



# STORMWATER MANAGEMENT PLAN

ACEnergy Battery Farm  
438 Lobbs Road, Glenbrae, VIC

Prepared for ACEnergy Pty Ltd  
By Planit Consulting Pty Ltd

May 2023

J7690 | SWMP V03 - Final

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### Project Details

|                        |                            |
|------------------------|----------------------------|
| Project Name           | 438 Lobbs Road             |
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- Hydrological and Hydraulic modelling to estimate peak flow rates in the existing scenario and proposed scenario (DRAINS modelling);
- Provide recommendations for stormwater conveyance and options for a proposed detention system;
- Provide recommendations for the proposed treatment train;
- Provide details of opportunities for Water Sensitive Urban Design; and
- Assess and report on any impacts to the existing wetland downstream of the site.
- Providing conclusions/recommendations with regard to stormwater management of the site.

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## 2 Site Assessment

### 2.1 Site Description

The subject site (Figure 2) is located in Glenbrae, VIC, which forms part of the Pyrenees Shire Local Government Area. 438 Lobbs Road consists of a parcel of existing pastoral land of approximately 115Ha. It is proposed to install a battery farm that contains rows of Battery Energy Storage System (BESS) and Medium Voltage Power Station (MVPS) containers resembling ~40ft shipping container, which connect to a substation/control room. The proposed site has a total footprint of approximately 10.5 hectares and is to be located on the north-west corner of the site.

For the purposes of this report, the “subject site” refers to only the portion of land within 438 Lobbs Road that will be utilised for the development.

It is anticipated that project delivery will require a hardstand crushed rock area to be delivered for the BESS area. A hardstand area will also be required for the substation. The site boundary will consist of a vegetated buffer along the western, southern and northern boundaries with access being provided via Forest Road to the north. Any stormwater requirements associated with further expansion will be assessed as part of the future approvals process.

Vehicular access/exit for the proposed site will be via Forest Road which abuts the northern boundary of the site. The access road will not have an impact of stormwater conveyance conditions.

The paddock and accessway have historically been cropped for agricultural purpose. No geotechnical investigations or site visits were completed in the preparation of this report.

There is an existing wetland located approximately 120m west of the Forest Road / Lobbs Road intersection. The wetland is approximately 400m in diameter and is split in half by Forest Road. It is unclear if the wetland is balanced by a culvert placed beneath Forest Road or whether it topples over the Forest Road crown when inundated.

It is expected that catchments to the south-east of this wetland (including the subject site) drain to and pass through this wetland via sheet flow and concentrated overland channels within the road reserves. Based on site photos and publicly available aerial imagery, the wetland appears to be shallow and consists of sparsely positioned low lying vegetation with a small cluster of trees either side of Forest Road.

There is a natural watercourse that runs through the south-west corner of 438 Lobbs road which travels east to west. The watercourse is approximately 600mm south of the proposed development area and will have no impact upon the proposed works. Upon crossing through 438 Lobbs Road, the watercourse continues to the south-east and does not feed the existing wetland.

The site is located within an Environmental Sensitivity Overlay (ESO1) and will require assessment to ensure the wetland will not be adversely impacted by the project.

Figure 2 provides a summary of existing site conditions.

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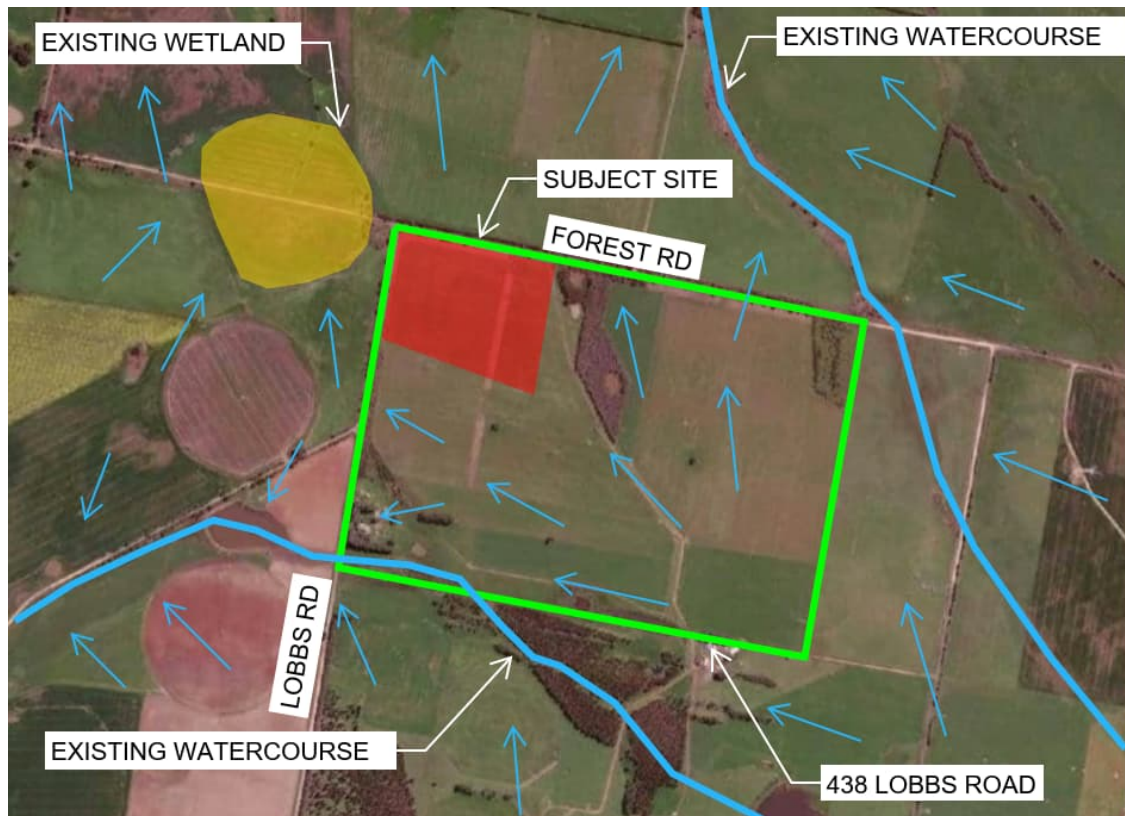


Figure 2 – Summary of Site Conditions

To confirm the location of existing services, a “Dial Before You Dig” (DBYD) search was requested for the vicinity of the development area. There are no existing assets located within Lobbs Road and Forest Road as indicated on the DBYD plans. The only existing assets within the region are the Ausnet Transmission Line, Nextgen Cable and Powercor Transmission lines that run east-west through the south-west corner of 438 Lobbs Road.

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## 2.2 Site Topography

The site has a gentle 1% - 1.5% fall over the surface running south-east to north-west. The subject site itself is largely void of any significant topographical features. The highest point of the subject site that is intended to be developed sits at approximately 430.4m AHD with the lowest point sitting at 422.2m AHD.

No feature survey was provided as part of this project. Publicly available contour information 'Vicmap Elevation DEM 10m' was utilised within the assessment and is provided in Figure 3.

