# ORMOND ST, BANNOCKBURN

## SITE STORMWATER MANAGEMENT PLAN


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**Client:** Land Owners at 5, 20, 25 and 30 Ormond St, Bannockburn
**Project:** Ormond St, Bannockburn
**Document:** Site Stormwater Management Plan

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## Project Details

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## Document Location

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# Executive Summary

TGM Group Pty Ltd has been engaged by Land Owners at 5, 20, 25 and 30 Ormond St, Bannockburn to develop a Site Stormwater Management Plan (SSMP) to support a planning scheme amendment application of the land at Ormond Street, Bannockburn.

The subject land currently comprises four (4) small rural holdings of approximately 17 hectares, located to the south west of the Township of Bannockburn.

The proposed development will result in an increase in impervious surfaces resulting in an increase in stormwater runoff volumes and pollutants.

Clause 56.07-04 of the Victoria Planning Provisions details the requirement for Residential sub-divisions to achieve the objectives of the State Environment Protection Policy (SEPP) as set out by the Urban Stormwater best practice environmental management guidelines (BPEMG) for stormwater quality.

The study shows that stormwater pollutants can be mitigated to meet 'best-practice' objectives with the adoption of a combination of a central wetland and two raingardens. The resulting stormwater contaminant removal efficiency can be seen in Table A.

Table A: Stormwater quality treatment efficiency

| Criteria | Reduction (%) | Objective (%) |
|---|---|---|
| Total Suspended Solids (kg/yr) | 80 | 80 |
| Total Phosphorus (kg/yr) | 62.1 | 45 |
| Total Nitrogen (kg/yr) | 45.8 | 45 |
| Gross Pollutants (kg/yr) | 100 | 70 |

Stormwater quantity is able to be mitigated to predeveloped flow rates with the adoption of a combination of a central detention basin (located above the wetland storage level) and isolated detention basins for the small catchments (Catchments S3 & S4) not able to be directed to the centrally located detention system. The resulting flows present at the sites legal point of discharge can be seen in Table B and Table C

Table B: Stormwater Discharge – Eastern Outlet

| AEP | Existing Conditions | | Developed Conditions | |
|---|---|---|---|---|
| | Critical Event Duration | Critical Peak Discharge ($m^3/s$) | Critical Event Duration | Critical Peak Discharge ($m^3/s$) |
| 1% | 1hour | 0.940 | 20min | 0.785 |
| 10% | 45min | 0.326 | 10min | 0.311 |
| 20% | 1.5hour | 0.247 | 10min | 0.250 |

Table C: Stormwater Discharge – Western Outlet

| AEP | Existing Conditions | | Developed Conditions | |
|---|---|---|---|---|
| | Critical Event Duration | Critical Peak Discharge ($m^3/s$) | Critical Event Duration | Critical Peak Discharge ($m^3/s$) |
| 1% | 3hour | 0.120 | 20min | 0.062 |
| 10% | 4.5hour | 0.047 | 25min | 0.037 |
| 20% | 4.5hour | 0.032 | 10min | 0.029 |

We note that the exact outflows achieved will be confirmed at the detailed design stage of the project.

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# 1 Introduction

TGM Group Pty Ltd has been engaged to develop a Site Stormwater Management Plan (SSMP) to support a Rezoning Application of land at 5, 20, 25, and 30 Ormond Street, Bannockburn, herein known as 'the site'.

The following report details the adopted analytical process and design outcomes for this study. A stormwater quality and a hydrological model were developed to predict the efficiency of the proposed treatment system in the reduction of contaminants and pollutants and to ensure that the site discharge can be maintained to predeveloped rates

The following report discusses the ability of the proposed development to discharge stormwater flows directly to Bruce Creek.

# 2 Stormwater Objectives

The objective of the following SSMP is to mitigate adverse impacts on stormwater discharges resulting from the development of the site. The SSMP will focus on stormwater discharge quality and quantity and the provision of mitigation facilities that will be designed to meet the conditions and requirements for stormwater management.

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.

The following stormwater management plan will provide details on the proposed stormwater treatment facilities 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

The site constitutes four (4) small rural holdings with a combined area of approximately 17 hectares. The site is located south west of the Township of Bannockburn.

The site is identified in the Bannockburn Urban Design Framework Plan as land set aside for 'Consolidated Residential' as adopted by the Golden Plains Shire. Therefore, strategic planning support has been be established to validate a Planning Scheme Amendment application to rezone this land for residential development in the short term.

The site is located on the west bank of Bruce Creek. Specifically, part of 25 Ormond St is subject to inundation caused by Bruce Creek flooding, as shown in Figure 3.2.

