INFRASTRUCTURE SERVICING REPORT

Jetty Rd Urban Growth Area (Stage 2) – South of Bellarine Rail Trail

Reference No. 30043260E

Prepared for APD Projects Pty Ltd

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1 Introduction

This report has been produced by SMEC Australia at the request of our client APD Projects Pty Ltd for the purposes of providing a high-level infrastructure servicing analysis. The service analysis is associated with the land proposed for rezoning as part of Stage 2 of the Jetty Road Urban Growth Zone (UGZ) south of the Bellarine Rail Trail. The area can be seen presented in Figure 1 and will be known from herein within this report as the 'Subject Site'.

The Subject Site is located within the City of Greater Geelong, approximately 17 km east of the Geelong CBD and covers an area of approximately 53 Ha. The site is bound by Portarlington Road along it southern boundary, Tivoli Drive along its western boundary, Bellarine Rail Trail along its northern boundary and Jetty Road/Hackwill Place along its eastern boundary.

The scope of the report is to identify any servicing and/or engineering constraints to urban development of the Subject Site in accordance with the local authority requirements.

Figure 1: Locality Plan

Where specific information has not been available, we have made strategic assumptions based on conversations and informal advice from authorities and similar project experience.

The information in this report is subject to variation upon formal advice from the relevant authorities, detailed design and provision of survey information. SMEC will not accept any responsibility for changes in authority requirements or more accurate information received after the date of this report.

Based on the Company's experience and the investigations carried out, SMEC believes that the Subject Site does have availability and access (subject to service authority approvals and negotiations) for connection to all necessary services, and that these services can accommodate the proposed development of the Subject Site.

2 Engineering Infrastructure Overview

2.1 Servicing Authorities

The following authorities are applicable to the Subject Site:

Table 1: Local Service Authorities

AUTHORITY	INFRASTRUCTURE
City of Greater Geelong	Local Roads and Drainage
Barwon Water	Sewer and Potable Water
Powercor - Geelong	Electricity (Distribution)
AusNet Services	Electricity (Transmission)
AusNet (Gas) Services Pty Ltd	Gas
NBN Co/Opticomm (Vic)/Optus/Telstra	Communications
Telstra/NBN/Opticomm	Telecommunications

2.2 Roads

2.2.1 Tivoli Drive

The existing cross section of Tivoli Drive constructed by the City of Greater Geelong in 2021 is an interim cross section which provides access to Stage 1 of the Jetty Road UGZ. As part of the development of the Subject Site the ultimate cross section of Tivoli Drive will need to be constructed between Portarlington Road and the Bellarine Rail Trail. The existing 16 m cross section will be increased to a 32 m cross section, requiring land take from the proposed development area along its western boundary. The ultimate cross section proposed for Tivoli Drive is presented in Figure 2. The number of connections which the development will have to Tivoli Drive is still to be determined, however it is expected to be in the order of two to three.