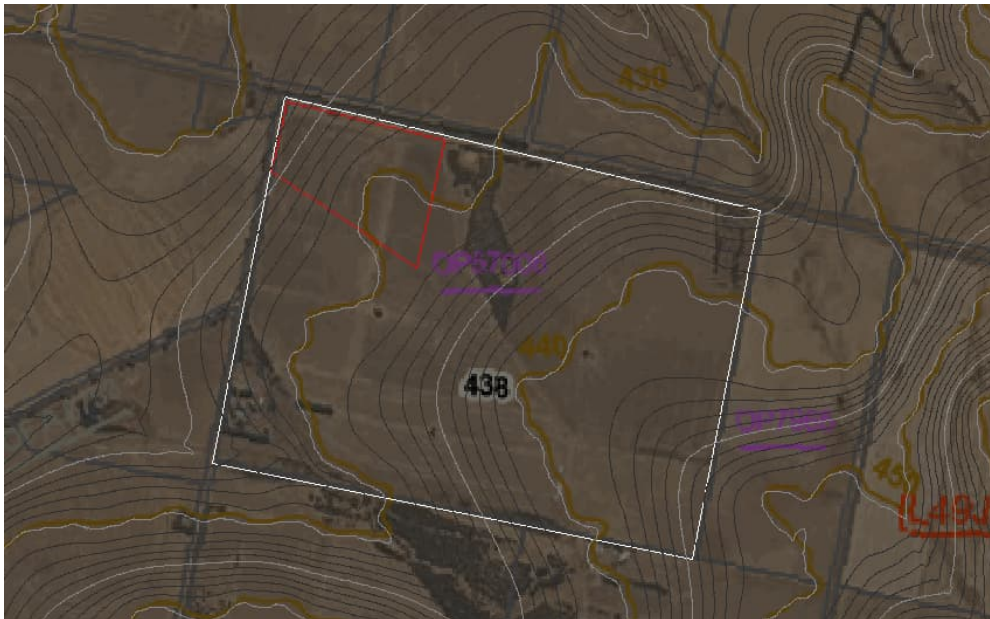


Figure 3 – Publically available Contours

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## 3 Catchment Analysis

### 3.1 Existing Conditions

#### 3.1.1 Existing Conditions

The subject site does not contain any stormwater infrastructure; all stormwater conveyance discharges as sheet flow across the site. The flows are intercepted by existing open drains within the road reserves of Lobbs and Forest Road. The flows within these channels continue onwards to the west where they are captured by the existing downstream wetland.

A weakly defined crest runs through 438 Lobbs Road from the south-east to the north-west which splits the flows within and external to the site to the north and south respectively. There are two defined waterways located within the vicinity of the subject site (defined in Figure 4). The southern watercourse runs through the south-west corner of 438 Lobbs Road and continues to the south. The northern watercourse abuts the tip of the north-west corner of 438 Lobbs Road prior to continuing to the north-west. Neither of these watercourses feed stormwater runoff through the developable portion of the site or to the existing wetland.

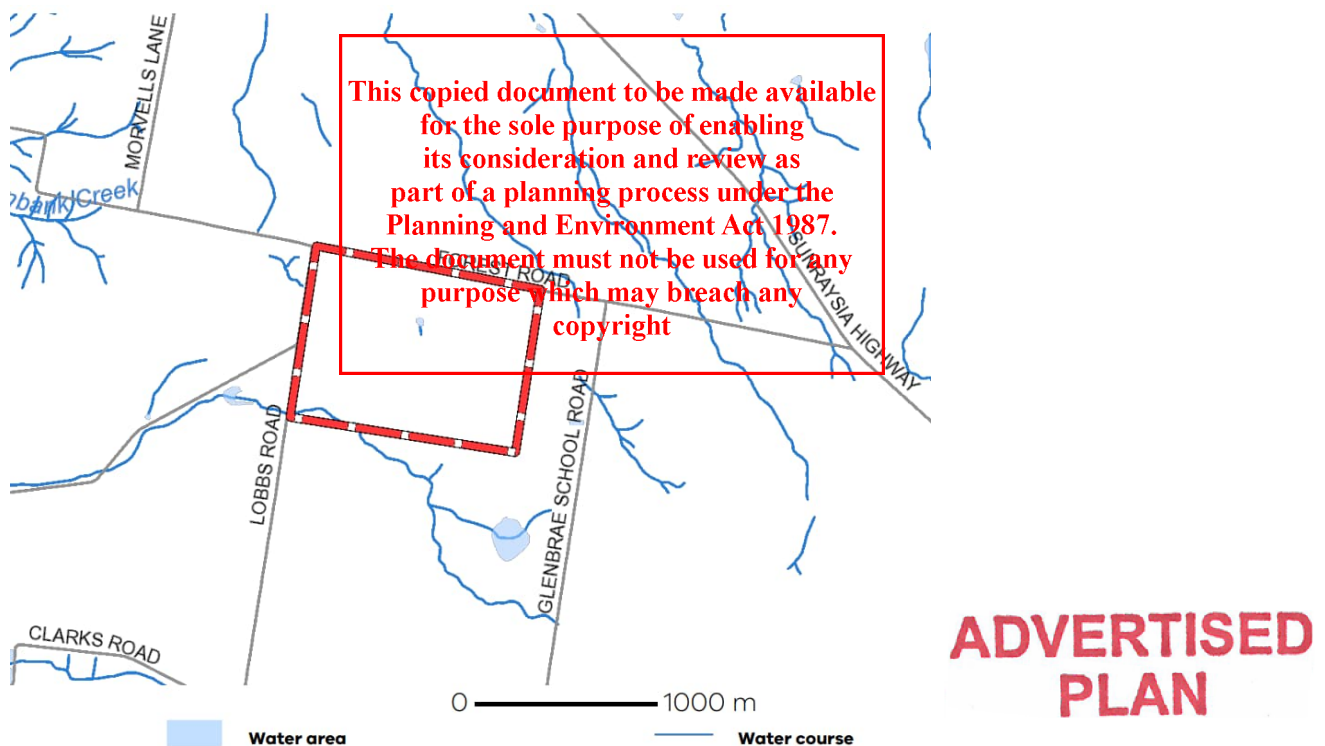


Figure 4 –Existing Watercourse Locations

#### 3.1.2 Developed Conditions

The battery farm site will be constructed along the northern boundary of the title, this will result in an increase in impervious surface. Runoff from the site will need to be attenuated from post-development flows to pre-development flows. The location of the site will also mean that localised external catchments will need to be routed around the proposed development area.

A summary of catchment characteristics and layouts are presented in Figure 5.

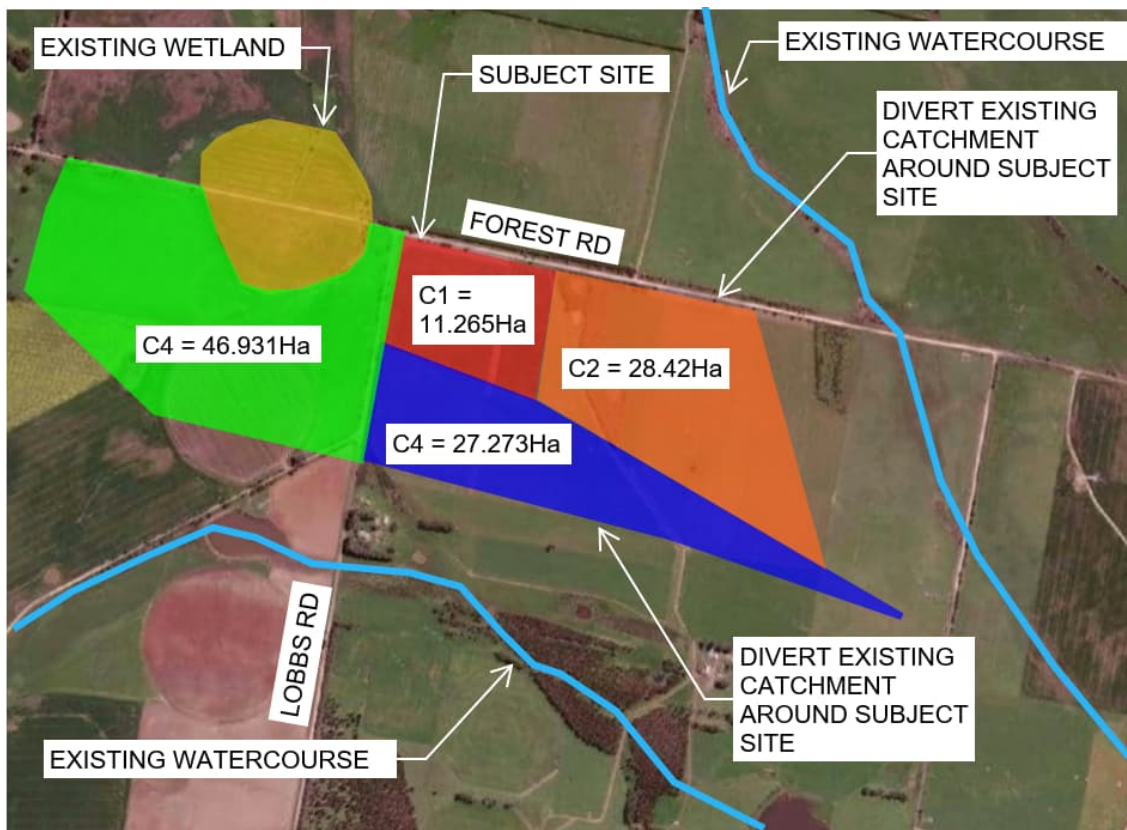


Figure 5 – Regional Catchment characteristics

Refer Table 1 for a summary of catchment characteristics.