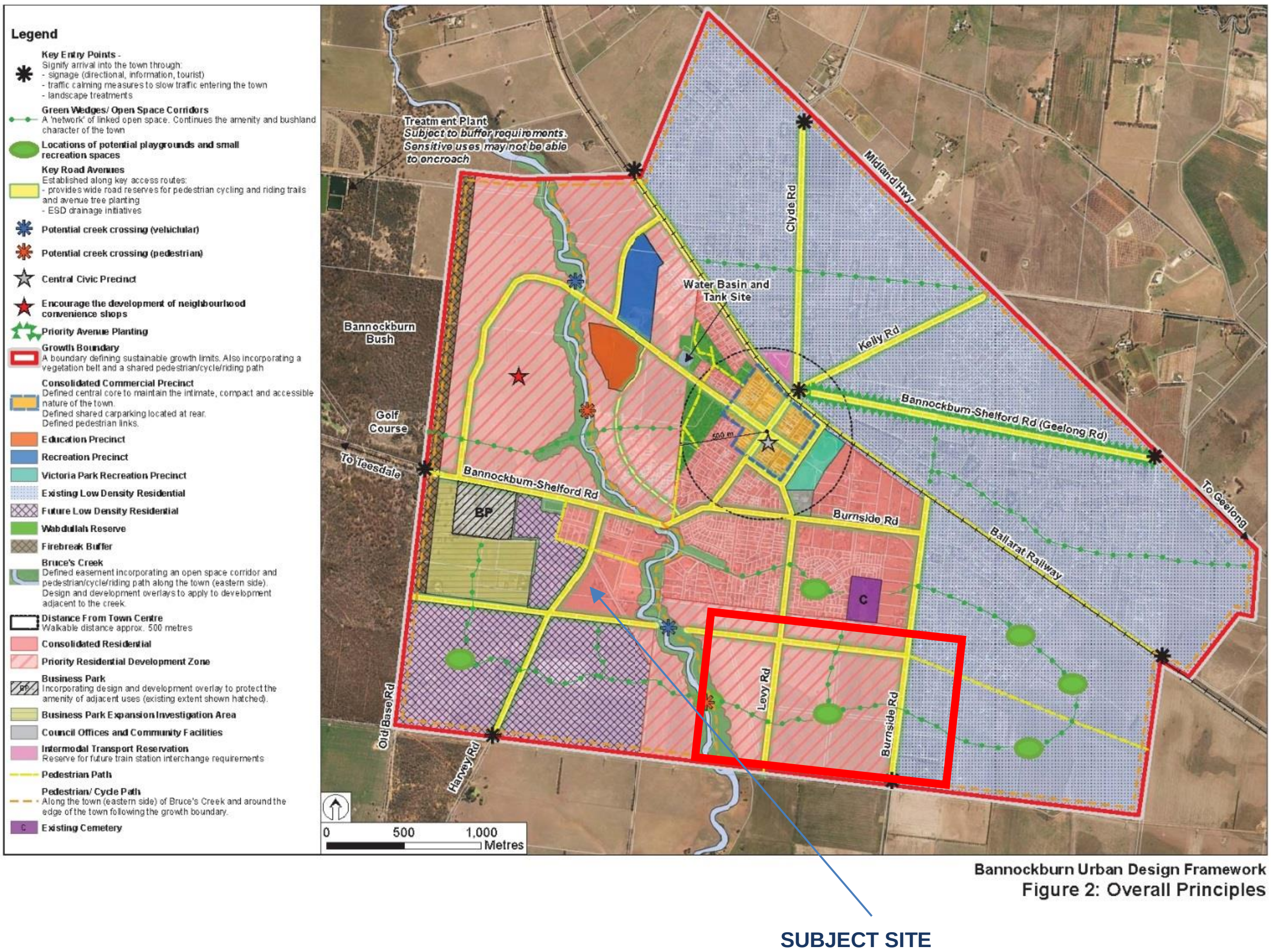



**Figure 3.1:** Bannockburn urban design framework, land use and activities

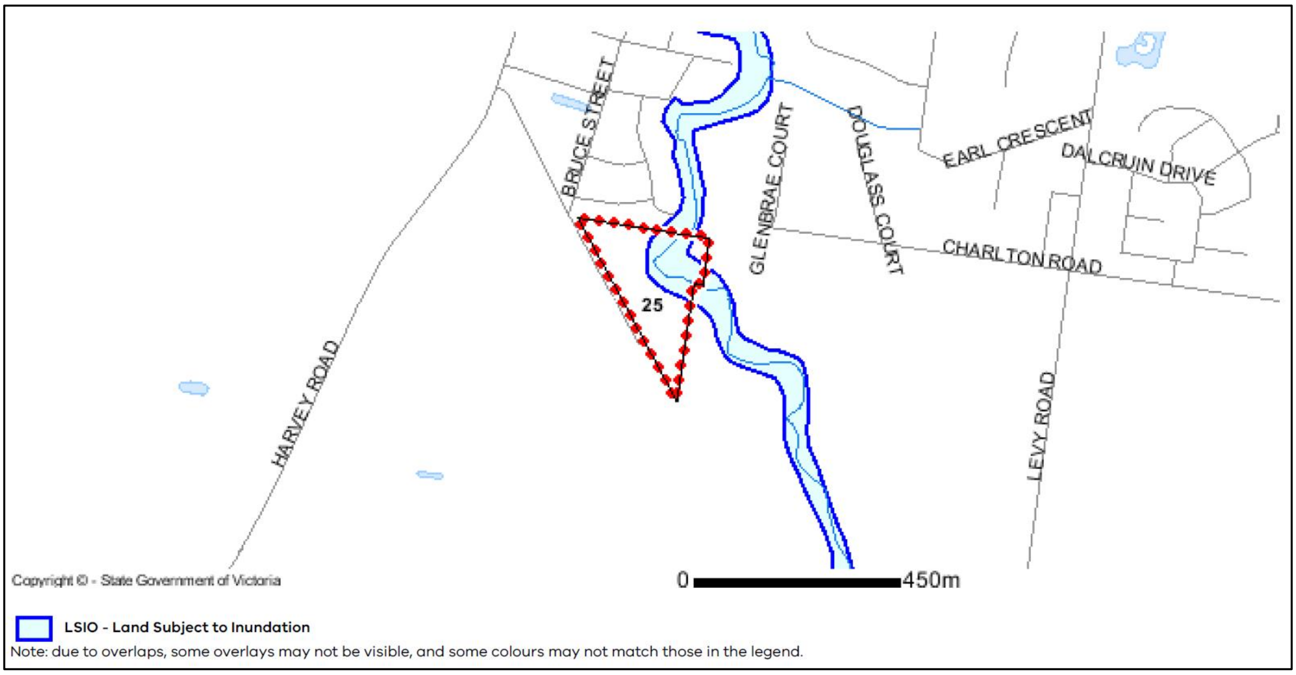



**Figure 3.2:** Land Subject to Inundation area[1]

The northern, western and central portion of the proposed development site exhibits flat slopes with no distinct topographical features. The central and eastern sections flow eastwards toward Bruce Creek with the eastern portion of the site, located on the western bank of Bruce Creek dropping steeply down towards the creek. The south western portion of the site generally quite flat although ultimately discharges west and into Harvey Street. The existing site and existing contours can be seen in Figure 3.3 below

[1] Planning Maps Online, http://services.land.vic.gov.au/landchannel/jsp/map/PlanningMapsIntro.jsp. Accessed on 7 Feb 2019.

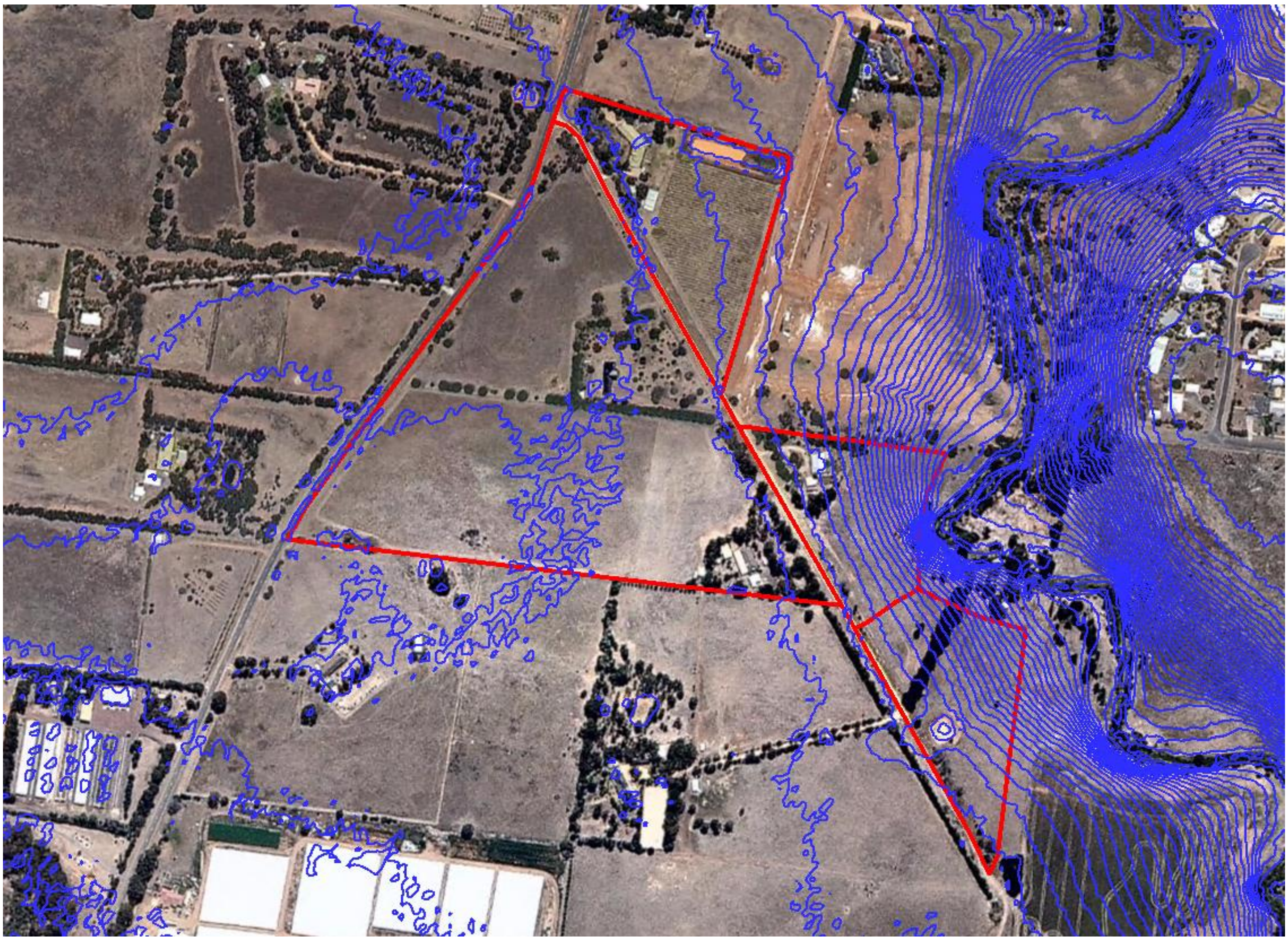


**Figure 3.3:** Existing Contours

## 3.2 Developed Site

The proposed development of the site will include construction of a General Residential (GRZ1) sub-division, as well as associated road network and reserve area.

The preliminary plan of sub-division, shown in Figure 3.4, proposes construction of approximately 170 lots. The proposed lots range in size from 431 m² to 2,168 m². There are three (3) super lots (3,990 m² – 6,137 m²) delineated around the existing residences to be retained.

The majority of lots are within the 590 m² to 1,000 m² range. Therefore, a fraction impervious of 70% was adopted for new lots. The larger lots around existing residences adopted a modelled fraction impervious based on actual area.

Fraction impervious was calculated based on Melbourne Water MUSIC Guideline[2], the initial plan (Figure 3.4), and the aerial imagery (nearmap[3]).

The proposed development results in a change of fraction impervious of the site from 4.7% under existing conditions to 58.6% under developed conditions.

Due to the layout of the site and the proximity to Bruce Creek a single end of line treatment system will not be practical. The below proposal analyses an integrated strategy including a centrally located wetland/detention facility servicing the majority of the development with additional rainwater tanks being provided on allotments that cannot drain directly to the centrally located system. It is proposed that the lots containing rainwater tanks will be subject to a restriction on title such as a section 173 agreement as indicated by catchments S2B, S3 & S4 as shown in Figure 3.4 and Section 6

Two bioretention systems are also proposed adjacent the Bruce Creek outfall to ensure flows that cannot be directed to the centrally located wetland system can undergo stormwater quality treatment ensuring best practice guidelines are met and maintained.

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

[3] Nearmap. https://www.nearmap.com.au/. Accessed on 9 May 2018.


| LAND BUDGET | |
|---|---|
| SITE AREA | 16.14ha |
| OPEN SPACE | 0.95ha |
| NET DEVELOPABLE AREA | 15.19ha |
| No. OF LOTS | 191 |
| LOT DENSITY | 12.6 LOTS/ha |

| Rev. | Revision | Date |
|---|---|---|
| - | - | - |

**PLAN OF PROPOSED SUBDIVISION**

5, 20, 25 & 30 ORMOND STREET
BANNOCKBURN, 3331

Job Number: 17170-100
Sheet: 1 of 1
Date of Survey: N/A
Date of Issue 08/11/2021
LENGTHS ARE IN METRES
12.50 0 12.50 25 37.50 50 62.50
1:1250
At Size A1
Survey: N/A Drawn: AD Checked: CM
DWG: 17170-100-PROPOSAL1 REV: 3

**Figure 3.4:** Plan of Prosposed Sub Division (TGM Group 2021)

## 3.3 Site Topography and Stormwater Conveyance

### 3.3.1 Topographical data

The Digital Elevation Model (DEM) used in this study was generated using LIDAR data from the *2003-04 National Action Plan for Salinity and Water Quality Corangamite LiDAR* dataset flown between 19 July 2003 and 10 August 2003.

