.ua/water/designRainfalls/. Accessed on 5th September 2019

demonstrated in Figure 5-4. It is important to understand the hydrological losses within the catchment and the relationship of the losses to both full storms and storm bursts.

For this analysis, 10 storm burst temporal patterns were extracted for 24-hour duration periods, for each AEP event. By analysing the hydrological response to the ensemble temporal patterns, one critical pattern was selected for each of the 24 durations. The fixed temporal patterns over the entire study for design flood estimation were implemented and the spatial variation was not considered. The analysed events and durations are shown in Table 5-7.

Table 5-7 Analysed Rainfall Patterns, Durations and Events.

Number of Storm Burst Patterns in Ensemble (per event duration)	Storm Durations Analysed (minutes)		Event Probability Range Analysed (AEP)	
			(%)	(1 in x)
10	10	120	1	100
	15	180	10	10
	20	270	20	5
	25	360		
	30	540		
	45	720		
	60	1,080		
	90	1,440		

The median value of the peak discharges generated for 10 temporal patterns under pre-developed conditions has been calculated. The critical temporal pattern was selected by identifying the temporal pattern characterised by the peak discharge closest to the median for each of the 24-hour durations. The procedure has been then repeated for each of event probability.

5.4 Hydrological Model Simulations

Sensitivity analysis models applied 100% pervious surfaces within the catchments. Impervious surfaces and urban characteristics were integrated into the temporal pattern selection and existing/developed conditions hydrological models.

The modelling work was conducted through the study area for the 1%, 10% and 20% AEP. Adopted Storm temporal Burst Patterns for the subject site can be seen in Table 5-8.

Table 5-8 Adopted Storm Burst Patterns.

Duration	Temporal Pattern No.		
	1% AEP	10% AEP	20% AEP
10min	1	1	1
15min	3	1	1
20min	7	1	1
25min	1	1	1
30min	8	1	1
45min	1	1	1
1hour	3	1	1
1.5hour	4	7	3
2hour	5	4	1
3hour	7	2	5
4.5hour	8	7	2
6hour	2	2	1
9hour	8	6	6
12hour	7	4	4
18hour	3	2	4
24hour	4	1	1

6 Modelling Results

The results of the stormwater hydrology and water quality analysis are shown in this section. Design has been undertaken to meet stormwater quality 'best practice' standards and to calculate the requirements of site detention basins and constructed wetlands within the developed catchment.

6.1 Existing Site Discharge

The permissible site discharge (PSD) from the LPOD for each even probability was determined using the local hydrological model under existing conditions. The runoff hydrograph for the 1% AEP for the four catchments is shown in Figure 6-1 to Figure 6-4. The critical peak discharges for the 1%, 10% and 20% AEPs have been tabulated in Table 6-1 to Table 6-4.

Table 6-1 Validated Peak Discharges – Northern Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	20min	0.11
20%	10min	0.04

Table 6-2 Validated Peak Discharges – Southern Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	2hr	0.235
20%	10min	0.059

Table 6-3 Validated Peak Discharges – Eastern Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	2hr	0.062
20%	20min	0.022

Table 6-4 Validated Peak Discharges – Western Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	2hr	0.048
20%	15min	0.012

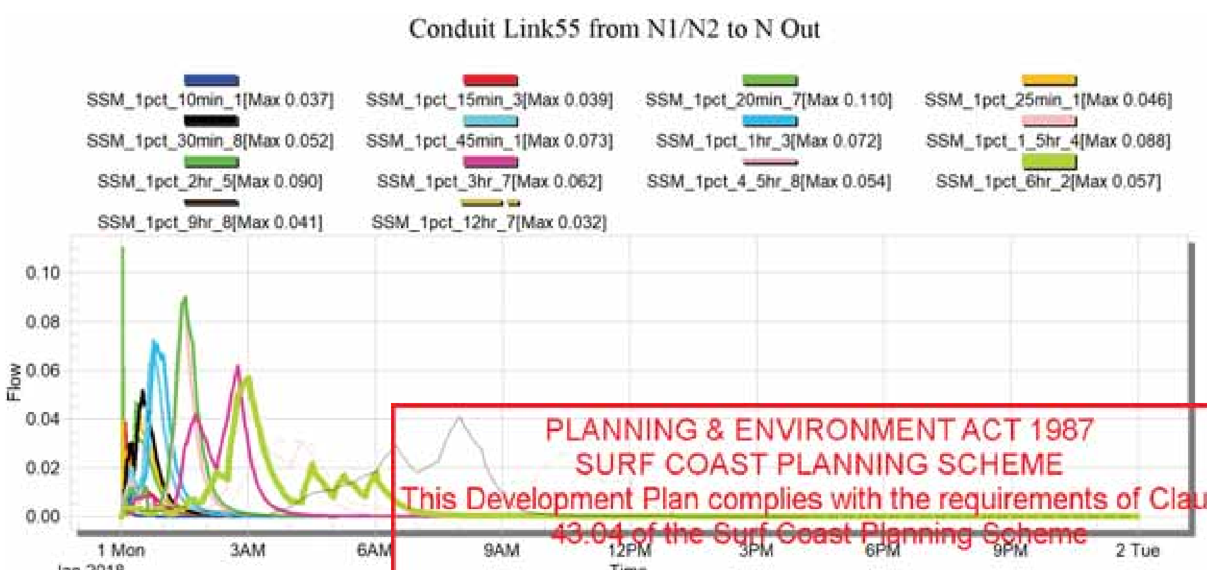


Figure 6-2 1% AEP Hydrographs (Northern Catchment)

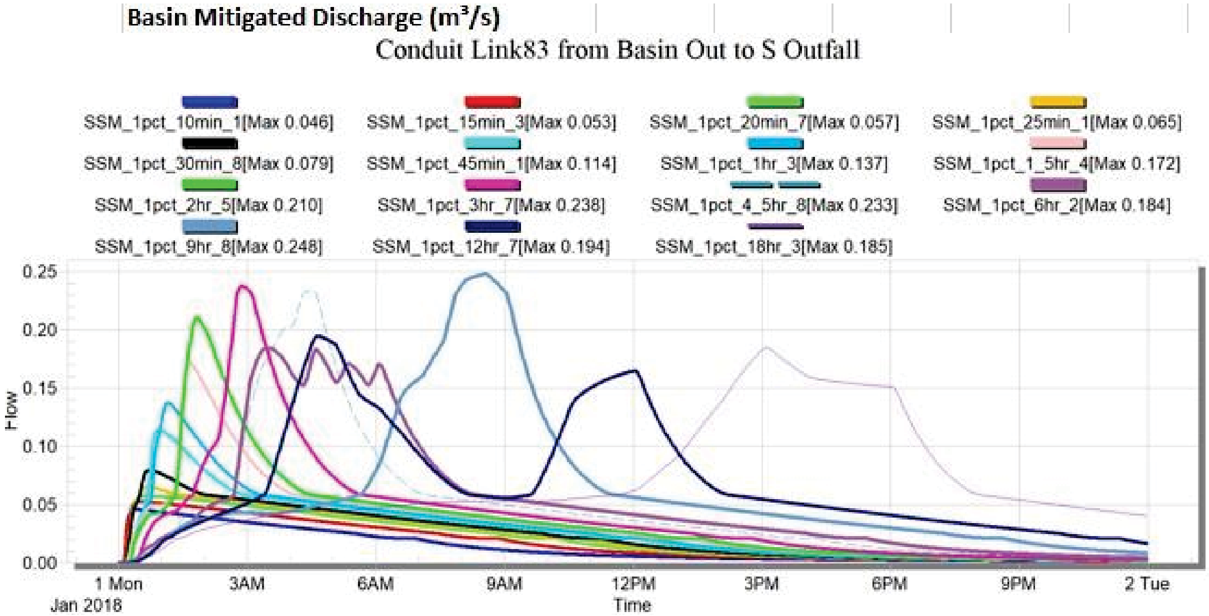


Figure 6-3 1% AEP Hydrographs (Southern Catchment)