Table 1– Existing Catchment Characteristics

| Catchment           | Catchment Name          | Impervious percentage to be applied (%) | Area (Ha) | Impervious area (Ha) | Pervious area (Ha) |
|---------------------|-------------------------|---|-----------|----------------------|--------------------|
| Existing Conditions | C1 – Subject Site       | 0                                       | 10.5      | 0                    | 10.5               |
|                     | C2 – External Catchment | 5                                       | 28.420    | 1.421                | 26.999             |
|                     | C3 – External Catchment | 5                                       | 27.273    | 1.364                | 25.909             |
|                     | C4 – External Catchment | 5                                       | 46.931    | 2.346                | 44.585             |
| Total               |                         |   | 113.89    | 5.13                 | 108.76             |

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## 3.2 Developed Conditions and Legal Point of Discharge

Estimations of the increase in impervious area were carried out using Melbourne Water's MUSIC Modelling Guidelines (2018), areal imagery assessment and assessment of the proposed layout plan for the development site. A summary of developed catchments is presented in Figure 6, Catchment characteristics are presented in Table 2.

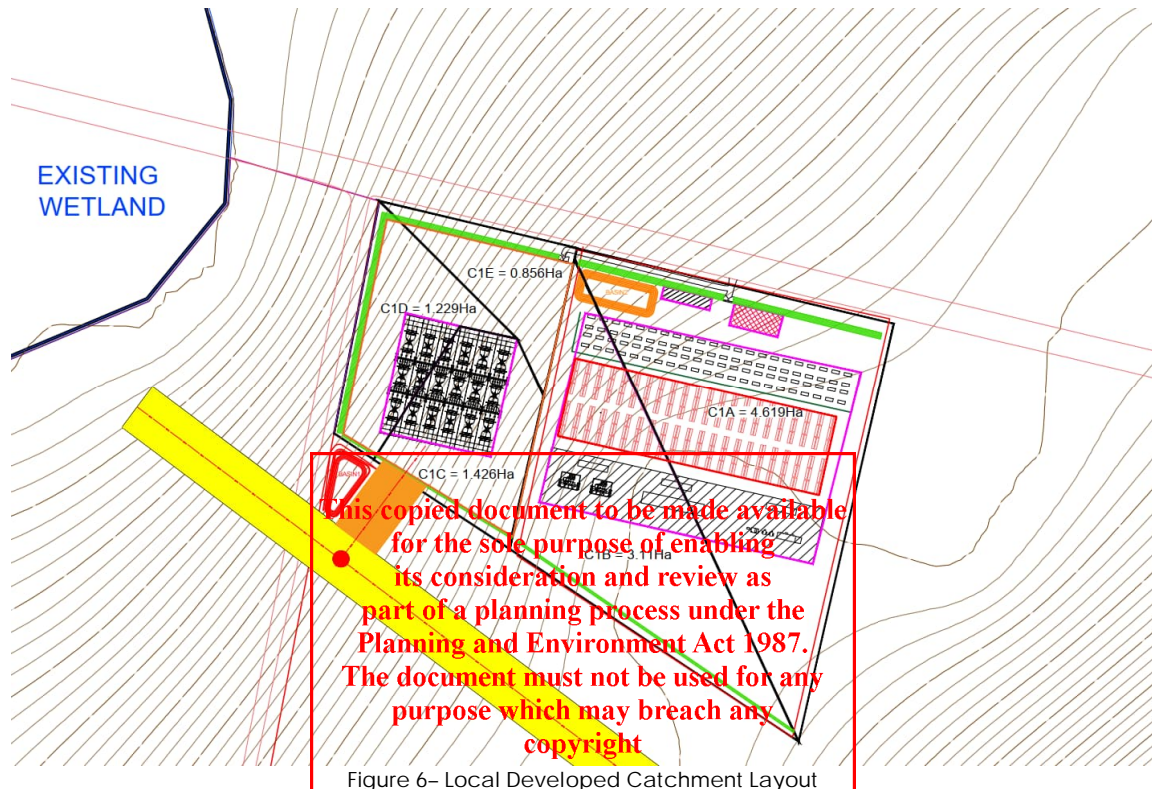


Table 2 – Post Development Catchment Characteristics

| Catchment            | Catchment Name | Impervious percentage to be applied (%) | Area (Ha) | Impervious area (Ha) | Pervious area (Ha) |
|----------------------|----------------|---|-----------|----------------------|--------------------|
| Developed Conditions | C1A            | 42                                      | 4.619     | 1.92                 | 2.70               |
|                      | C1B            | 44                                      | 3.11      | 1.37                 | 1.74               |
|                      | C1C            | 37                                      | 1.426     | 0.52                 | 0.90               |
|                      | C1D            | 23                                      | 1.229     | 0.28                 | 0.95               |
|                      | C1E            | 10                                      | 0.856     | 0.09                 | 0.77               |
| Total                |                |   | 10.5      | 4.18                 | 7.07               |

\* C2, C3 & C4 remain unchanged from existing conditions provided in table 1 above.

This highpoint is located at the southeast corner of the developable portion of the site. To minimise the amount of site discharge points and to compliment site topography, all flows will be directed toward the intersection of Forest Road and Lobbs Road.

In order to limit flows back to predeveloped flow rates prior to discharge offsite, two detention basins will be constructed in the South-West and Northern portions of the site to capture flows from catchments C1C and C1A & C1B, respectively. Ultimately, flows from catchments C1D and C1E will remain undetained due to the relevant areas being proposed for future switching stations which inhibits the provision of detention basin anywhere within the boundaries of C1C, C1D, and C1E catchments. Notwithstanding, it is proposed that catchment C1C be graded to the south to allow interception by proposed basin 2.

Additionally, external flows from the east and south are to be directed around the subject site via either the windrow method or by appropriately sized open drains as part of the detailed design plans for the site.

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## 4 Methodology / Results

### 4.1 Stormwater Quantity

#### 4.1.1 Model Hydrology & Parameters

For the stormwater quantity assessment, DRAINS software has been utilised using ARR 2019 procedures. The design rainfall data has been collected from the ARR Data Hub for the following longitude and latitude:

- Longitude: 143.5606
- Latitude: -37.3292

The Intensity Frequency Data (IFD) for the area has been adopted from the data supplied by the Bureau of Meteorology (refer Table 3 – Rainfall Intensity (mm/hr) in Glenbrae VIC (mm, Source: Bureau of meteorology, 2021).