The LIDAR data has a resolution of 0.6 m with a horizontal accuracy of ± 1.5 m and a vertical accuracy of ± 50 cm. The DEM was rendered using ESRI ArcGIS at a sampling resolution of 5 metres.

The DEM generated for the analysis is shown in Figure 3.5.

### 3.3.2 Bruce Creek

Bruce Creek is situated immediately east of the site. The Creek alignment dissects the north-east corner of 25 Ormond Street.

Bruce Creek originates approximately 16 km north of the site and has an upper catchment area in the vicinity of 30 to 40 km$^2$. Critical flooding within Bruce Creek is driven by longer duration rainfall events and from runoff generated within the upper catchment area.

The majority of the site is at an elevation greater than 100 m AHD before grading down to Bruce Creek.

The Bruce Creek alignment invert is at an elevation of approximately 70 m AHD through 25 Ormond Street.

Bruce Creek ultimately receives all stormwater runoff from the site.

### 3.3.3 Stormwater Conveyance

The site straddles a minor ridgeline creating two (2) site discharge direction, east and west. Ultimately, all stormwater runoff from the site enters Bruce Creek. The east catchment discharges to Bruce Creek directly, whereas, the smaller west catchment discharges through neighbouring properties and into the existing Harvey Street table drains before entering Bruce Creek approximately 3 km south of the site.

The difference in elevation between the east and west catchment is approximately 0.5 metres. It is expected that the developed surface levels will generally be graded out to allow conveyance of the majority of stormwater flows to Bruce Creek within the east catchment. A small amount of allotments along the Harvey Street frontage are proposed to continue directing flows west to maintain flow to downstream water uses. The exact arrangement will be confirmed through civil functional design at a later stage.

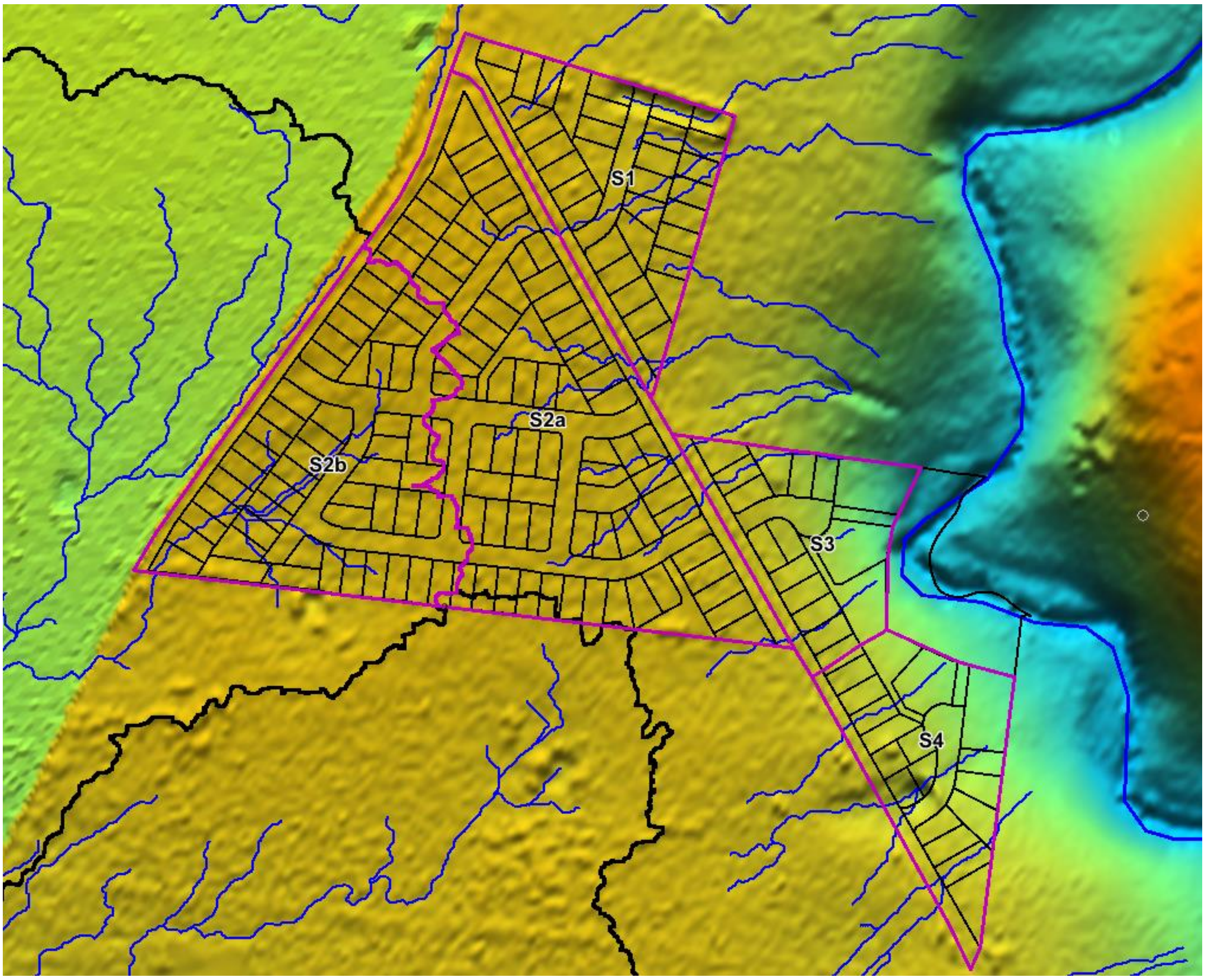



**Figure 3.5:** DEM and Stormwater flow path

# 4 Hydraulics and Hydrology

To ensure flows are maintained to predeveloped flowrates a 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 2019 (ARR2019) 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.

## 4.1 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 4-1.

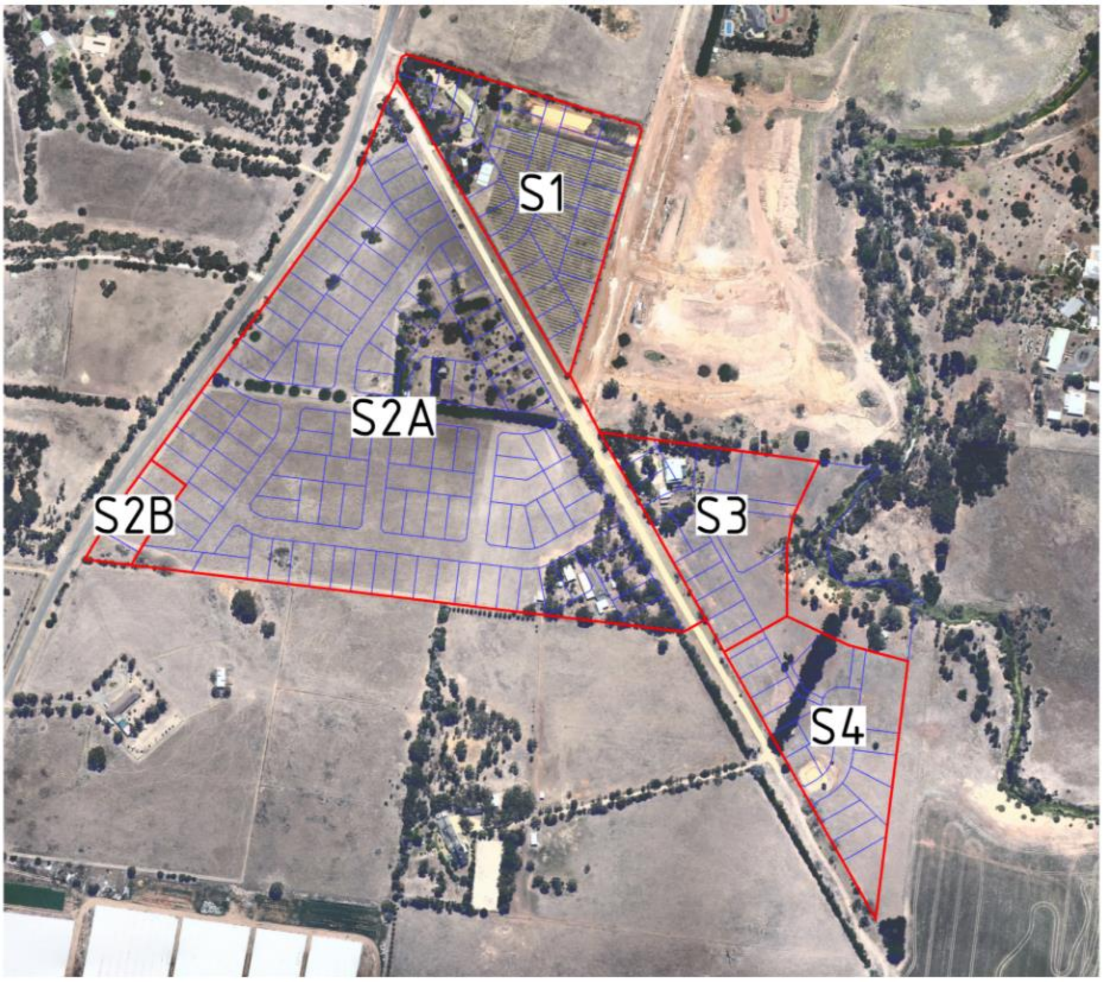



Figure 4-1 Overall Design Catchments

## 4.2 Model Parameters

### 4.2.1 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[4].

Table 4-1 Fraction Impervious

| Land Use | Impervious Surface (%) |
|---|---|
| Agricultural | 0.0 |
| Existing Buildings | 1 |
| Rural Road Reserve | 0.2 |
| Small Residential Lots (Under 1000m2) | 0.7 |
| Large Residential Lots (Typically 2000m2) | 0.3 |

Details of the existing conditions input values are shown in Table 4-2 below.