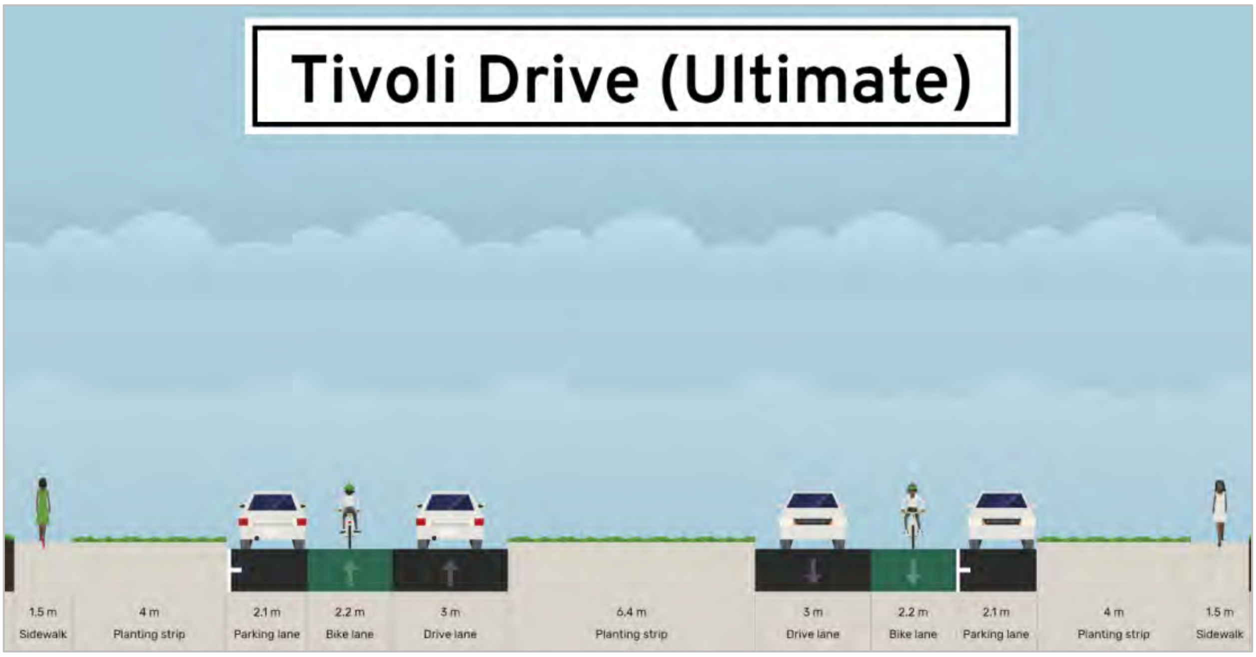

Figure 2: Tivoli Drive Ultimate Cross Section

2.2.2 Portarlington Road

No direct access to Portarlington Road will be made as part of the proposed development, therefore there will be no upgrade works associated with Portarlington Road.

2.2.3 Jetty Road / Hackwill Place

As part of the construction of the Drysdale Bypass completed by VicRoads in 2020, Hackwill Place was truncated at its southern end, immediately south of the service centre as presented in **Figure 3**. Any further access to Hackwill Place would be subject to Council approval.

It is expected that access to the eastern portion of the proposed development will be attained from Hackwill Place, through the construction of a Right-In, Left-Out intersection(s). Traffic will then be required to exit via the Jetty Road intersection.

Subject to further assessment as part of a traffic impact assessment, access from Jetty Road may be attained provided that sufficient separation to the existing intersection can be achieved.

Figure 3: Hackwill Place truncation at Portarlington Rd

2.2.4 Internal Roads

All internal roads will be delivered in accordance with the Infrastructure Design Manual (IDM) and the City of Greater Geelong (COGG) standards as applicable. The internal roads will be required to accommodate emergency services, refuse collection, construction and residential vehicles, whilst meeting accessibility requirements and relevant standards.

Internal road reserve widths are expected to comprise the following typical layouts which are common to existing Geelong growth areas:

- 20 m road reserve – Main entry boulevard.
- 16 m road reserve – Typical Local Access Street.
- 13.5 m road reserve - Typical Local Access Street Beside Public Open Space/Drainage Reserve; and
- 10 m road reserve – Concrete Access Lane.

2.3 Stormwater Drainage

2.3.1 General Requirements

The City of Greater Geelong is the responsible authority for all roadworks, minor and major drainage within the proposed development, and will be responsible for maintenance of drainage assets.

All proposed stormwater drainage will need to comply with best practice for stormwater management under clause 56.07 of the 'Victorian Planning Provisions and the stormwater quality objectives of the Urban Stormwater - Best Practice Environment Guidelines' (Victorian Stormwater Committee 1999).

The detailed design and documentation of all drainage infrastructure must address all major and minor drainage system element requirements and comply with City of Greater Geelong's Infrastructure Design Manual (IDM). The following design standards and reference documents must be complied with:

Stormwater Drainage Design

- Melbourne Water: Design Guidelines
- The Institute of Engineers, Australia (1998): Australian Rainfall and Runoff

Water Sensitive Urban Design

- Victoria Stormwater Committee (1999): Urban Stormwater – Best Practice Environmental Management Guidelines
- Melbourne Water (2004): WSUD Engineering Procedures – Stormwater
- Engineers Australia (2006): Australian Runoff Quality

Urban Waterways

- Royal Life Saving: Guidelines for Water Safety in Urban Water Developments
- Melbourne Water – Wetland Design Manual

2.3.2 Subject Site Catchment

The Subject Site has an area of approximately 53 hectares of developable land which generally grades from the south-east to the north-west, residing between 55 m to 75 m AHD.