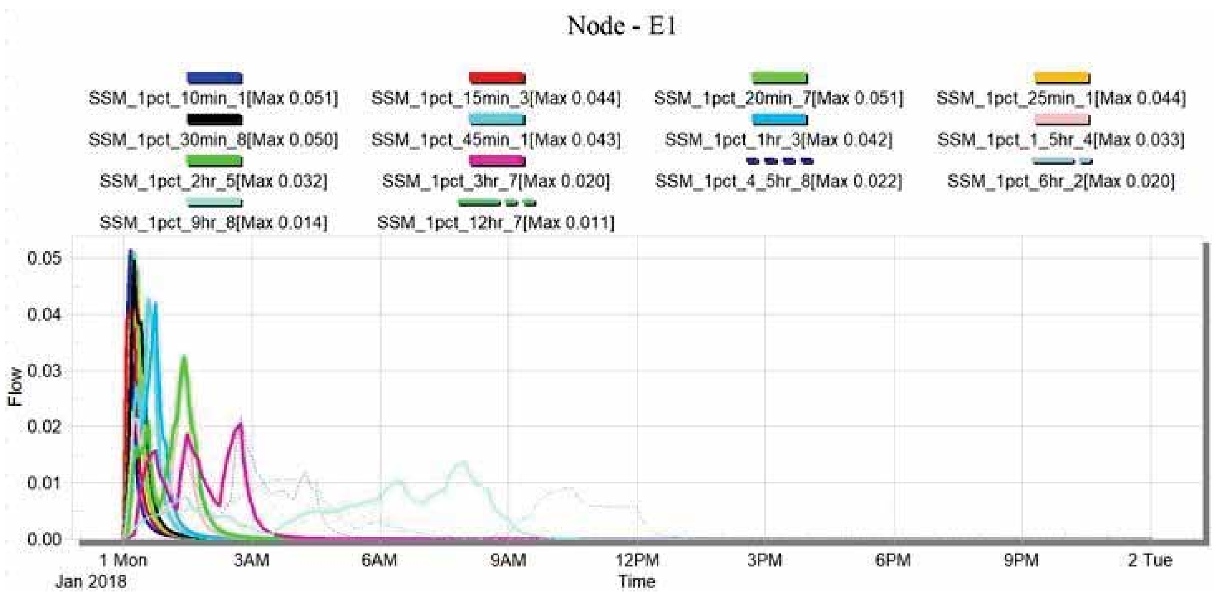


Figure 6-4 1% AEP Hydrograph (Eastern Catchment)

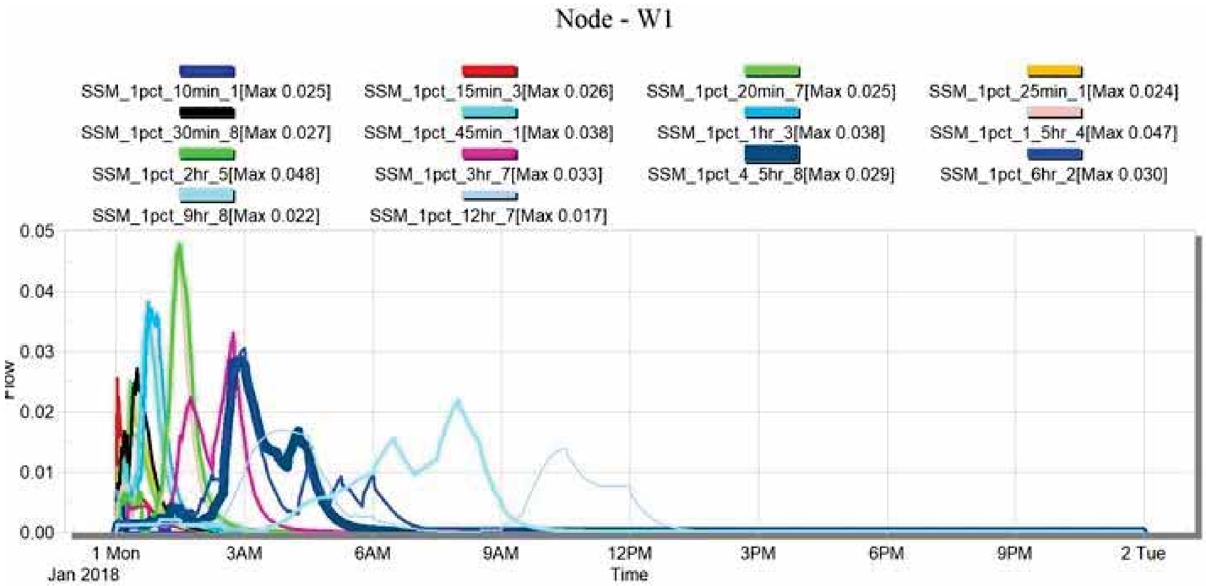


Figure 6-5 1% AEP Hydrograph (Western Catchment).

6.2 Developed Site Characteristics.

The characteristics of the major catchments guided the design process and the measures proposed to be implemented to ensure the site meets best practice guidelines.

Based on the concept subdivision plan, the site is separated naturally into 4 distinct catchments.

> Northern Catchment
> Eastern Catchment
> Southern Catchment
> Western Catchment

The hydrological analysis simulated with XP-STORM provided a detail distributed hydrological model of the study catchment. The resulting stormwater hydrographs were then evaluated to adopt an appropriate stormwater mitigation approach for the whole site.

6.2.1 Developed Conditions – Southern Catchment

The majority of the development will be directed to the central located stormwater treatment facility located within the southern catchment. The facility consists of a GPT, Sedimentation Basin and Bioretention Basin above these quality treatment measures will be a detention system designed to mitigate flows to pre-developed rates. The facility is proposed to discharge to the existing underground drainage network located within Baston Street.

Table 6-5 Developed Peak Discharge – Southern Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	9hr	0.235
20%	9hr	0.057