Table 4-2 Existing Catchment Parameters

| Catchment | Area (ha) | Fraction Impervious | Impervious Area (ha) | Slope (%) |
|---|---|---|---|---|
| S1 | 2.684 | 0.098 | 0.29 | 0.520 |
| S2A | 6.356 | 0.047 | 0.3 | 0.630 |
| S2B | 4.083 | 0 | 0 | 0.200 |
| S3 | 1.886 | 0.084 | 0.158 | 0.105 |
| S4 | 2.058 | 0.950 | 0.46 | 0.390 |

Details of the proposed conditions input values are shown in Table 4-3 below.

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

Table 4-3 Design Catchment Parameters

| Catchment | Area (ha) | Fraction Impervious | Impervious Area (ha) | Slope (%) |
|---|---|---|---|---|
| S1 | 2.684 | 0.55 | 1.193 | 0.62 |
| S2A | 10.038 | 0.625 | 6.278 | 0.40 |
| S2B | 0.401 | 0.7 | 0.281 | 0.19 |
| S3 | 1.886 | 0.468 | 0.883 | 10.53 |
| S4 | 2.058 | 0.518 | 1.066 | 6.57 |

Rainwater tanks are proposed to control peak discharge from the development site catchments where flows could not be directed to the central treatment facility. Catchments S2B, S3 and S4 were further delineated to take into consideration the portion of the roof area that would be intercepted by the rainwater tank and the area that would not. It was assumed that 70% of the roof area of each allotment would be directed to the rainwater tanks. Rainwater tanks of 2kL (2m3) and 5kL (5m3) were provided for the conventional lots (Area under 1000m2) and the larger lots (above 1000m2) respectively. An average roof area of 300m2 and 600m2 has been used for the small and large lots respectively.

The catchment delineation for the rainwater tank assessment can be seen below in Table 4-4 below.

Table 4-4 Design Catchment Parameters

| Catchment | Area (ha) | Lots per Catchments | Fraction Impervious | Impervious Area (ha) | Pervious Area (ha) |
|---|---|---|---|---|---|
| S2B – Roof | 0.106 | 5 | 1 | 0.106 | 0 |
| S2B - Balance | 0.296 | 0 | 0.7 | 0.157 | 0.139 |
| S3 – Roof (Small Lots) | 0.274 | 13 | 1 | 0.274 | 0 |
| S3 – Roof (Large Lots) | 0.127 | 3 | 1 | 0.127 | 0 |
| S3 - Balance | 1.505 | 0 | 0.43* | 0.648 | 0.857 |
| S4 – Roof (Small Lots) | 0.317 | 15 | 1 | 0.317 | 0 |
| S4– Roof (Large Lots) | 0.127 | 3 | 1 | 0.127 | 0 |
| S4 - Balance | 1.591 | 0 | 0.45* | 0.729 | 0.862 |

*Weighted average of large and small lot impervious areas

### 4.2.2 Loss Parameters

XP-STORM was run as an Initial Loss (IL) and Continuing Loss (CL) model using parameters provided from the ARR Data Hub[5].

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

Table 4-5 Adopted Hydrological Loss Parameters

| Surface | Storm Initial Loss (mm) | Pre-Burst Depth (mm) | Adopted Losses | |
|---|---|---|---|---|
| | | | Burst Initial Loss (mm) | Continuing Loss (mm/hr) |
| Pervious | 13.00 | 1.50 | 11.50 | 2.3 |
| Impervious | 0 | | 0 | 0 |

### 4.2.3 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 4-6.

Table 4-6 Manning's Coefficients 'n' Adopted in the Hydrological Model.

| Surface | Manning's Coefficients 'n' |
|---|---|
| Pervious | 0.038 |
| Impervious | 0.018 |

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

### 4.2.4 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 of the larger catchments S1 and S2 were initially considered to be undeveloped (pre-development) to allow comparison.

[5] ARR Data Hub, http://data.arr-software.org/ .
[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.

#### 4.2.5 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 140 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 140 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.

#### 4.2.6 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 4-7 and shown in Figure 4-2.

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.

[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 4-7 Manning's 'n' and Peak Discharges

| Event AEP (%) | Area (ha) | Manning's 'n' Adopted | RFFE Discharge ($m^3/s$) | XP STORM Discharge ($m^3/s$) |
|---|---|---|---|---|
| 50 | 13.1 | 0.035 | 0.07 | 0.05 |
| 20 | | | 0.13 | 0.12 |
| 10 | | | 0.18 | 0.17 |
| 5 | | | 0.24 | 0.24 |
| 2 | | | 0.33 | 0.36 |
| 1 | | | 0.41 | 0.45 |

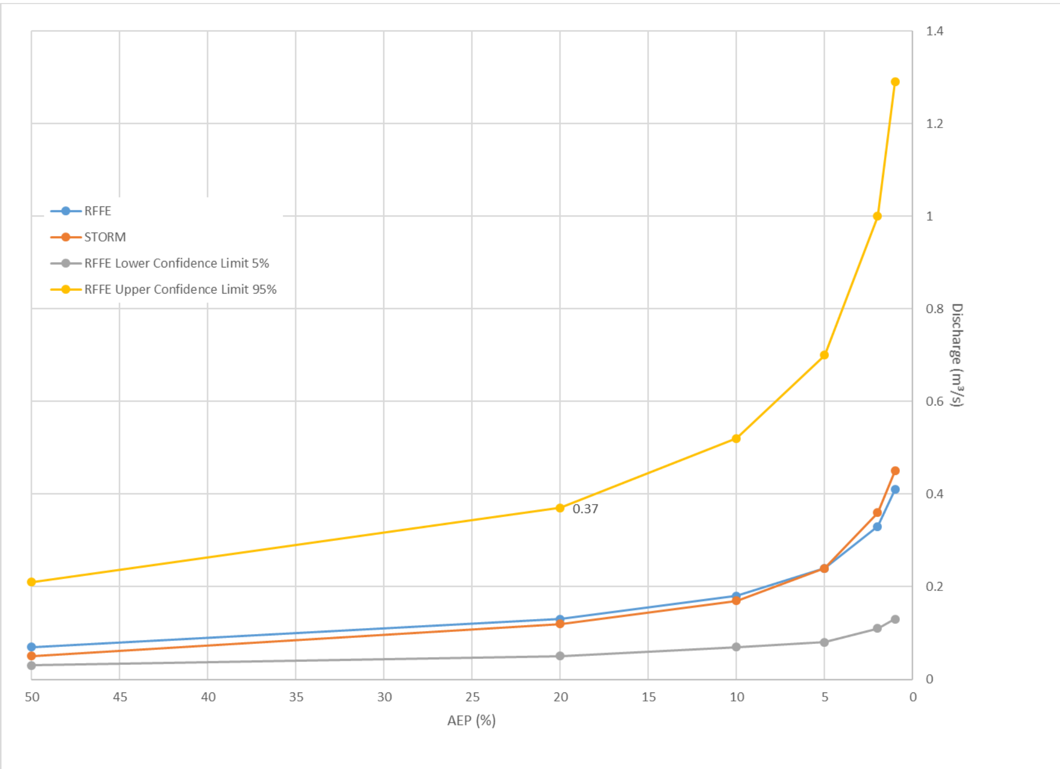


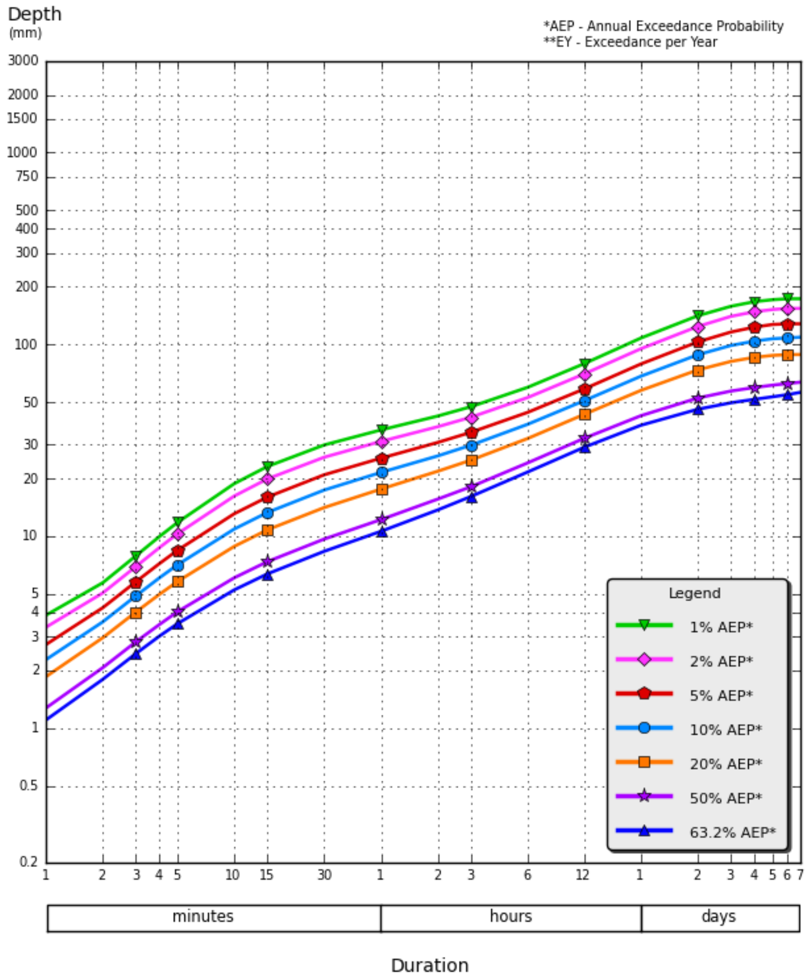

Figure 4-2 Estimated Peak Discharge – XP-STORM vs RFFE

## 4.3 Temporal Pattern Selection

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.