An existing drainage flow path exists on the site running diagonally across the property from the dam in the south east corner to a Grated Outlet Pit at the low point of the site in the north west corner, as presented in **Figure 4**. The grated outlet pit is connected to a culvert crossing beneath Tivoli Drive.

In accordance with the SWMS developed by Water Technology a Wetland Retarding Basin (WLRB) will be constructed in the North West corner of the site to detain back to pre-developed condition. Flows will then be discharged via the existing culverts beneath Tivoli Drive.

A linear constructed waterway and wetland system will also be constructed in general accordance with **Figure 4**. Multiple sediment basins will be constructed to receive and treat flows from the designated site catchments. The sediment basins will use Melbourne Water Sediment Pond Transfer pits to allow Council to maintain an "offline" status and efficiently maintain the asset as required.

Figure 4: Proposed Outfall Drainage Strategy

As water exists the Tivoli Drive culverts, the flows will be contained within the existing channel located within the Council rail trail reserve on the north side of Curlewis Golf Course. The flows will then pass through the existing culvert beneath the rail trail and enter the development site at 91-125 Coriyule Road.

A "no detention strategy" has been assumed for this drainage outfall of 91-125 Coriyule Road based on the downstream open waterway in the adjacent farmland having sufficient capacity to receive unattenuated flows.

Figure 5: Drainage Outfall through 91-125 Coriyule Road (Water Technology, 2021)

2.3.3 Underground Drainage

Underground drainage is required to be designed in accordance with the Local Government Infrastructure Design Manual (IDM). The current minimum standard as nominated in the IDM for Urban Development within the Geelong Region is AEP 20% (5 Year ARI Event). Should the site require supplemented control to convey surface flows away from existing properties or to direct localised catchments to the retarding basin, increased capacity in both inlet and underground drainage could be adopted.

The underground stormwater infrastructure for the Subject Site will be positioned within road reserves and drainage easements to the satisfaction of CoGG. Outlet will be via discharge to the new sediment basin at the north-west corner of the site, subject to Council approval.

2.3.4 Overland Flow

All overland flow both within and exiting the Subject Site is required to be demonstrated to comply with IDM minimum standards for control and safe conveyance.

The residual component of total flow between the AEP 20% (1 in 5-year ARI flow) and AEP 1% (1 in 100-year ARI flow), referred to as gap flow, will require overland flow paths to assist in exiting the site safely to the Drainage facility. It is envisaged that a large component of the catchment gap flow will be conveyed within the Subject Site via road reserve to the point of discharge (constructed waterway / wetland reserve). Pavement construction with appropriate grading will be required to be to promote safe conveyance of overland flow with adequate freeboard to all allotments to the satisfaction of COGG.

Proposed allotments and existing properties located near flow paths will need to be designed to demonstrate that adequate freeboard is achieved. Minimum freeboard to all properties will be 300mm as specified in IDM (600 mm desirable) where adjoining known designated floodway, creeks, rivers etc and in other areas overland flows will need to be demonstrated to be safely contained within the road reserve. Both internal and external catchments contributing to the Subject Site will need to be considered.

2.3.5 Flood Overlay

No flood overlay exists on the proposed development site.

2.3.6 Stormwater Quality & Retardation

All stormwater at the Subject Site will need to be treated with Sediment Controls prior to discharge from the site. This will be achieved through the construction of a multiple sediment ponds within the drainage reserve in conjunction with the Wetland Retarding Basin asset in the north west corner of the site. The sediment ponds will receive all stormwater flows from the Subject Site & treat these prior to release into the culverts crossing Tivoli Drive and into the downstream waterway system.