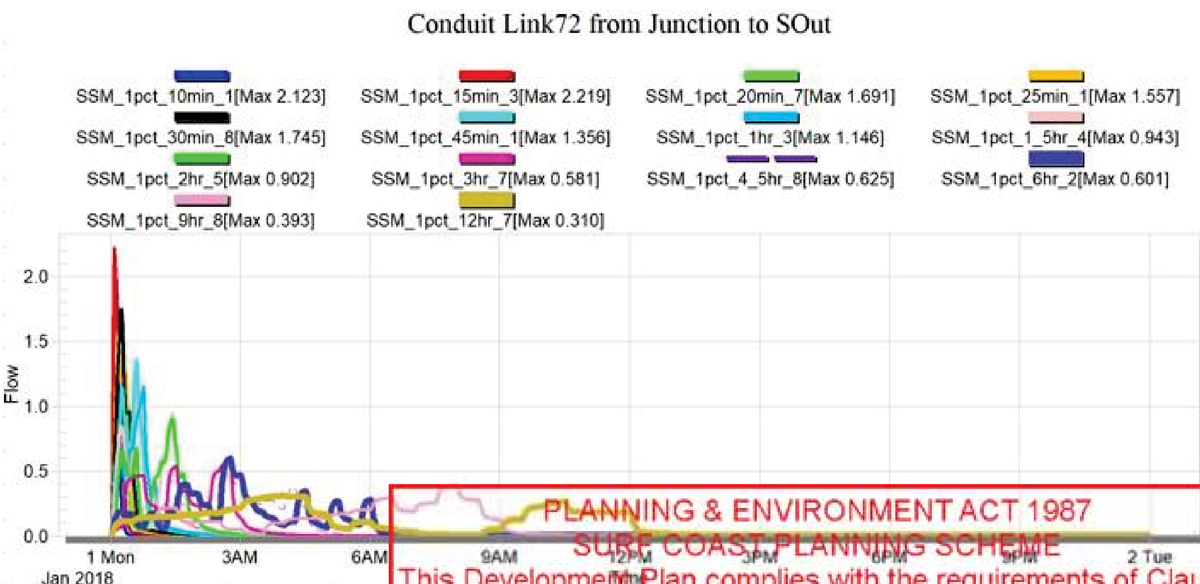


Figure 6-7 Developed 1% AEP Hydrograph – Southern Catchment

Detention basin retarding flows to pre-developed rates.

Table 6-6 Southern Stormwater Treatment Basin Details Summary

	1%AEP	20%AEP
Peak Outflow (m^3/s)	0.235	0.057
Critical Duration (min)	540	540
Top Water Level	92.02	91.74
Sedimentation Area (m^2)	750	750
Bioretention Area (m^2)	341	341
Basin Volume (m^3) (Top Water Level to Top Extended Detention	2996	1934
Orifice IL (m AHD)	91.76	91.25
Outlet Configuration (modelled)	0.15m High, 0.22m Side Orifice in Outlet Pit	0.8m Weir in top of Outlet pit

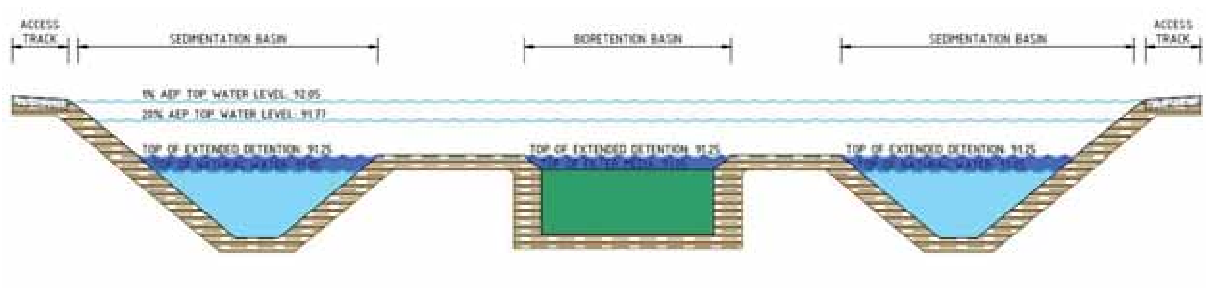


Figure 6-8 Typical Section of Proposed Basin.

6.2.2 Developed Conditions – Northern Catchment.

The northern catchment is located along the northern boundary of the site and consists of 0.606ha of agricultural land. This catchment will be discharged offsite without going through a stormwater treatment system, with additional treatment being undertaken in the south.

Table 6-7 Developed Peak Discharge – Northern Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	15min	0.062
20%	15min	0.033

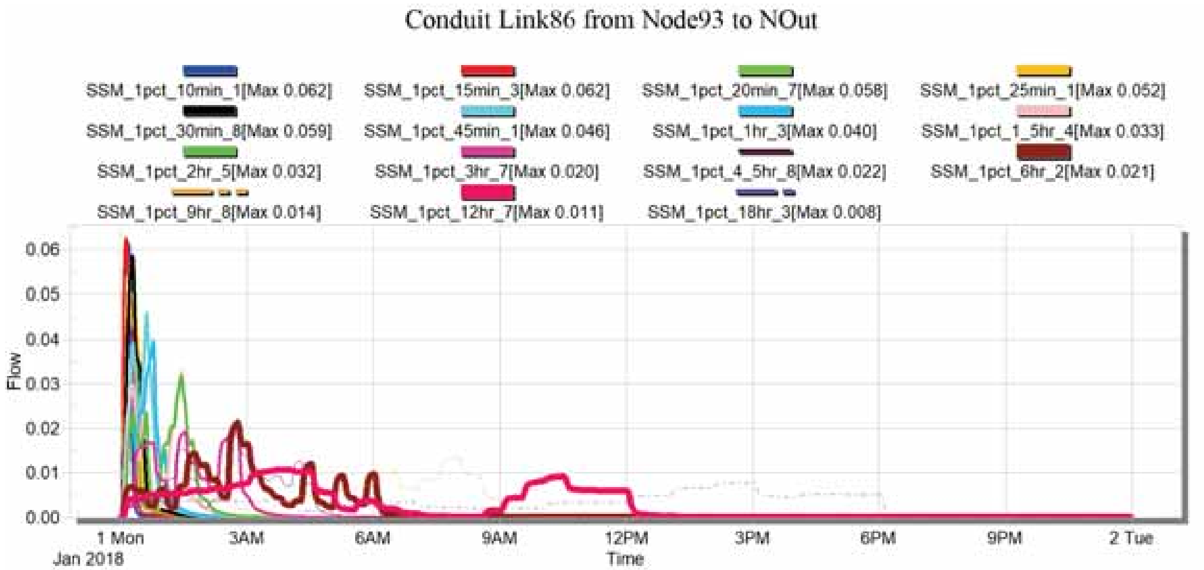


Figure 6-10 Developed 1% AEP Hydrograph – Northern Catchment

Due to the decrease in size of the northern catchment flows rates to not exceed pre-developed rates.

Table 6-8 Northern Catchment Outlet Summary

	1%	20%
Developed Peak Flow (m^3/s)	0.062	0.033
Developed Storm Duration (min)	15	15
Pre-Developed Peak Flow (m^3/s)	0.11	0.04
Pre-Developed Storm Duration (min)	120	15

During construction fronting Witcombe street new kerb and channel will be provided as well as an underground drainage system that will take lot and road runoff and discharge in the open drain on the eastern side of Gladman Street.

Flows in the 1% AEP are within 10%, minor flows are slightly higher but will be contained within the large outfall open drain located within Gladman Street discharging North.

6.2.3 Developed Conditions – Eastern Catchment

The eastern catchment is located in the South East corner of the development and consist of 0.311ha of agricultural land. This catchment is proposed to discharge into Austin Street un-detained.