### 4.3.1 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) website11. The IFD curves are shown in **Error! Reference s ource not found.**.






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.

11 Bureau of Meteorology, http://www.bom.gov.ua/water/designRainfalls/.

Figure 4-3 2016 IFD Curves – Bureau of Meteorology 2019

### 4.3.2 Critical Storm Burst Pattern Selection

For this analysis, 10 storm burst temporal patterns were extracted for events up to 24-hour duration, for each AEP event. By analysing the hydrological response to the ensemble temporal patterns, one critical pattern was selected for each of the 16 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 4-8.

Table 4-8 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 | 1080 | | |
| | 90 | 1440 | | |

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 duration up to of the 6-hour durations. The procedure has been then repeated for each of event probability.

## 4.4 Hydrological Model Simulations

Sensitivity analysis models applied 100% pervious surfaces within the catchments. Impervious surfaces and urban characteristics were subsequently 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 4-9.

Table 4-9 Adopted Storm Burst Patterns.

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

# 5 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 and quantity 'best practice' standards and to calculate the requirements of site detention basins, rainwater tanks and constructed wetlands within the developed catchment.

## 5.1 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 eastern and western catchments is shown in Figure 5-1 and Figure 5-2 respectively. The critical peak discharges for the 1%, 10% and 20% AEPs for the eastern and western catchments have been tabulated in Table 5-1 and Table 5-2 respectively.

Table 5-1 Validated Peak Discharges – Eastern Combined Catchment

| AEP | Critical Event Duration | Critical Peak Discharge ($m^3/s$) |
|---|---|---|
| 1% | 1hour | 0.940 |
| 10% | 45min | 0.326 |
| 20% | 1.5hour | 0.247 |

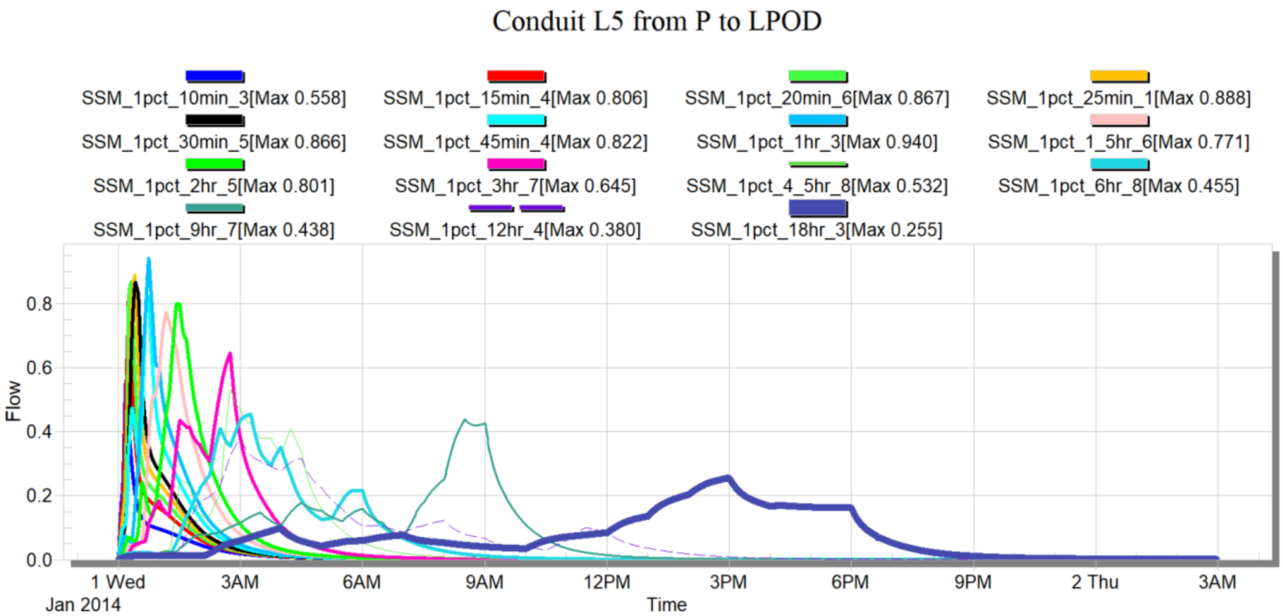



Figure 5-1 1% AEP Hydrographs – Eastern Outlet

Table 5-2 Validated Peak Discharges – Western Catchment (S2B)

| AEP | Critical Event Duration | Critical Peak Discharge ($m^3/s$) |
|---|---|---|
| 1% | 3hour | 0.120 |
| 10% | 4.5hour | 0.047 |
| 20% | 4.5hour | 0.032 |

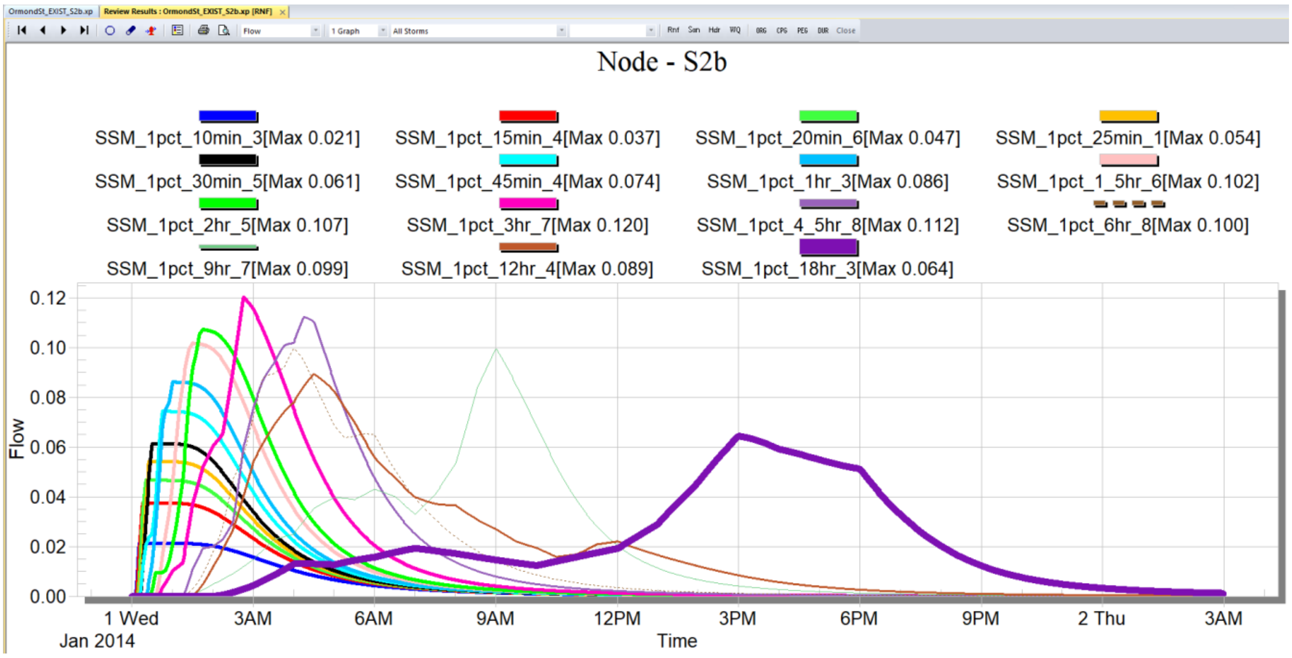



Figure 5-2 1% AEP Hydrographs – Western Outlet (S2B)

## 5.2 Developed Site Characteristics.

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

Catchments S1 and S2A will be directed to a centrally located treatment facility, whereas catchment S2B will make use exclusively of rainwater tanks to control peak flows from the site. Catchment S3 and S4 will control the peak outflows via the use of rainwater tanks in addition to small bioretention systems (inclusive of sediment forebay and gross pollutant trap) to ensure best practice targets are achieved.

The opportunity to direct all catchment flows to the centrally located system was investigated however due to existing hydrological and topographic conditions a dispersed approach need to be adopted. The above proposal was simulated with XP-STORM and the resulting stormwater hydrographs were then evaluated to ensure compliance.