Table 3 – Rainfall Intensity (mm/hr) in Glenbrae VIC (mm, Source: Bureau of meteorology, 2021)

| Duration | AEP (%) |       |      |      |      |      |      |
|----------|---------|-------|------|------|------|------|------|
|          | 63.20%  | 50%#  | 20%* | 10%  | 5%   | 2%   | 1%   |
| 1 min    | 82.2    | 94.7  | 137  | 169  | 202  | 250  | 289  |
| 2 min    | 67.8    | 77.5  | 111  | 136  | 164  | 201  | 232  |
| 3 min    | 61.4    | 70.3  | 101  | 124  | 158  | 182  | 210  |
| 4 min    | 56.5    | 64.9  | 93.3 | 113  | 137  | 169  | 195  |
| 5 min    | 52.5    | 60.4  | 87   | 107  | 128  | 158  | 182  |
| 10 min   | 39.2    | 45.2  | 65.8 | 80.9 | 96.8 | 120  | 139  |
| 15 min   | 31.7    | 36.6  | 53.2 | 65.6 | 78.6 | 97.5 | 113  |
| 20 min   | 26.8    | 31    | 45   | 55.8 | 66.5 | 82.5 | 95.9 |
| 25 min   | 23.4    | 27    | 39.2 | 48.3 | 57.9 | 71.8 | 83.4 |
| 30 min   | 20.9    | 24.1  | 34.9 | 42.9 | 51.5 | 63.8 | 74   |
| 45 min   | 16.1    | 18.5  | 26.6 | 32.7 | 39.1 | 48.3 | 56   |
| 1 hour   | 13.3    | 15.3  | 21.8 | 26.7 | 31.9 | 39.3 | 45.5 |
| 1.5 hour | 10.3    | 11.7  | 16.5 | 20.1 | 23.9 | 29.3 | 33.8 |
| 2 hour   | 8.54    | 9.69  | 13.6 | 16.5 | 19.5 | 23.8 | 27.3 |
| 3 hour   | 6.63    | 7.49  | 10.4 | 12.5 | 14.7 | 17.8 | 20.4 |
| 4.5 hour | 5.18    | 5.83  | 7.98 | 9.55 | 11.2 | 13.4 | 15.3 |
| 6 hour   | 4.36    | 4.9   | 6.67 | 7.93 | 9.23 | 11.1 | 12.6 |
| 9 hour   | 3.42    | 3.83  | 5.19 | 6.15 | 7.12 | 8.51 | 9.62 |
| 12 hour  | 2.87    | 3.21  | 4.34 | 5.14 | 5.94 | 7.09 | 8    |
| 18 hour  | 2.21    | 2.49  | 3.37 | 3.99 | 4.61 | 5.49 | 6.2  |
| 24 hour  | 1.82    | 2.05  | 2.79 | 3.31 | 3.84 | 4.58 | 5.17 |
| 30 hour  | 1.56    | 1.76  | 2.4  | 2.86 | 3.32 | 3.97 | 4.47 |
| 36 hour  | 1.37    | 1.54  | 2.11 | 2.52 | 2.94 | 3.51 | 3.97 |
| 48 hour  | 1.1     | 1.24  | 1.71 | 2.05 | 2.41 | 2.88 | 3.26 |
| 72 hour  | 0.791   | 0.894 | 1.24 | 1.5  | 1.77 | 2.13 | 2.42 |

Rainfall data files utilised for the DRAINS model can be provided on request.

The Initial Losses (IL) – Continuing Losses (CL) model has been utilised to assess the proposed stormwater drainage. Accordingly, values for IL and CL have been assigned from the retrieved IL and CL values from the ARR Data Hub. Table 4 summarises the adopted hydrological loss parameters.

Table 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   | 26                      | 1.6                  | 24.40                   | 4.5                     |
| Impervious | 1                       |                      | 0                       | 0                       |

#### 4.1.2 Hydraulic Assessment

To demonstrate compliance, a comparative analyses of the flow rates generated from the median storm events up to and including the 1% AEP event from 5 minutes to 72 hours for the pre and the ultimate development scenario has been carried out.

For this development the minor event has been assigned as the 10% AEP event and the major event has been assigned as the 1% AEP event.

## 4.2 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 critical peak discharges for the 1% and the 10% AEPs of the existing lumped catchment are tabulated in Table 5.

Table 5 – Predeveloped Peak Outflow using DRAINS hydrological model.

| AEP | Critical Event Duration | Critical Peak Discharge (m <sup>3</sup> /s) |
|-----|-------------------------|---|
| 1%  | 2 Hr                    | 0.510                                       |
| 10% | 3 Hr                    | 0.140                                       |

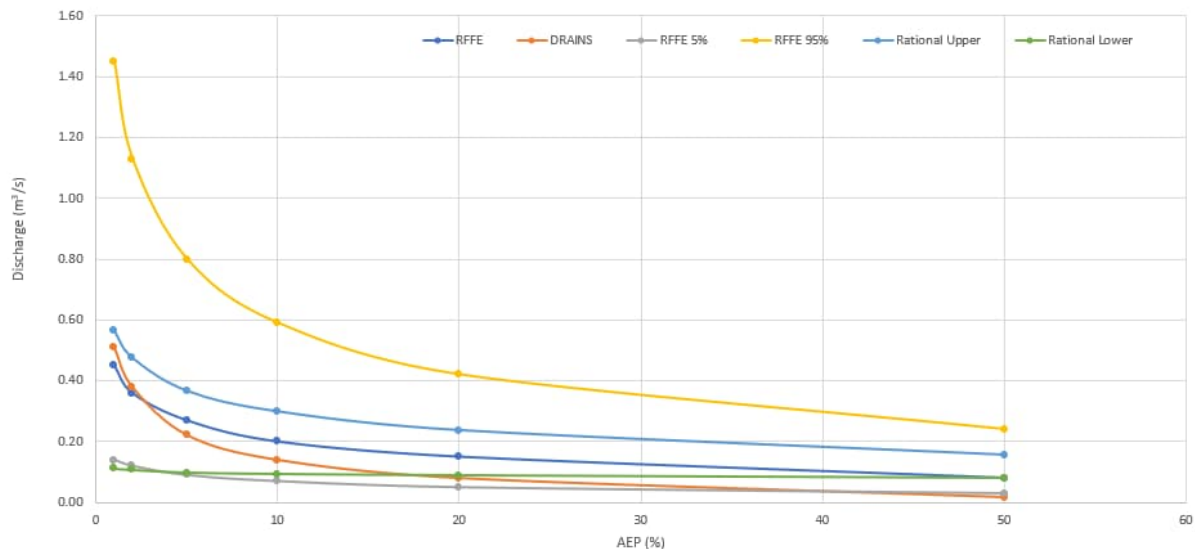
#### 4.2.1 Existing Site Discharge Validation

The above figures obtained from the hydrological model were determined using the modelling inputs discussed in section 4.1.1 above. To ensure these flows were reasonable the resultant modeled flows were compared against pre-developed flows determined using the Regional Flood Frequency Estimate (RFFE), as well as the Rational Method (utilizing Friend's Equation for the Time of Concentration), the outcome of this comparison is shown below in Figure 7.

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## Existing Catchment



~~Figure 7- Predeveloped Rates Peak Flow Validation~~

The hydrological model estimated a major peak discharge between those of rational and RFFE models. However, the hydrological model estimated minor event peak discharge lower than the RFFE figure and in between the two rational method scenarios considered. It is on this basis that the DRAINS model is considered the most representative model available for the subject catchment and will be used for hydrological modeling of the various scenarios.

### 4.3 Developed Site Discharge

Due to the increase in fraction imperviousness resulting from the development of the subject site initial modelling was undertaken to confirm the requirement for onsite detention. The detention facility will be sized to cater for events up to and including the 1% AEP critical duration event. The outfall from the detention basin will be sized to ensure that outgoing flow is equal to or less than the 1% AEP and 10% AEP event pre-developed flows for the critical storm durations. Figure 8 below illustrates the proposed stormwater conveyance and detention network.

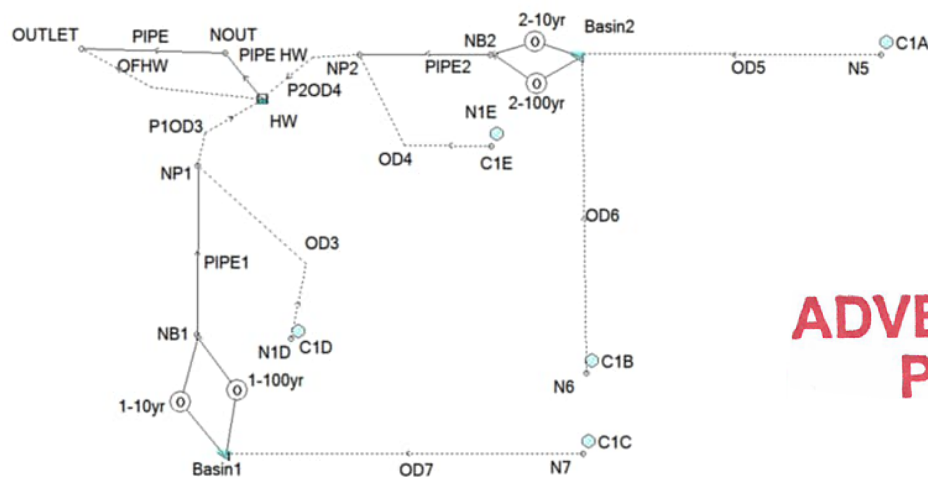


Figure 8 - Stormwater Conveyance Network

#### 4.3.1 Stormwater Detention

Two detention basins were modelled in DRAINS utilising an Initial Losses (IL) – Continuing Losses (CL) model to assess the detention and outfall requirements of the developed site. The results from the model are represented below.