Table 6-9 Developed Peak Discharge – Eastern Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	10min	0.042
20%	10min	0.017

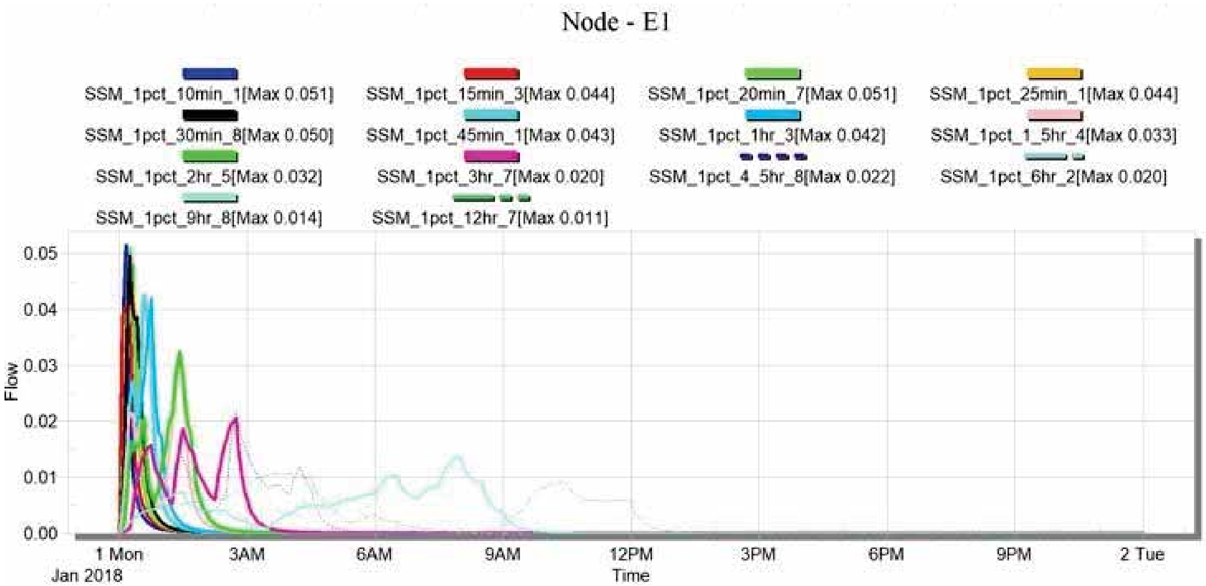


Figure 6-12 Developed 1% AEP Hydrograph – Eastern Catchment

By decreasing the size of the existing eastern catchment, the developed catchment discharge has been limited to pre-developed flow rates or below pre-developed flow rates in both major and minor.

Table 6-10 Eastern Catchment Outlet Summary

	1%	20%
Developed Peak Flow (m^3/s)	0.042	0.017
Developed Storm Duration (min)	10	10
Pre-Developed Peak Flow (m^3/s)	0.062	0.022
Pre-Developed Storm Duration (min)	120	20

Kerb and channel will be constructed along Austin Street along with underground drainage and discharge into an existing open drain out falling at the eastern corner of the site.

6.2.4 Developed Conditions – Western Catchment

The western catchment is located in the south-west corner or the development fronting both Austin and Stephenson Street. The catchment is 0.169ha agricultural land, with the catchment being proposed to drain freely into Austin Street and Stephenson Street.

Table 6-11 Developed Peak Discharge – Western Catchment

AEP	Critical Event Duration	Critical Peak Discharge (m^3/s)
1%	15min	0.030
20%	15min	0.013

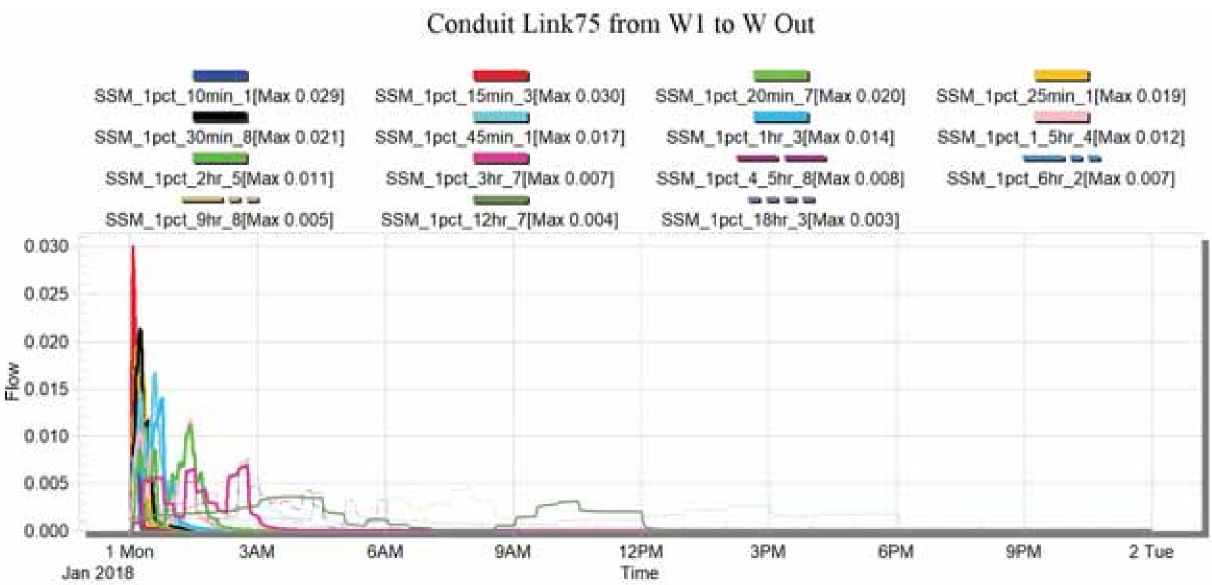


Figure 6-14 Developed 1% AEP Hydrograph – Western Catchment

Table 6-12 Western Catchment Outlet Summary

	1%	20%
Developed Peak Flow (m^3/s)	0.03	0.013
Developed Storm Duration (min)	15	15
Pre-Developed Peak Flow (m^3/s)	0.048	0.012
Pre-Developed Storm Duration (min)	120	15

1% AEP flowrates are within 10% of the predeveloped rates, 20% AEP is 12 l/s above the pre-developed initial flow estimations. It is anticipated that the additional storage provided within the underground network and/or minor additional redirection of flow at the detailed design stage will produce an satisfactory design discharge and design outcome.

6.3 Stormwater Quality Modelling

The ability of the development to meet stormwater quality 'best practice' standards and the performance of the treatment system was continuously simulated using MUSIC. The MUSIC model utilised rainfall data from the Winchelsea Post Office site from 1998 to 2010 which was sourced from the eWater pluviography rainfall data.

An overall view of the MUSIC model is provided below in Figure 6-10.

Details of treatment measures proposed throughout the site can be found in section *6.3.3.1* and *6.3.3.3* below.

Due to the existing topography of the site three catchments leave the site untreated. To ensure stormwater quality best practice measures are meet the **main catchment has been overtreated to ensure that the overall site exceeds water quality treatment requirements**.