### 5.2.1 Centralised Detention Storage

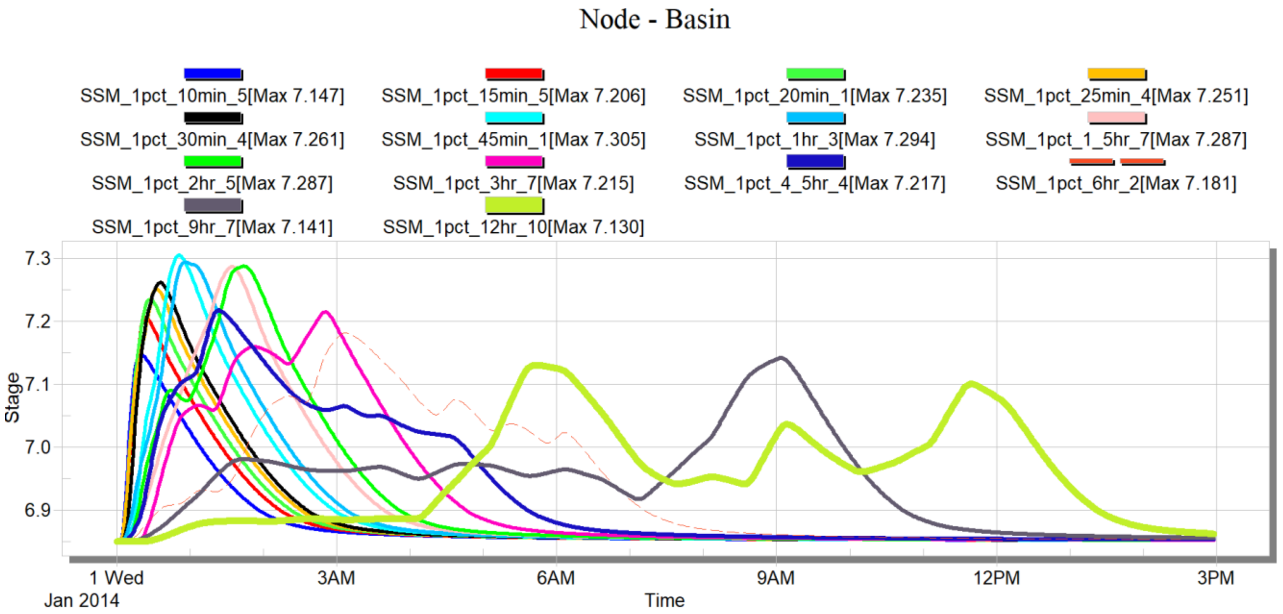
Due to the topography of the subject site, where practical, the proposed development site is directed to the centrally located stormwater treatment facility. The facility consists of a Sedimentation Basin and Wetland system and above these quality treatment measures will be a detention system designed to mitigate flows. The facility is proposed to discharge to an underground drainage network that will subsequently connect into the Bruce Creek drainage outfall. It should be noted that the achieved detention basin peak flow rates are below the locale catchment PSD due to the inability to direct the entire development site to the system.

The peak discharges from the centrally located basin are summarised in Table 5-3

Table 5-3 Developed Peak Discharge – Detention Basin Outflows

| AEP | Critical Event Duration | Critical Peak Discharge ($m^3/s$) |
|---|---|---|
| 1% | 3 hours | 0.547 |
| 10% | 9 hours | 0.204 |
| 20% | 9 hours | 0.144 |

Figure 5-3 Developed 1% AEP Hydrograph




Key parameters of the Detention basin retarding flows are shown in Table 5-4 below.

Table 5-4 Stormwater Treatment Basin Details Summary

| | 1%AEP | 10% AEP | 20%AEP |
|---|---|---|---|
| Peak Outflow ($m^3/s$) | 0.547 | 0.204 | 0.144 |
| Critical Duration (min) | 180 | 540 | 540 |
| Top Water Level | 100.775 | 100.686 | 100.666 |
| Basin Volume ($m^3$) (Top Water Level to Top Extended Detention | 3203 | 2766 | 2667 |
| Orifice IL (m AHD) | - | 100.6 | 100.05 |
| Outlet Configuration (modelled) | - | 900mm x 1200mm Grated Pit | 100mm x 100mm Side Orifice in Outlet Pit |

### 5.2.2 Rainwater Tanks

As previously discussed, due to site topography preventing total site flows being directed to the centrally located detention system, rainwater tanks were investigated for the purpose of detention of stormwater flows prior to discharge into the downstream drainage network.

It was assumed that 70% of the roof area of each allotment would be directed to the rainwater tanks. Rainwater tanks of 2kL (2m3) and 5kL (5m3) were provided for the conventional lots (Area under 1000m2) and the larger lots (above 1000m2) respectively. An average roof area of 300m2 and 600m2 has been used for the small and large lots respectively.

Flows not intercepted by the rainwater tanks were modelled as discharging directly to the site outfall.

Key parameters of the rainwater tanks shown in Table 5-5 below

Table 5-5 Stormwater Treatment Basin Details Summary

| Catchment | Lots per Catchments | Tank Size (Per Lot) | Outlet (Low flow) | Outlet (High Flow) | Outflow Per Cluster (20% AEP) | Outflow Per Cluster (1% AEP) |
|---|---|---|---|---|---|---|
| S2B | 5 | 2kL | $0.002m^2$ @ Invert | $0.07m^2$ @ Obvert | 0.008 $m^3/s$ | 0.018 $m^3/s$ |
| S3 (Small Lots) | 13 | 2kL | $0.0025m^2$ @ Invert | $0.071m^2$ @ Obvert | 0.012 $m^3/s$ | 0.066 $m^3/s$ |
| S3 (Large Lots) | 3 | 5kL | $0.003m^2$ @ Invert | $0.018m^2$ @ Obvert | 0.01 $m^3/s$ | 0.026 $m^3/s$ |
| S4 | 15 | 2kL | $0.0025m^2$ | $0.071m^2$ | 0.013 $m^3/s$ | 0.076 $m^3/s$ |

| | | | | | | |
|---|---|---|---|---|---|---|
| (Small Lots) | | | @ Invert | @ Obvert | | |
| S4 (Large Lots) | 3 | 5kL | $0.003m^2$ @ Invert | $0.018m^2$ @ Obvert | 0.029 $m^3/s$ | 0.01 $m^3/s$ |

Rainwater Tanks were initially investigated, however following discussions with Council this option was not considered feasible.

### 5.2.3 S3 & S4 Catchment Detention Storage

As an alternative to Rainwater tanks, detention basins were investigated for the purpose of detaining stormwater flows prior to discharging into Bruce Creek.

Due to the topography of the site, both Catchment S3 and Catchment S4 will require a separate detention basin. The detention facilities consist of Bioretention systems for quality treatment measures combined with a detention system designed to mitigate flows.

For Catchment S3, further consideration will be required at detailed design to ensure that the detention facility remains outside the LSIO overlay. It is considered that this will be achieved with retaining walls greater than 1.5m. The water treatment and detention facility will also absorb on a lot, which is shown in Figure 6-2.

Similarly of Catchment 4, the detention and treatment facility will be located within the proposed drainage reserve, however based on current concepts it is not expected to impede on the LSIO overlay.

Key parameters for the Detention basins are shown in Table 5-6 below.

Table 5-6 Stormwater Treatment Basin Details Summary

| Catchment | Basin Volume ($m^3$) | Average Depth (m) |
|---|---|---|
| S3 | 210 | 0.5 |
| S4 | 230 | 0.5 |

Exact sizing of stormwater detention noted throughout will be optimised through the detailed design phase. Inlet and outlet configurations will also be confirmed through the detailed design phase

### 5.2.4 Overall Site Discharge

Flows from the development site will discharge into the surrounding catchment at two locations. The western site will discharge into Harvey Street whilst the eastern areas will be conveyed to a single location for controlled discharge to Bruce Creek. The below Table 5-7 and Table 5-8 summarises proposed discharges at the legal point of discharge for the eastern and western catchments respectively.

Table 5-7 Eastern Developed Peak Discharge – Bruce Creek

| AEP | Existing Conditions | | Developed Conditions | |
|---|---|---|---|---|
| | Critical Event Duration | Critical Peak Discharge ($m^3/s$) | Critical Event Duration | Critical Peak Discharge ($m^3/s$) |
| 1% | 1hour | 0.940 | 20min | 0.785 |
| 10% | 45min | 0.326 | 10min | 0.311 |
| 20% | 1.5hour | 0.247 | 10min | 0.250 |

Table 5-8 Western Developed Peak Discharge – Henry Street

| AEP | Existing Conditions | | Developed Conditions | |
|---|---|---|---|---|
| | Critical Event Duration | Critical Peak Discharge ($m^3/s$) | Critical Event Duration | Critical Peak Discharge ($m^3/s$) |
| 1% | 3hour | 0.120 | 20min | 0.062 |
| 10% | 4.5hour | 0.047 | 25min | 0.037 |
| 20% | 4.5hour | 0.032 | 10min | 0.029 |

Exact sizing of treatment measures noted throughout will be optimised through the detailed design phase.

## 5.3 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 Geelong North site from 1971 to 1980 using a 6-minute timestep which was sourced from COGG data.

An overall view of the MUSIC model is provided below in Figure 5-4.