System attributes are presented below in Table 6 through to Table 8 below.

Table 6 – Developed Outflow Rates from Detention Basins

| AEP | Basin 1                 |                                | Basin 2                 |                                |
|-----|-------------------------|--------------------------------|-------------------------|--------------------------------|
|     | Critical Event Duration | Critical Peak Discharge (m³/s) | Critical Event Duration | Critical Peak Discharge (m³/s) |
| 1%  | 3hr                     | 0.041                          | 3hr                     | 0.38                           |
| 10% | 3hr                     | 0.007                          | 3hr                     | 0.043                          |

Table 7 – Detention Basins Summary

| Characteristics            | 1% AEP Event |         | 10% AEP Event |         |
|----------------------------|--------------|---------|---------------|---------|
|                            | Basin 1      | Basin 2 | Basin 1       | Basin 2 |
| Top of Water Level (m)     | 424.189      | 423.69  | 424.42        | 425.12  |
| Basin Volume (m³)          | 387          | 1834    | 195           | 1072.7  |
| Basin Area @ TWL (m²)      | 725          | 1461    | 660           | 1116    |
| Basin Invert Level (m AHD) | 424.1        | 423.996 | 424.1         | 423.996 |

Table 8 – Detention Basins Outlets

| Basin   | Parameter                   | Stage   | Size        |
|---------|-----------------------------|---------|-------------|
| Basin 1 | Circular Orifice (Minor)    | 424.1   | 80mm        |
|         | Rectangular Orifice (Major) | 424.42  | 250W x 100H |
| Basin 2 | Circular Orifice (Minor)    | 423.996 | 140         |
|         | Rectangular Orifice (Major) | 425.12  | 480W x 400H |

#### 4.3.2 Ultimate Site Discharge

As described in section 3.2 of the current report, and considering that catchments C1D and C1E will remain un-detained due to the future site retractions, the ultimate site discharge will consist of the detention basins outflows and un-detained flows from the aforementioned catchments. Table 9 below summaries the ultimate site discharge under developed against the existing conditions.

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Table 9 - Ultimate Site Discharge Summary

| AEP | Critical Peak Discharge (m <sup>3</sup> /s) – Basin 1 | Critical Peak Discharge (m <sup>3</sup> /s) – Basin 2 | Critical Peak Discharge (m <sup>3</sup> /s) – Basins Combined | Un-detained Flows (m <sup>3</sup> /s) | Site Peak Discharge (m <sup>3</sup> /s) | PSD  |
|-----|---|---|---|---------------------------------------|---|------|
| 1%  | 0.041   | 0.38  | 0.421   | 0.089                                 | 0.476                                   | 0.51 |
| 10% | 0.007   | 0.043   | 0.05  | 0.091                                 | 0.141                                   | 0.14 |

## 4.4 Stormwater Quality

### 4.4.1 Stormwater Quality Objectives

As the proposed development shall impact the quality of stormwater runoff, treatment measures in line with best practice pollutant removal targets are proposed to improve the quality of stormwater runoff prior to discharge off site.

The proposed treatment system shall be designed to reduce pollutants discharging from the development area. In accordance with the Urban stormwater best practice environmental management guidelines (referenced by the EPA), Figure 9 presents the required stormwater pollutant retention prior to discharge offsite.

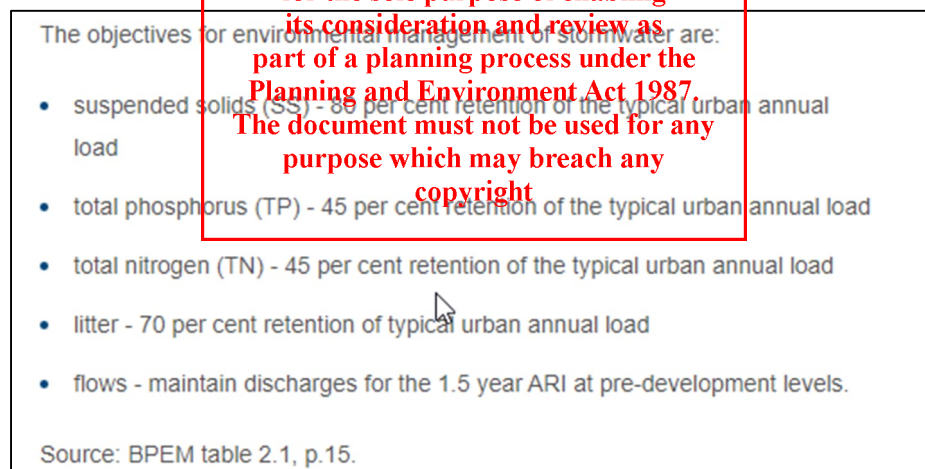


Figure 9 - Site Pollutant Retention Requirements (Extract from the EPA)

### 4.4.2 Stormwater Quality Model

To demonstrate compliance with stormwater quality Objectives, MUSIC software has been utilised. The model has generally been set up in accordance with Melbourne Water's MUSIC Modelling Guidelines. Refer below for the assigned model parameters.

#### Rainfall Data

6 minute rainfall data was obtained from the Bureau of Meteorology for the Ballarat Aerodrome. The MUSIC model was run with a 6 minute timestep for the full dataset to ensure an accurate assessment. Table 10 below summaries the catchment properties adopted in the model.

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Table 10 - MUSIC Simulations – Lot/Road Properties

| Catchment Characteristics             |             |
|---------------------------------------|-------------|
| Catchment Name                        | See Table 2 |
| Zoning/Surface Type                   | Mixed       |
| Impervious Area (%)                   | See Table 2 |
| Pervious Area (%)                     | See Table 2 |
| 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           |

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#### Catchment Input

MUSIC catchments have been assigned as per Figure 10. It should be noted that any catchment bypassing the developed area (including the external catchment directly to the east and south of the battery farm) have been excluded from the model.

#### Treatment System Parameters

It was determined that a nominated treatment train that includes the proposed detention basins as well as proposed swales (OD3 – OD7 and POD3 and POD4), to convey runoff towards the respective legal points of discharge was required to meet the quality objectives outlined in Figure 9. Modelled parameters of the treatment train are provided in Table 11.

Table 11. Treatment Train Parameters – MUSIC Inputs

|                           | OD3   | OD4  | OD5  | OD6  | OD7  | POD3 | POD4  |
|---------------------------|-------|------|------|------|------|------|-------|
| Inlet Properties          |       |      |      |      |      |      |       |
| Low Flow By-pass (m³/s)   | 0.0   |      |      |      |      |      |       |
| High Flow By-pass (m³/s)  | 100.0 |      |      |      |      |      |       |
| Storage Properties        |       |      |      |      |      |      |       |
| Length (m)                | 107   | 45.6 | 202  | 252  | 139  | 90   | 104.4 |
| Slope (%)                 | 1     | 2.8  | 0.89 | 1.11 | 3.27 | 0.99 | 1.92  |
| Base Width (m)            | 1     | 0.5  | 2    | 0.5  | 0.2  | 1    | 1     |
| Top Width (m)             | 9     | 4    | 5    | 4    | 2.7  | 9    | 9     |
| Depth (m)                 | 1.5   | 0.35 | 0.3  | 0.35 | 0.25 | 1.5  | 1.5   |
| Vegetation Height (m)     | 0.5   | 0.2  | 0.2  | 0.2  | 0.15 | 0.5  | 0.5   |
| Exfiltration Rate (mm/hr) | 3.6   | 3.6  | 3.6  | 3.6  | 3.6  | 3.6  | 3.6   |

#### Results

Figure 10 below presents the MUSIC layout and Table 12 presents the results of the MUSIC modelling results; results are consistent and compliant with proposed pollutant retention objectives (Figure 9).