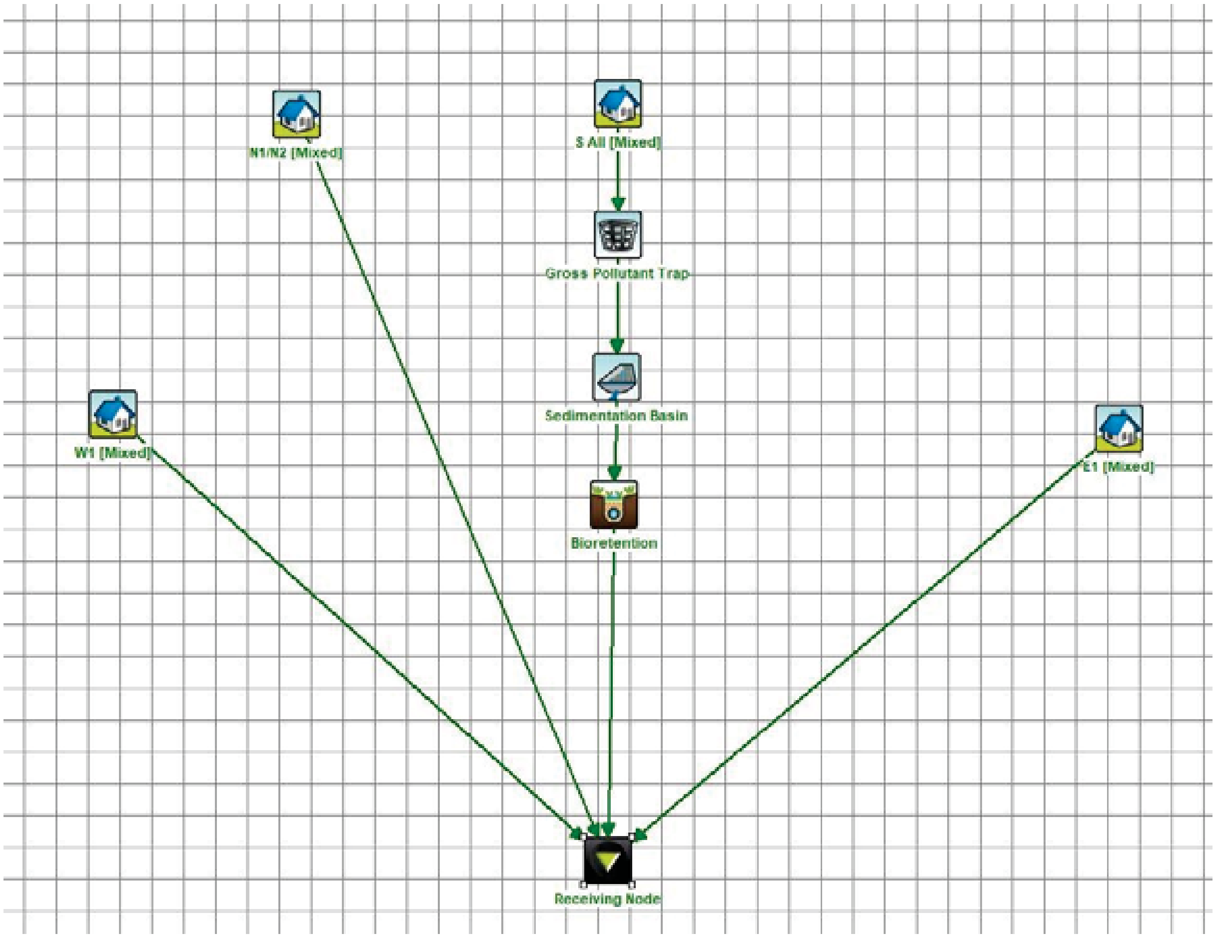


Figure 6-15 Overall MUSIC Model Network

6.3.2 Lot Based Inputs

The characteristics of the catchment modelled in MUSIC are detailed in Table 6-13. These characteristics have been compiled using City of Greater Geelong Design Notes No.3.

Table 6-13 MUSIC Simulations – Lot/Road Properties (Excluding Roof Area).

Catchment Characteristics	
Zoning/Surface Type	Mixed
Impervious Area (%)	75%
Pervious Area (%)	25%
Impervious Area Properties	
Rainfall Threshold (mm/day)	1.00
Pervious Area Properties	
Soil Storage Capacity (mm)	30
Initial Storage (% of Capacity)	30
Field Capacity (mm)	20
Infiltration Capacity Coefficient -a	200
Infiltration Capacity Coefficient – b	1
Groundwater Properties	
Initial Depth (mm)	10
Daily Recharge Rate (%)	25
Daily Baseflow Rate (%)	5
Daily Deep Seepage Rate (%)	0

6.3.3 End of line Treatment Inputs

The characteristics of the end of line treatment measures modelled in MUSIC are detailed in Table 6-14 to Table 6-16. The figures displayed below do not include additional land area requirements such as land taken up by internal and external batters, access tracks, offsets to building etc, which may form a large percentage of the overall footprint depending on the final location of the systems.

6.3.3.1 Gross Pollutant Trap (GTP)

Table 6-14 Gross Pollutant Trap – MUSIC Inputs

Inlet Properties		
Low Flow By-pass (m^3/s)	0	
High Flow By-pass (m^3/s)	100	
Target Element	Flow Based Capture Efficiency	
	Inflow	% Capture
Gross Pollutants (kg/ML)	0.0	99
	1.0	99
Total Suspended Solids (kg/ML)	0.0	55
	1.0	55
Total Phosphorus (mg/L)	0.0	0
	1.0	0
Total Nitrogen (mg/L)	0.0	0
	1.0	0

6.3.3.2 Sedimentation Basin.

Table 6-15 Sedimentation Basin – MUSIC Inputs

Inlet Properties	
Low Flow By-pass (m^3/s)	0.0
High Flow By-pass (m^3/s)	100.0
Storage Properties	
Surface Area (m^2)	750
Extended Detention Depth (m)	0.20
Permanent Pool Volume (m^3)	450
Initial Volume (m^3)	450
Exfiltration Rate (mm/hr)	0
Evaporative Loss as % of PET	75
Outlet Properties	
Equivalent Pipe Diameter (mm)	300
Overflow Weir Width (m)	2.0
Notional Detention Time (hrs)	0.444

6.3.3.3 *Bioretention Basin*

Table 6-16 Bioretention Basin – MUSIC Inputs

Inlet Properties	
Low Flow By-pass (m^3/s)	0.0
High Flow By-pass (m^3/s)	100.0
Storage Properties	
Extended Detention Depth (m)	0.20
Surface Area (m^2)	250
Filter and Media Properties	
Filter Area (m^2)	250
Unlined Filter Media Perimeter (m)	60
Saturated Hydraulic Conductivity (mm/hr)	100
Filter Depth (m)	0.50
TN content of Filter Media (mg/kg)	800
Orthophosphate Content of Filter Media (mg/kg)	55
Infiltration Properties	
Exfiltration Rate (mm/hr)	0
Lining Properties	
Is base lined?	No
Vegetation Properties	
Vegetated with Effective Nutrient Removal Plants	
Outlet Properties	
Overflow Weir Width (m)	2.00
Underdrain Present	Yes
Submerged Zone with Carbon Present	Yes
Depth	0.30

6.3.4 End of Lines Efficiencies

The efficiencies of the treatment train described above is as follows;

Table 6-17 Stormwater Quality Treatment Efficiencies

Criteria	Reduction (%)	
	Results	Target
Total Suspended Solids (kg/yr)	80.1	80
Total Phosphorus (kg/yr)	51	45
Total Nitrogen (kg/yr)	45.2	45
Gross Pollutants (kg/yr)	87.9	70

Efficiencies and sizes of treatment measures noted throughout have the ability to be optimised throughout the detailed design phase.

7 Conclusions

It can be seen that by conceptually designing the subdivision an 8.87ha catchment can be directed to a centrally located treatment facility which allows developed flows to be detained to pre-developed rates for both the 1%AEP and 20%AEP. Given the existing topography of the site three minor catchments were unable to be routed to the treatment facility, by minimising the catchment size developed flows were able to be restricted to pre-developed rates.