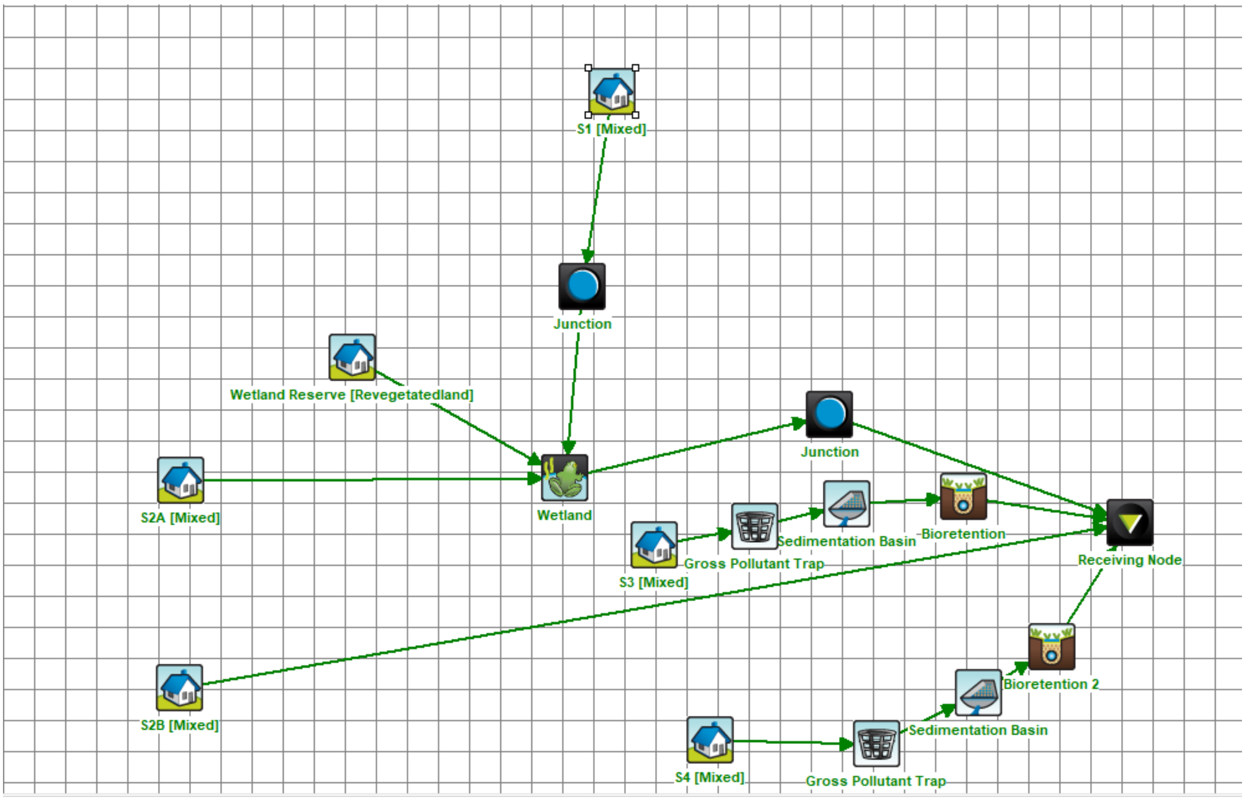



Figure 5-4 Overall MUSIC Model Network

The water quality treatment infrastructure consists of a conventional wetland/sedimentation basin centrally located to treat the majority of flows generated within the development site. Where site topography does not allow for flows to be directed to the centrally located system, additional treatment is provided via two bioretention systems at the boundary of catchments S3 and S4 developed area. The bioretention systems are proposed to be complimented by a Gross Pollutant Trap and sediment forebay to protect the filter media form the build-up of sediment.

In should be noted that no phosphorus or nitrogen removal has been factored into this modelling and that the sediment removal efficiency adopted within this model was the lowest of the values provided in published papers by SPEL[12], Rocla[13], Humes[14] and Urban Asset Solutions[15] of 49% removal of suspended solids.

The rainwater tanks previously discussed in section 5.2.2 have not been considered in the water quality portion of the assessment.

[12] SPEL Vortceptor Hydrodynamic GPT 'Technical Data Sheet'
[13] Rocla CDS Unit Technical Summary
[14] Humes HumeGard GPT Technical Manual
[15] EcoSol Gross Pollutant Trap Technical Specifications

### 5.3.2 Lot Based Inputs

The characteristics of the catchment modelled in MUSIC are detailed in Table 5-9. These characteristics have been compiled using City of Greater Geelong Design Notes No.3.

Table 5-9 MUSIC Simulations – Lot/Road Properties.

| Catchment Characteristics | |
|---|---|
| Zoning/Surface Type | Mixed |
| **Impervious Area Properties** | |
| Rainfall Threshold (mm/day) | 1.00 |
| **Pervious Area Properties** | |
| Soil Storage Capacity (mm) | 120 |
| Initial Storage (% of Capacity) | 25 |
| Field Capacity (mm) | 50 |
| 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 |

### 5.3.3 Treatment Inputs

The characteristics of the end of line treatment measures modelled in MUSIC are detailed in Table 5-10, Table 5-11 and Table 5-12. 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.

Table 5-10 Central Sedimentation Basin / Wetland – MUSIC Inputs

| Inlet Properties | |
|---|---|
| Low Flow By-pass ($m^3/s$) | 0.0 |
| High Flow By-pass ($m^3/s$) | 100.0 |
| Inlet Pond ($m^3$) | 460 |
| **Storage Properties** | |
| Surface Area ($m^2$) | 2200 |
| Extended Detention Depth (m) | 0.35 |
| Permanent Pool Volume ($m^3$) | 500 |
| Initial Volume ($m^3$) | 500 |
| Exfiltration Rate (mm/hr) | 0 |
| Evaporative Loss as % of PET | 125 |
| **Outlet Properties** | |
| Equivalent Pipe Diameter (mm) | 47 |
| Overflow Weir Width (m) | 10 |
| Notional Detention Time (hrs) | 70.2 |

Table 5-11 S3 and S4 End of line Sedimentation Basin – MUSIC Inputs

| Inlet Properties | | |
|---|---|---|
| | S3 | S4 |
| Low Flow By-pass ($m^3/s$) | 0.0 | 0.0 |
| High Flow By-pass ($m^3/s$) | 0.200 | 0.200 |
| **Storage Properties** | | |
| Surface Area ($m^2$) | 40 | 40 |
| Extended Detention Depth (m) | 0.20 | 0.20 |
| Permanent Pool Volume ($m^3$) | 2 | 2 |
| Initial Volume ($m^3$) | 2 | 2 |

| | | |
|---|---|---|
| Exfiltration Rate (mm/hr) | 0 | 0 |
| Evaporative Loss as % of PET | 75 | 75 |
| Outlet Properties | | |
| Equivalent Pipe Diameter (mm) | 100 | 100 |
| Overflow Weir Width (m) | 2 | 2 |
| Notional Detention Time (hrs) | 0.213 | 0.213 |

Table 5-12 S3 and S4 Bioretention Basin – MUSIC Inputs

| Inlet Properties | | |
|---|---|---|
| | S3 | S4 |
| Low Flow By-pass ($m^3/s$) | 0.0 | 0.0 |
| High Flow By-pass ($m^3/s$) | 0.200 | 0.200 |
| Storage Properties | | |
| Extended Detention Depth (m) | 0.30 | 0.30 |
| Surface Area ($m^2$) | 28 | 28 |
| Filter and Media Properties | | |
| Filter Area ($m^2$) | 2 | 2 |
| Unlined Filter Media Perimeter (m) | 24 | 24 |
| Filter Depth (m) | 0.5 | 0.5 |

### 5.3.4 End of Lines Efficiencies

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

Table 5-13 Stormwater Quality Treatment Efficiencies

| Criteria | Reduction (%) | |
|---|---|---|
| | Results | Target |
| Total Suspended Solids (kg/yr) | 80.0 | 80 |
| Total Phosphorus (kg/yr) | 65.7 | 45 |
| Total Nitrogen (kg/yr) | 46.4 | 45 |
| Gross Pollutants (kg/yr) | 100 | 70 |

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

# 6 Functional Design Outcomes and Limitations

To ensure the site can facilitate the treatment measures proposed above a conceptual design was undertaken to ensure flows could be directed to the proposed treatment facility, discharged to the relevant legal point of discharge and that expected infrastructure could be accommodated within the proposed site layout.

Site investigations confirmed the steep grades within the eastern portion of the development site indicated by the lidar survey used throughout this investigation. Due to the steep grades that exist earthworks batters have been shown to account for a large amount of land area however at max batter slopes of 1V:4H are still able to be integrated into the existing surface.

As per Council's comment, the detention facilities for Catchments S3 & S4 are required to be outside the LSIO overlay. Under these limitations, for Catchment S3, the basin will be located within the proposed developable lot. The current lot layout will need to be reconfigured. For Catchment S4, the detention facility will be located within the drainage reserve. Retaining walls up to 1m will be required, inclusive of further battering to facilitate the detention basin footprint. Exact location and sizing of the detention facilities will be optimised during the detailed design phase.

Determination of the treatment configuration, inclusive of a Gross Pollutant Trap and sediment forebay, for the most effective removal of suspended solids will be analysed during the detailed design phase.

The conceptual design of the major central treatment facility has allowed for the provision of a maintenance track around the perimeter of the facility, however it would be expected that this will be reduced during the detail design based on the adopted Melbourne Water guidelines and the provision of access from the surrounding road network. It was determined that the additional maintenance track and other detail provided at this concept stage will account for additional design stage requirements such as sediment drying areas, centre bunds and general changes to the site layout.

Table 6-1 and Table 6-2 below provides an indication of the required areas required to achieve the modelling outcomes provided within the SMP and is reflective of the footprints shown in Figure 6-2.