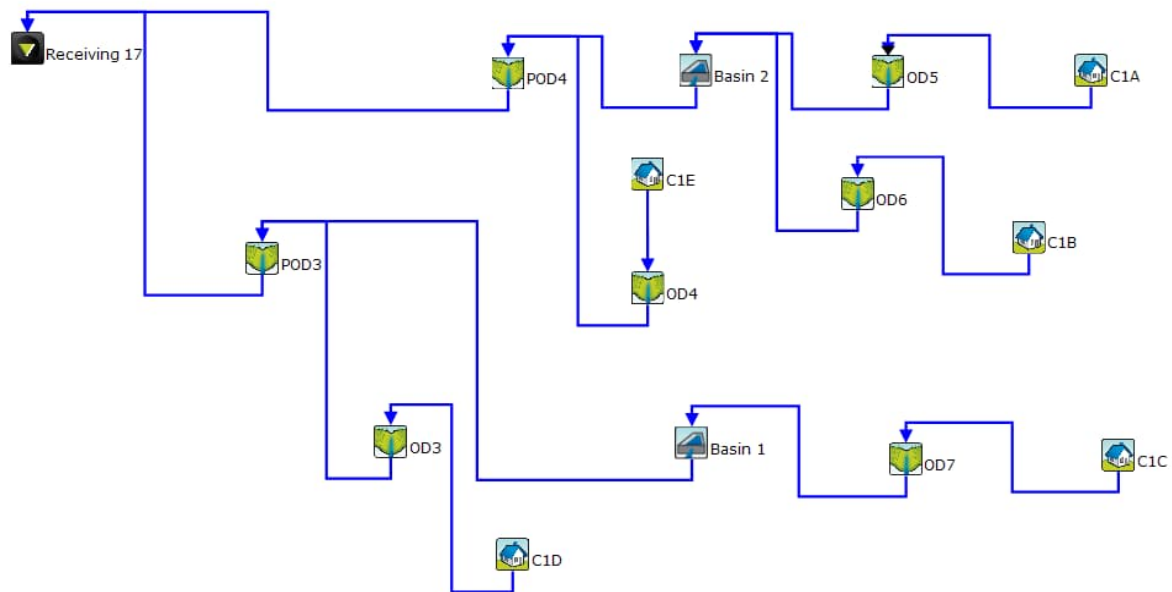


Figure 10- MUSIC Layout – Water Quality Treatment

Table 12 – Music Results

| Scenario                       | Source Load | Residual Load | Reduction (%) |
|--------------------------------|-------------|---------------|---------------|
| Flow (ML/yr)                   | 671.83      | 567.32        | 16            |
| Total Suspended Solids (kg/yr) | 130835.26   | 26463.46      | 80            |
| Total Phosphorus (kg/yr)       | 358.48      | 104.19        | 71            |
| Total Nitrogen (kg/yr)         | 2500.31     | 1278.97       | 49            |
| Gross Pollutants (kg/yr)       | 3800.43     | 0.00          | 100           |

The Music Assessment demonstrated that vegetated swales and an end of line sedimentation/detention basin provides sufficient treatment and pollutant reduction for stormwater runoff.

#### 4.4.3 Assessment Limitations

The water quality and quantity assessments carried out above do not address prevention of harm to the environment and human health associated with storing and handling of various substances. All design of site infrastructure including Batteries, fuel areas, vehicle laydowns, etc. should be designed in accordance with the relevant act and guidelines from the appropriate authority such as the Environment Protection Authority (EPA), Standards Australia and others as required.

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## 4.5 Stormwater Conveyance

Stormwater conveyance is required to be undertaken in and around the subject site to ensure that surface flow within the site is directed to the proposed detention basin and to ensure that flows external to the site do not impact on the proposed development.

To achieve this the following open drain and culvert arrangement is proposed in Figure 11.

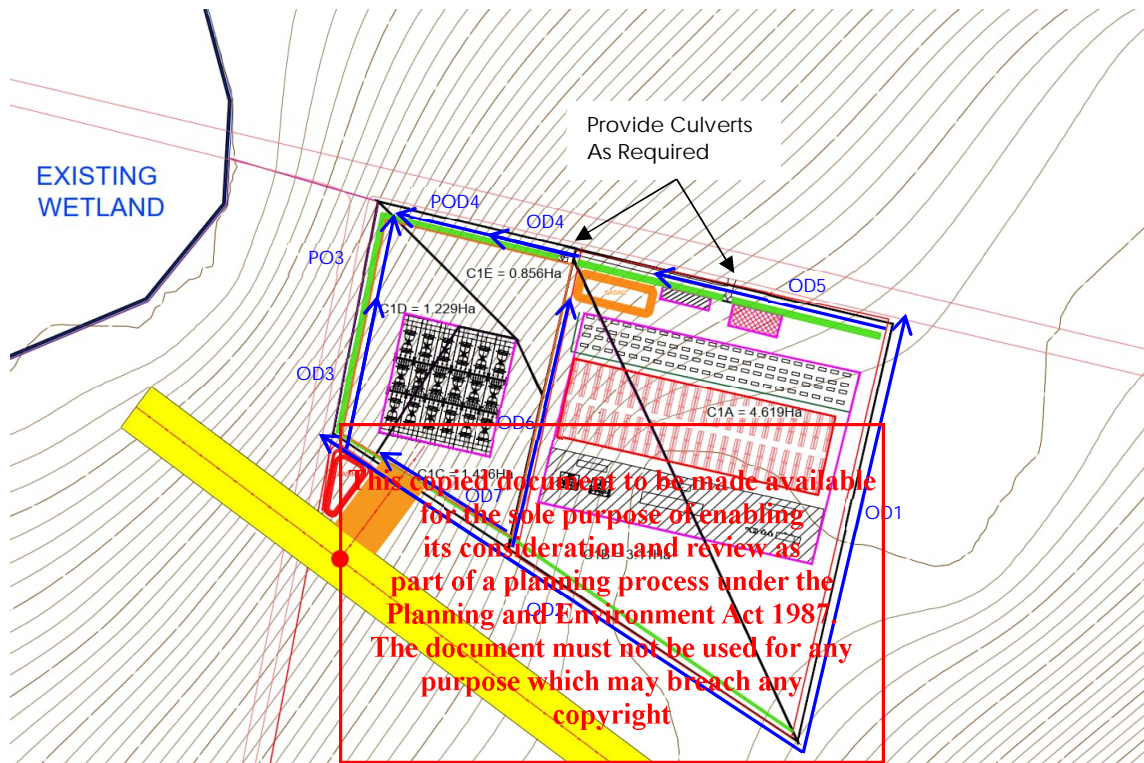


Figure 11 - Open Drain and Culvert Arrangements.

Table 13 also summaries the open drain characteristics.

Table 13 – Open Drain Requirements

| Name | Depth (mm) | Base Width (mm) | Side Slope (1H:xV) | Comment   |
|------|------------|-----------------|--------------------|---|
| OD1  | 300        | -               | 10                 | Provide bund, garden bed or similar to prevent external flows from entering site. |
| OD2  | 300        | -               | 10                 | Provide bund, garden bed or similar to prevent external flows from entering site. |
| OD3  | 1500       | 1               | 5                  | Vegetated open drain  |
| OD4  | 350        | 0.5             | 5                  | Vegetated open drain  |
| OD5  | 300        | 2               | 5                  | Vegetated open drain  |
| OD6  | 350        | 0.5             | 5                  | Vegetated open drain  |
| OD7  | 250        | 0.2             | 5                  | Vegetated open drain  |
| POD3 | 1500       | 1               | 4                  | Vegetated open drain  |
| POD4 | 1500       | 1               | 4                  | Vegetated open drain  |

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## 5 Assessment of Downstream Wetland

There currently lies an existing shallow wetland approximately 120m west of the subject site, which receives flows from upstream catchments inclusive of runoff from the subject site. The existing wetland is approximately 400m in diameter and is split through the middle by Forest Road. It is unclear if the wetland is balanced by an existing culvert or whether it topples over the crest of the roadway when inundated.