Table 7-1 Overall Site Pre-Development Discharge Summary

	1%	20%
North Pre-Developed Peak Flow (m^3/s)	0.11	0.04
North Pre-Developed Storm Duration (min)	120	15
East Pre-Developed Peak Flow (m3/s)	0.062	0.022
East Pre-Developed Storm Duration (min)	120	20
South Pre-Developed Peak Flow (m3/s)	0.235	0.059
South Pre-Developed Storm Duration (min)	120	10
West Pre-Developed Peak Flow (m3/s)	0.048	0.012
West Pre-Developed Storm Duration (min)	120	15

Table 7-2 Overall Site Developed Discharge Summary

	1%	20%
North Developed Peak Flow (m^3/s)	0.062	0.033
North Developed Storm Duration (min)	15	10
East Developed Peak Flow (m3/s)	0.042	0.017
East Developed Storm Duration (min)	10	10
South Developed Peak Flow (m3/s)	0.235	0.057
South Developed Storm Duration (min)	540	540
West Developed Peak Flow (m3/s)	0.03	0.013
West Developed Storm Duration (min)	15	15

The site stormwater objectives for the Austin Street development can be achieved for stormwater discharge quality and quantity control utilising techniques including GTP, Sedimentation, Detention and Bioretention (or approved proprietary products) centrally located within a single overall basin. By oversizing the basin to equate for the three untreated catchments quality targets can be achieved for the site.

Table 7-3 Stormwater Quality Treatment Efficiencies – Overall Site

Criteria	Reduction (%)	
	Results	Target
Total Suspended Solids (kg/yr)	80.1	80
Total Phosphorus (kg/yr)	51	45
Total Nitrogen (kg/yr)	45.2	45
Gross Pollutants (kg/yr)	87.9	70

Appendix A Overall Development Plan

DEVELOPMENT PLAN

135-235 AUSTIN STREET
WINCHELSEA

SITE AREA	**9.085 HA**
ENCUMBERED OPEN SPACE	
Tree Reserve	0.037 HA
Drainage Reserve	0.490 HA
TOTAL	**0.527 HA**
NET DEVELOPABLE AREA	**8.558 HA**
Road Reserves	2.612 HA

LOT AREAS

300m^2 - 400m^2	13
400m^2 - 500m^2	78
500m^2 - 600m^2	27
600m^2 - 700m^2	7
700m^2 - 800m^2	0
800m^2 - 900m^2	1
TOTAL	**126**

LOT DENSITY *(PER NDA)*	**14.7 HA**
AVERAGE LOT SIZE	471m^2

STAGE	AREA	LOTS
1	1.608 HA	18
2	0.968 HA	12
3	1.232 HA	19
4	1.176 HA	18
5	0.761 HA	11
6	0.706 HA	12
7	1.457 HA	20
8	1.177 HA	16

STAGE BOUNDARIES
RESIDENTIAL LOT
RESERVE - OPEN SPACE
RESERVE - BASIN
TRAFFIC MANAGEMENT MEASURE
PROPOSED 1.5m WIDE PATH
PROPOSED 2.5m WIDE PATH
TREE TO BE REMOVED

Cardno TGM
ABN 11 125 568 461
Level 1, 27-31 Myers Street (PO Box 1137) Geelong, VIC Australia 3220
Phone +61 3 5202 4600 Fax +61 3 5202 4691
Email: victoria@cardno.com.au Web: www.cardno.com

P:\17797 - 135-235 AUSTIN STREET WINCHELSEA\PLANS\DEVELOPMENT PLANS\17797-100-V18.DWG

REF	17797-100
SCALE	1:2000 (@A3)
DATE	15/06/2021
VERSION	18

Appendix B Feature Survey

PERMANENT SURVEY MARKS (SMES)			
DESCRIPTION	MGA94 ZONE 55 EASTING	MGA94 ZONE 55 NORTHING	AHD LEVEL
LAKE WOLLARD PM	237647.542	5763078.959	92.907
LAKE WOLLARD PM	237075.145	5762712.033	94.098

SURVEY CONTROL POINTS
DATE OF SURVEY: 01/11/2018

DESCRIPTION	PLANE EASTING	PLANE NORTHING	AHD LEVEL
TBM 1 - STAR PICKET	237706.61	5763022.29	93.02
TBM 2 - STAR PICKET	237421.00	5763234.26	92.16
TBM 3 - STAR PICKET	237427.89	5763274.81	92.03
TBM 4 - STAR PICKET	237214.90	5763052.58	94.03
TBM 5 - STAR PICKET	237155.06	5763041.54	92.39
TBM 6 - STAR PICKET	237419.46	5762877.81	91.74
TBM 7 - STAR PICKET	237494.20	5762790.36	94.73
TBM 8 - STAR PICKET	237619.30	5762768.32	94.56

Legend

001	PSM
003	TITLE PEG
015	STAR PICKET
102	TOP OF BANK
103	TOE OF BANK
104	NATURAL SURFACE
201	TREE
308	SIDE ENTRY PIT
310	JUNCTION PIT
316	WING WALL
320	PIT INVERT
321	PIPE INVERT
401	CENTRELINE
402	BITUMEN - SPOT
403	BITUMEN - EDGE
406	KERB - LIP
407	KERB - INVERT
408	KERB - BACK
410	PATH
411	DRIVEWAY
412	TRACK
503	SIGN
711	LIGHT POLE
712	ELECTRICITY - POLE
713	[redacted]
716	ELECTRICITY - PIT
717	ELECTRICITY - OVER HEAD
721	TELSTRA - PIT
722	TELSTRA - PILLAR
723	[redacted]
751	STOP VALVE
752	FIRE PLUG
753	FIRE HYDRANT
755	WATER METER
758	TAP
760	TANK
761	STAY
903	FENCE
904	GATE

Rev	Revision	Date

TGM
1/27-31 Myers Street (PO Box 1137)
Geelong Vic 3220
T 03 5202 4600
03 5202 4691
ABN 11 125 568 461

[redacted]

Services and assets have been plotted using their visible surface markers and/or from service authority records.

Additional unsighted services may exist.

Prior to any design, excavation or construction upon the site, the relevant authority should be consulted to verify locations of all surveyed services and check for the existence of any unsighted services.

[redacted]

Levels on this plan are to: Australian Height Datum (AHD) **based upon:** PM 28 With R.L 92.907
Contour Interval: 0.20m
Co-ordinate origin for this drawing is: M.G.A 94 Zone 55 **based upon:** PM 28 with E: 237647.542 N: 5763078.959
Bearing datum is: M.G.A 94 Zone 55 **based upon:** PM 28 and PM 31

[redacted]

This plan is prepared for the purposes of site evaluation and planning.

Bou[redacted] shown are approximate only and are subject to a title [redacted]ablishment survey.

[redacted] are an integral part of the plan, if the plan is [redacted] these notes are to accompany the copy.