Table 6-1 Functional Details Summary – Central Wetland

| | 1%AEP |
|---|---|
| Area at Permeant Pool | 2896m2 |
| Area at Extended Detention | 3500m2 |
| Area at 1% AEP Level | 4987m2 |
| Area at 150mm Free Board (Above 1% AEP | 5231m2 |
| Area at extent of batter (including maintenance access tracks) | 5600m2 |

Table 6-2 Functional Details Summary – Sediment Forebay and Bioretention System

| | S3 | S4 |
|---|---|---|
| Area at Invert | 70m2 | 70m2 |
| Area at Extended Detention | 100m2 | 40m2 |
| Area including cut/fill batters | 215m2 | 120m2 |

An overall development plan indicating the relative location and size of the various treatment infrastructure is shown in Figure 6-3. It should be noted that the exact location and size of the various infrastructure will be subject to detailed design. Lot layout to be corrected.

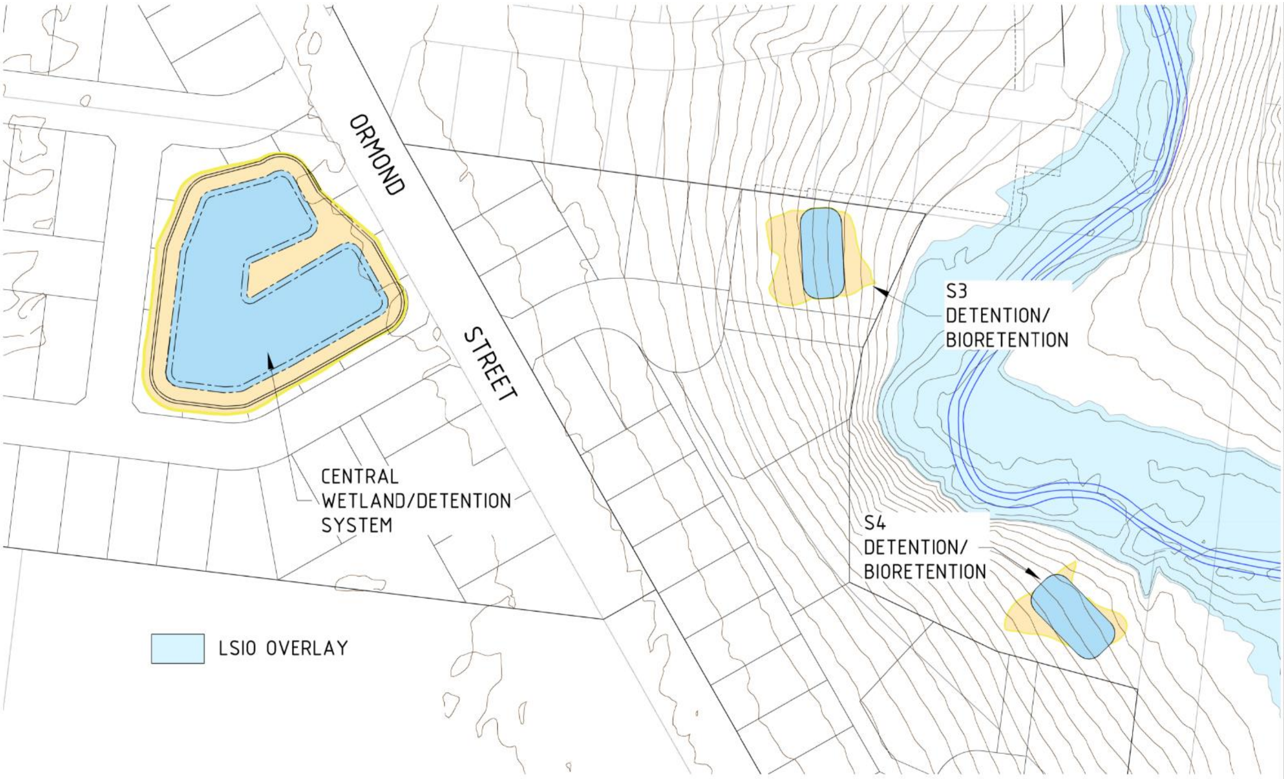



Figure 6-2 Conceptual Treatment Facility Layout

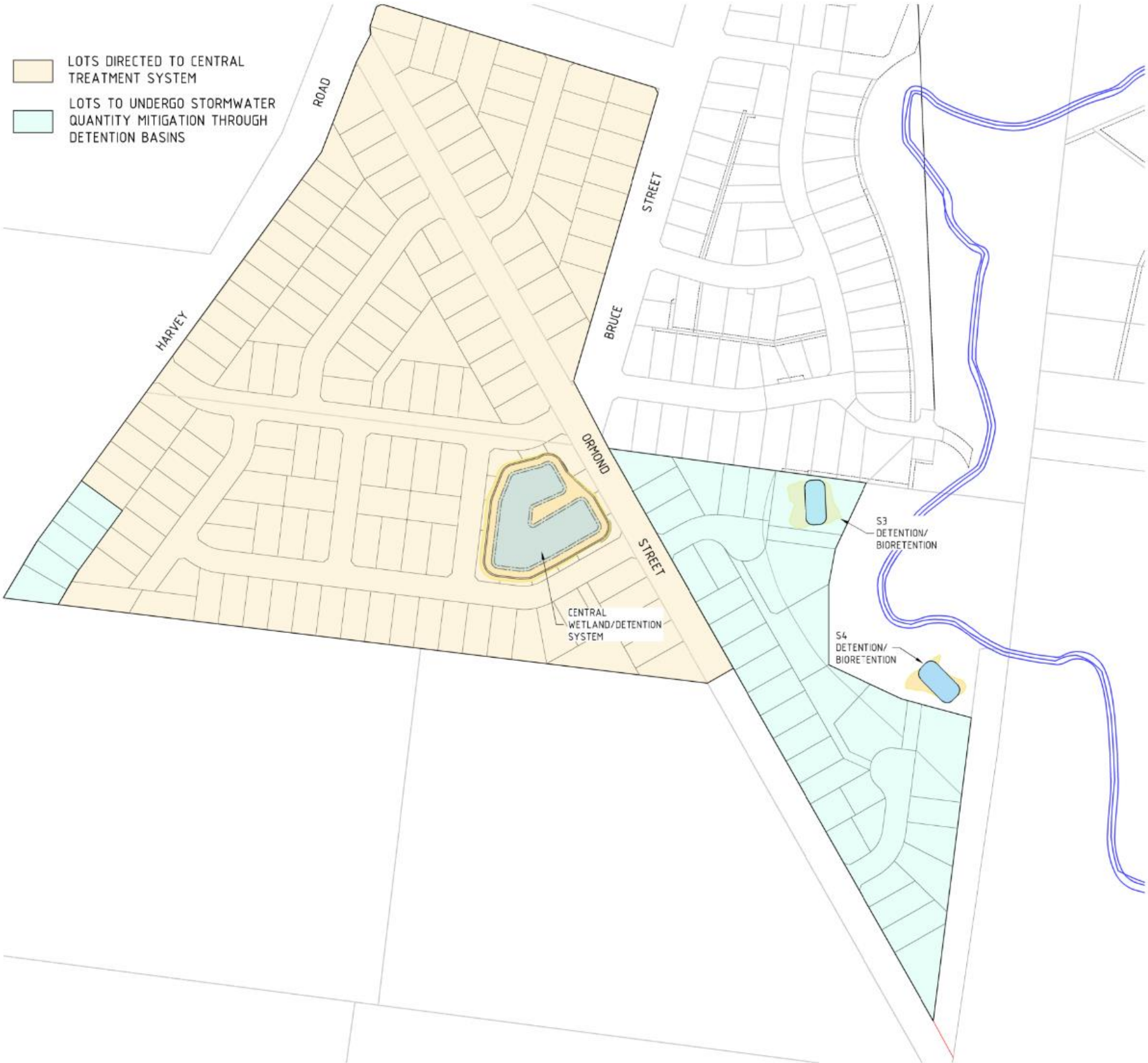



Figure 6-3 Overall Site Layout with treatment mechanisms

# 7 Conclusion

TGM recommends adoption of a wetland and raingarden treatment system to ensure the proposed development at 5, 20, 25, and 30 Ormond Street, Bannockburn meets best practice requirements for stormwater quality. Stormwater detention facilities are proposed to limit site discharge rates to predeveloped rates.

The stormwater generated from 5, 20, and 30 Ormond Street is suggested to be conveyed into and treated by a designed wetland and detention basin at the southeast corner of 30 Ormond Street, and then conveyed directly to Bruce Creek through a conventional drainage system.

Stormwater quality treatment resulting from flows generated within 25 Ormond Street is suggested to be treated by two separate raingardens before discharging into Bruce Creek. Raingarden footprints were restricted due to the steep topography. However, the wetland was increased to compensate for the restricted treatment capacity of the raingardens.

Initially, rainwater tanks were investigated to provide detention for 25 Ormond Street. However, after meetings with Council, this was considered not to be the desired approach. As an alternative design, stormwater discharge will be maintained to predeveloped rates up to the 1% AEP through the use of two separate detention basins.

A small number of allotments fronting Harvey Street will be maintained inclusive of rainwater tanks. This will ensure stock dams maintain flow however the provision of rainwater tanks will ensure flows do not exceed predeveloped rates.