Based on site photos and publicly available aerial imagery, it is expected that catchments C1 (subject site), C2, C3 and C4 drain to and pass through this wetland via sheet flow and concentrated overland channels within the road reserves. the wetland appears to be shallow and consists of sparsely positioned low lying vegetation with a small cluster of trees either side of Forest Road.

As part of this report, the impacts of the proposed development on existing wetland were assessed for any potential impacts that may be caused by and increase stormwater peak flows, increase in stormwater pollutants or any significant changes in mean annual runoff.

Both the DRAINS model and MUSIC model developed as part of the assessments were used to assess the impacts upon the existing development. The calibration and data used in developing these models are detailed within the previous sections of this report and shall be referred to in conjunction with the following assessment.

### 5.1 Peak Flow Discharge

Two stormwater detention basins have been provided as discussed in section 4.3 to ensure peak flows experienced by the existing wetland are maintained at pre-developed rates for events up to and including the 1% AEP stormwater event. Table 14 below compares the change in peak flow between developed and existing conditions.

Table 14 – Peak Flow Summary

| Scenario             | 1% AEP                 | 10% AEP                |
|----------------------|------------------------|------------------------|
| Existing Conditions  | 0.510m <sup>3</sup> /s | 0.140m <sup>3</sup> /s |
| Developed Conditions | 0.476m <sup>3</sup> /s | 0.141m <sup>3</sup> /s |

### 5.2 Water Quality and Mean Annual Runoff

Water balance models of the subject site and regional catchments were created using the MUSICX conceptual modelling software program. There is no flow data available appropriate for use in calibrating the catchment runoff response in the model and hence the adopted modelling parameters were based on standard parameters typical for catchments of this nature. This assessment focuses on the relative changes in hydrological regime between the developed and existing scenarios.

A daily time-step analysis was used as nominated above in section 4.4.2 and includes assessment of data over 45 years of data (from 20/8/1954 to 31/07/1999) to calculate a timeseries of flows discharging to the existing wetland. A MUSICX node representing the existing wetland was created to estimate the mean annual volume of water entering the wetland from the upstream catchments (inclusive of the developed site).

The parameters of the existing wetland modelled within MUSICX is shown in Table 15 below:

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Table 15 – Existing Wetland Parameters

| Inlet Properties                     |                         |
|--------------------------------------|-------------------------|
| Low-Flow Bypass                      | 0m <sup>3</sup> /s      |
| High-Flow Bypass                     | 100m <sup>3</sup> /s    |
| Inlet Pond Volume                    | 0m <sup>3</sup>         |
| Pervious Area Properties             |                         |
| Surface Area                         | 141,955m <sup>2</sup>   |
| Extended Detention Depth             | 0.25m                   |
| Permeant Pool Volume                 | 35,489m <sup>3</sup>    |
| Initial Volume                       | 35,489m <sup>3</sup>    |
| Vegetation Cover (% of surface area) | 50%                     |
| Exfiltration Rate                    | 0mm/hr                  |
| Evaporative Loss as % of PET         | 125%                    |
| Outlet Properties                    |                         |
| Equivalent Pipe Diameter             | Assumed no piped outlet |
| Overflow Weir Width                  | 400m                    |
| Notional Detention Time              | -                       |

Due to the absence of a feature survey of the site, the above information regarding the existing wetland was estimated using publicly available electronic contours. It should also be noted that for this reason the assessment investigates annual flow volumes and not water levels or volumes present within the existing wetland system.

The sub-catchment delineation of the regional catchments can be found in Figure 5 – Regional Catchment characteristics above.

#### 5.2.1 Assessment of Flow Changes to Existing Wetland

To gain an initial appreciation of how flows into the existing wetland would be affected by development of the subject site, runoff time series were generated for both cases. A summary of the results for the existing and developed case are presented in table 16 below.

Table 16 – Raw Data Statistical Comparison

| Univariate Total Statistic | Pre-Developed Condition | Developed Condition | Percentage Change |
|----------------------------|-------------------------|---------------------|-------------------|
| Minimum (ML/day)           | 0.00                    | 0.00                | 0                 |
| Maximum (ML/day)           | 115.5                   | 97.06               | 15.97             |
| Mean (ML/day)              | 0.074                   | 0.073               | 0.88              |
| Median (ML/day)            | 0.031                   | 0.03                | 3.29              |
| Std Deviation              | 0.58                    | 0.53                | -                 |
| Sum (ML)                   | 291974                  | 289402              | 0.88              |

Due to the large amount of data assessed, significant volumetric changes and statistical variations can be unclear when assessed solely on an annual basis. It is also to be expected that the receiving ephemeral system and downstream land uses are more likely to be affected by changes in the hydrological regime on a monthly and seasonal basis than annually. Accordingly, the below analysis is presented at a monthly time scale: 6 minute runoff volumes from the model are aggregated to give total m<sup>3</sup>/month for each month.

Figure 12 show the comparison of the flow timeseries in m<sup>3</sup>/month for each month, for both the predeveloped and the developed scenarios.

The following boxplots provide a visual representation of the following;

- the box extents plotted are the 25% and 75% percentiles;
- the dark horizontal bar in the box is the median value;
- the 'cross-hair' points indicate the 10th and 90th percentile values (solid dots on the end of the lines extending from the box);
- The coloured dots not connected to the box represent the outliers within the assessed data set

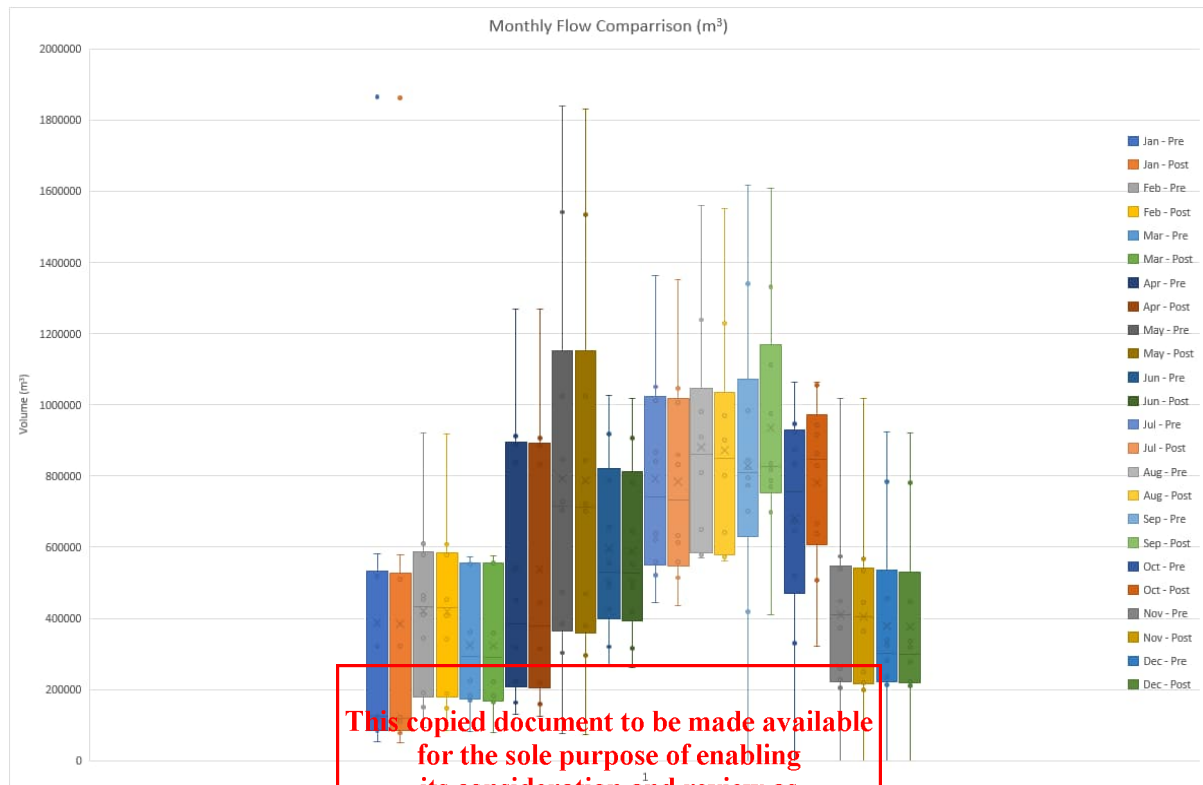


Figure 12- Box and Whisker Plot

As can be seen above only a very slight variation exists between the predevelopment and developed scenarios for the mean, quartile 3 and within the outliers for all months assessed.