Certification

I, [redacted] hereby certify that this [redacted] used to compile the plan are accurate and any [redacted] features shown are compiled from the sources [redacted]

Licensed Surveyor ([redacted] Act 2004) Date

PLAN OF FEATURE & LEVEL SURVEY

135 - 235 AUSTIN STREET
WINCHELSEA

ANTHONY ROBERT GRANT P/L

Sheet: 1 of 5
Date of Survey: 01/11/18
Date of Issue: 08/11/18
LENGTHS ARE IN METRES
1:25 0 1.25 2.5 3.75 5 6.25
At Size A1
DWG: 17797-100-FS01
REV: 1

PLANNING & ENVIRONMENT ACT 1987
SURF COAST PLANNING SCHEME
This Development Plan complies with the requirements of Clause 43.04 of the Surf Coast Planning Scheme

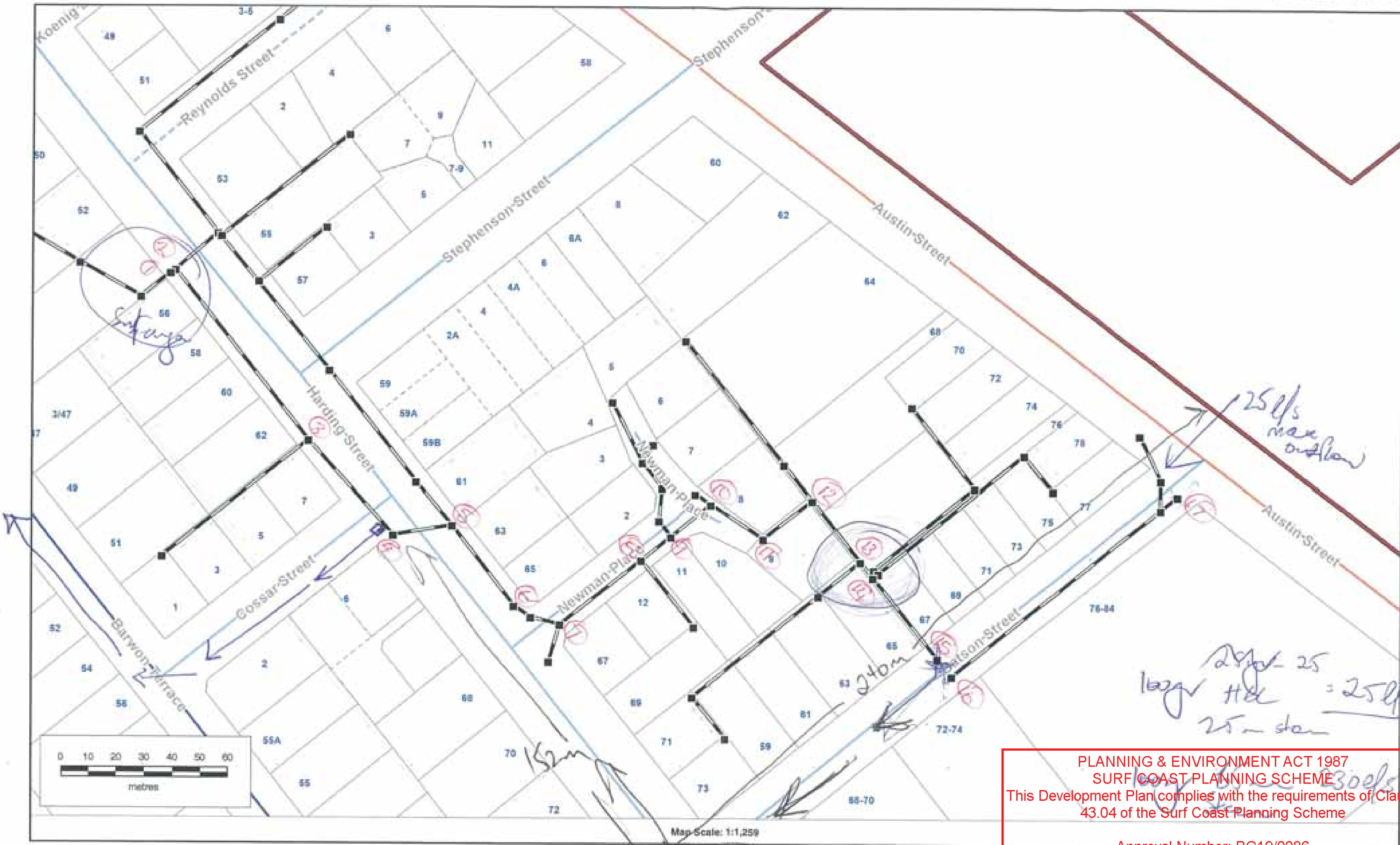
Approval Number: PG19/0086
Date: 4/08/2021 Sheet No: 52 of 56

Digitally Signed by the Responsible Authority
Karen Hose

THIS IS NOT A BUILDING APPROVAL

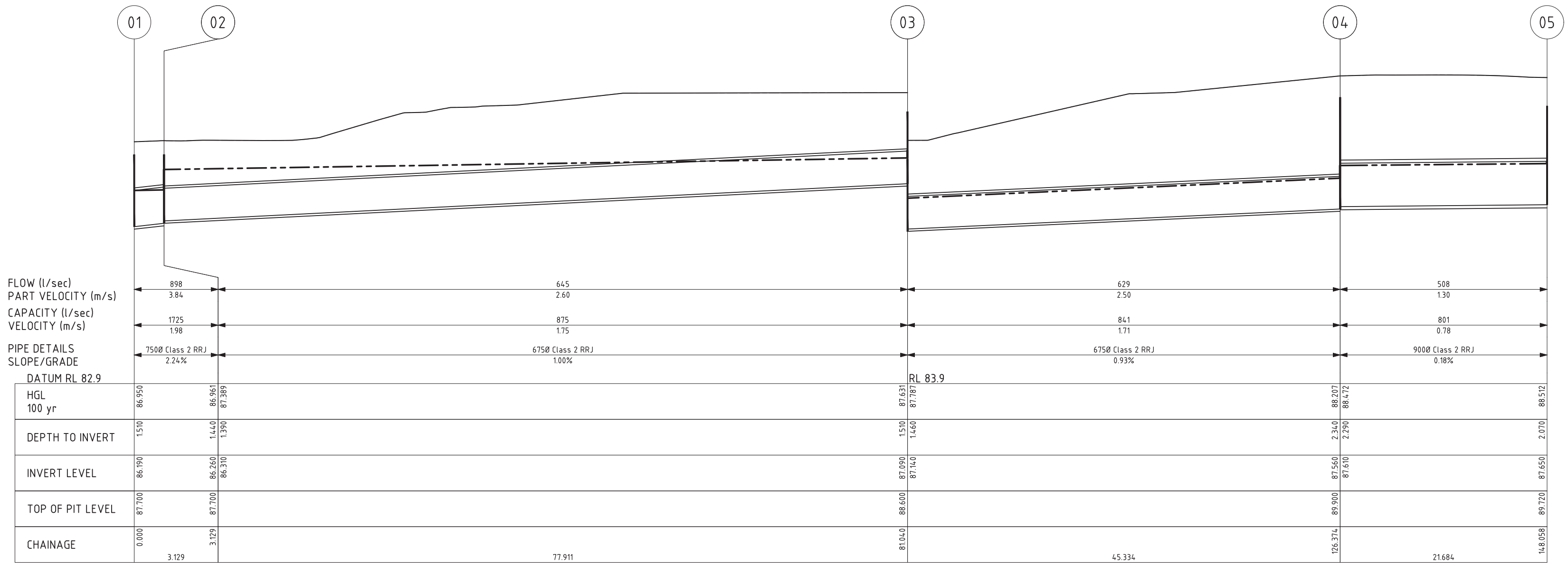
Appendix C Existing Drainage Information from Council