2.4 Sewer

Barwon Water is the responsible authority for the provision of sewer reticulation to service the Subject Site. They have provided preliminary advice in relation to servicing requirements for the subject site, pending formal execution of a Developer Deed.

The preliminary servicing information supplied by Barwon Water has been included in **Appendix A**.

DN225/DN300 trunk sewer lines are required to be constructed north of the Bellarine Rail trail to provide outfall for the Subject Site catchment. Barwon Water propose that these assets are constructed within reserve areas located in the future development of 91-125 Coriyule Road. The outfall will ultimately connect to the Clifton Springs No. 7 Pump Station located in Coriyule Road. The alignment can be seen presented in **Figure 6**. The DN300 trunk sewer would be reimbursable works in accordance with Barwon Water's developer deed conditions.

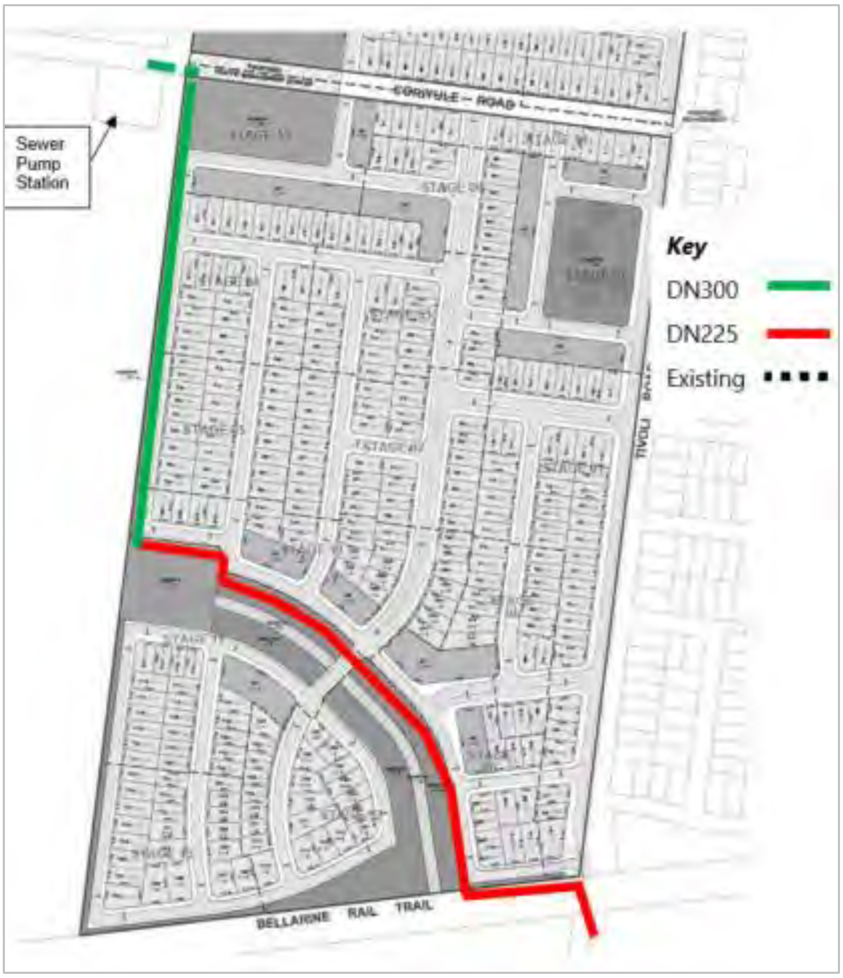

Figure 6: Outfall Sewer Alignment – Proposed sizing

It has been confirmed by Barwon Water that the existing DN225 sewer located in Coriyule Road and existing sewers located within Curlewis Parks Estate do not have capacity to service the proposed development south of the rail trail based on the proposed lot count.

Where development works proceed south of the rail trail prior to completion of the outfall trunk sewer alignment in 91-125 Coriyule Rd, a temporary sewer eduction tank could be considered in the north-west corner of the Subject site as presented in **Figure 7**, subject to Barwon Water approval.

Internal to the Subject site the development would be serviced by DN150 reticulation sewers constructed to MRWA and Barwon Water standards.