Comparison of the mean value only is provided in Table 17 and indicates that as experienced within the mean annual flow that a small variance also exists on a monthly basis, notably a slight reduction. This experienced reduction is expected to be due to the detention basins and formalising of open drains allowing flows more opportunity to infiltrate and evaporate.

Table 17 – Monthly Mean Volume Comparison (m³)

| Month     | Pre Developed Condition | Developed Condition | Percentage Change |
|-----------|-------------------------|---------------------|-------------------|
| January   | 387063.1                | 383509.6            | -0.92%            |
| February  | 421548.6                | 418755.9            | -0.66%            |
| March     | 324262.6                | 322981.5            | -0.40%            |
| April     | 541652.9                | 538334.7            | -0.61%            |
| May       | 791878.4                | 787279.4            | -0.58%            |
| June      | 595524.9                | 588455.7            | -1.19%            |
| July      | 791858                  | 784679              | -0.91%            |
| August    | 879760.9                | 871195.5            | -0.97%            |
| September | 921788.8                | 934455.4            | 1.37%             |
| October   | 756132.4                | 780395.1            | 3.21%             |
| November  | 455536.7                | 450013.3            | -1.21%            |
| December  | 419893.5                | 416497.6            | -0.81%            |

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## 6 Stormwater Management Strategy

### 6.1 Proposed Treatment/Detention System

Based on the results from this assessment, the required stormwater detention and treatment measures are:

- Two detention basins with a depth of storage volume of 387 and 1834m<sup>3</sup>;
- Two detention basin with minimum surface area @ top water level of 725 and 1461m<sup>2</sup> (based on 1 in 4 batters);
- Open drain or windrow to be provided on southern and eastern boundary to ensure external flows are directed around the subject site;
- Internal open drains to direct flows to detention basin.

Considering the relatively steep nature of existing terrain, and the absence of assets near the site low point, it may not be required that the detention basins need to be cut into the existing ground, alternatively a bund wall can be installed to induce ponding to the volume as nominated above. An outlet condition will need to be designed and installed to limit outflow to pre-developed rates. Confirmation of construction detail will need to be considered during the detailed design phase.

The below Figure 13 outlines the proposed stormwater management strategy for the site.

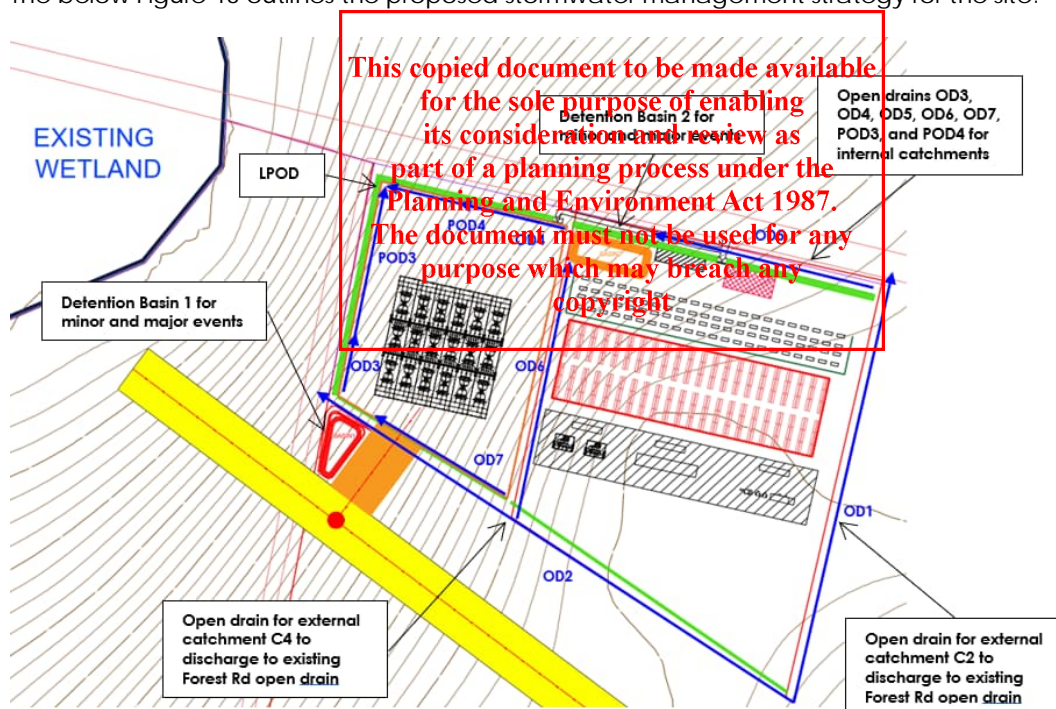


Figure 13- Schematic Stormwater Strategy

### 6.2 Construction Management

Prior to the commencement of construction the site must implement effective management practices consistent with relevant guidance, and to undertake monitoring where construction activities adjoin or cross surface waters to assess if beneficial uses are being protected.

Persons responsible for construction activities can reduce the risk of impacts on beneficial uses by managing their activities consistent with current best practice or guidance published by the Authority, including:



- the Environmental Guidelines for Major Construction Sites (EPA Publication 480),
- Construction Techniques for Sediment Pollution Control (EPA Publication 275),
- Doing it Right on Subdivisions: Temporary Environment Protection Measures for Subdivision Construction Sites (EPA Publication 960);

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## 7 Conclusion / Recommendations

The report shows that stormwater generated within the proposed development at 438 Lobbs Road, Glenbrae, can be treated to meet 'best practice' design objectives. This can be achieved with the adoption of a stormwater treatment train which consists of vegetated/grassed swales and two detention basin facilities.

Stormwater peak discharges generated by the 10% and 1% AEP storm event from the developed site can be managed to achieve a 'no-worsening' outcome when compared to existing conditions.

Stormwater runoff conveyed within the water quality treatment train will achieve the required reductions in pollutant loads using the outlined parameters.

The impacts of developing the subject site have been assessed for increases in stormwater quantity, stormwater quality and mean annual runoff, and have been deemed to cause no worsening effects on the existing downstream wetlands with the implementation of the proposed stormwater strategy.

Planit recommends that:

- A stormwater strategy is delivered per Figure 13 of this report;
- External catchments be directed away from the site and the associated proposed detention basins;
- Stormwater quality treatment is to be provided in the form of two detention basins and treatment swales within the site;
- Onsite detention is provided to a volume equating 387m<sup>3</sup>, for Basin 1 and 1834m<sup>3</sup> for Basin 2, assuming 0.6 and 1.7m depth, respectively;
- Outflow from the detention basins to be controlled to limit flows to pre-developed conditions;
- Earthworks and drainage design calculations are to be completed to confirm conveyance of external flow around site. It is anticipated that the hardstand area should be built up above existing surface levels to ensure appropriate conveyance of stormwater;
- Updated basin calculations are completed to inform stage-storage relationship during detailed design. Considering the shallow depth of Basin 1, any change in surface area and storage depth may impact the total volume required;
- Approvals/permit sought for all design and construction activities beneath the high voltage powerlines.

Based on the assessment undertaken, the proposed development can be delivered in a compliant manner to ensure no worsening of existing conditions.

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