Created on Friday, 22 November 2019 by WACHERulm



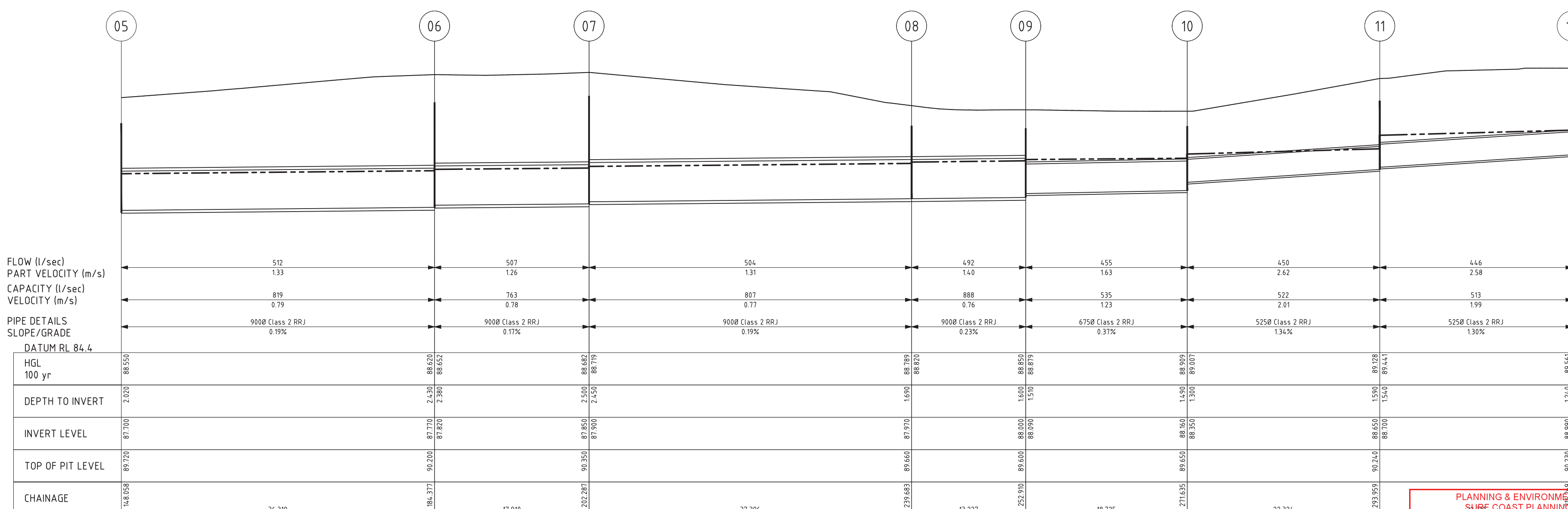
135 Austin Street, Winchelsea

PO BOX 350
TORQUAY, VICTORIA
INTERNET : www.surfcoast.vic.gov.au



DRAINAGE LONGITUDINAL SECTION - 65 min FULL AREA CONTRIBUTION FROM PROPOSED SUBDIVISION

SCALES: HORIZONTAL 1:250 VERTICAL 1:50



DRAINAGE LONGITUDINAL SECTION - 65 min FULL AREA CONTRIBUTION FROM PROPOSED SUBDIVISION

SCALES: HORIZONTAL 1:250 VERTICAL 1:50

PLANNING & ENVIRONMENT ACT 1987
SURF COAST PLANNING SCHEME
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Approval Number: PG19/0086
Date: 4/08/2021 Sheet No: 55 of 56

Digitally Signed by the Responsible Authority
Karen Hose

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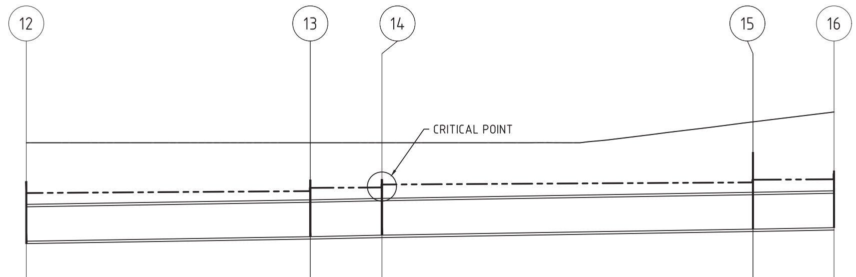
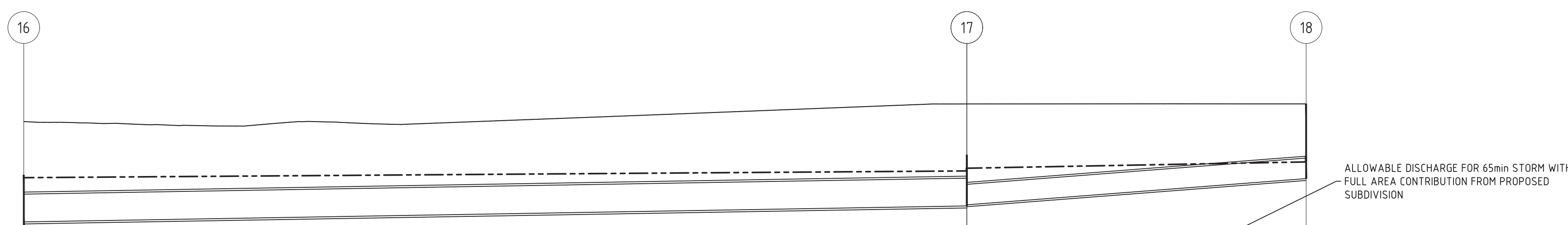


	12	13	14	15	16
HGL 100 yr	89.999	90.037 / 90.101	90.110 / 90.168	90.210 / 90.264	90.272
DEPTH TO INVERT	1.190	1.130	1.110	1.530	1.130
INVERT LEVEL	89.040	89.130	89.160	89.280	89.310
TOP OF PIT LEVEL	90.230	90.260	90.270	90.810	90.440
CHAINAGE	316.269	344.527	351.654	388.570	396.637

DATUM RL 85.8

FLOW (l/sec) / PART VELOCITY (m/s)	415 / 1.50	394 / 1.66	379 / 1.49	367 / 1.56
CAPACITY (l/sec) / VELOCITY (m/s)	493 / 1.13	567 / 1.07	498 / 1.03	533 / 0.99
PIPE DETAILS / SLOPE/GRADE	675Ø Class 2 RRJ / 0.32%	675Ø Class 2 RRJ / 0.42%	675Ø Class 2 RRJ / 0.33%	675Ø Class 2 RRJ / 0.37%
CHAINAGE	28.258	7.127	36.916	8.067

DRAINAGE LONGITUNDINAL SECTION – 65 min FULL AREA CONTRIBUTION FROM PROPOSED SUBDIVISION
SCALES: HORIZONTAL 1:250 VERTICAL 1:50



	16	17	18
HGL 100 yr	90.381	90.529 / 90.588	90.838
DEPTH TO INVERT	1.030	1.120 / 1.095	1.639
INVERT LEVEL	89.410	89.760 / 89.785	90.361
TOP OF PIT LEVEL	90.440	90.880	92.000
CHAINAGE	396.637	500.341	537.652

DATUM RL 86.1

FLOW (l/sec) / PART VELOCITY (m/s)	316 / 1.43	235 / 2.38
CAPACITY (l/sec) / VELOCITY (m/s)	373 / 1.08	354 / 1.48
PIPE DETAILS / SLOPE/GRADE	600Ø Class 2 RRJ / 0.34%	450Ø Class 2 RRJ / 1.54%
CHAINAGE	103.703	37.311

DRAINAGE LONGITUNDINAL SECTION – 65 min FULL AREA CONTRIBUTION FROM PROPOSED SUBDIVISION
SCALES: HORIZONTAL 1:250 VERTICAL 1:50