Figure 7: Outfall Sewer Alignment – Invert Levels

2.5 Potable Water

Barwon Water is the responsible authority for the provision of potable water reticulation to service the Subject Site. They have provided preliminary advice in relation to servicing requirements for the subject site, pending formal execution of a Developer Deed.

The preliminary servicing information supplied by Barwon Water has been included in **Appendix A**.

Barwon Water have advised that a DN450 Potable Water Feeder Main is required to connect the Clifton Springs Tank located in McKiernan Street to the existing DN300 Water main located in Tivoli Drive on the north side of the Bellarine Rail Trail. The indicative alignment can be seen presented in **Figure 8**. The location of the proposed connection point in Tivoli Drive can be seen presented in **Figure 9**.

Barwon Water have noted that the portion of the Feeder Main which passes through the development site may be best delivered as a Developer Works project, so that timing and location of the installation can be managed by the Developer. These works would be reimbursed by Barwon Water under the Developer Deed agreement.

It should be noted that land above the 65m-70m contour should be serviced from the existing Water Main located in the north verge of Portarlington Road, which is part of the Clifton Springs Pressure Boosted System. Refer to the area of developed which is situated above 65 m AHD in **Figure 10**.

The remainder of the land (below 65 m contour) will be serviced via the gravity system and will be facilitated by a connection to the DN450 Feeder Main. DN100 and DN150 water mains would then be constructed to service the Development.

Figure 8: Proposed DN450 Potable Water Feeder Main Alignment (Previous Development Layout)

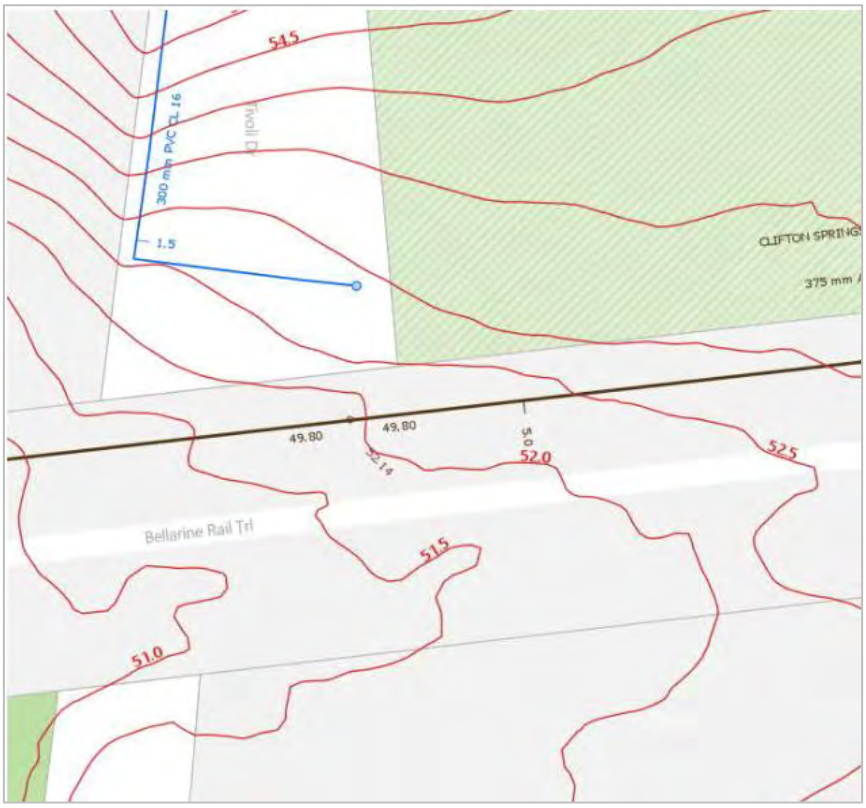

Figure 9: Location of DN450 Feeder Main connection to existing main (Blue Line) in Tivoli Drive

Figure 10: Land above 65 m AHD – proposed to be connected to ex. main in Portarlington Rd (Purple Line)

2.6 Electrical Infrastructure

Powercor is the responsible authority for the provision of electrical supply facilities to service the Subject Site. Power supply exists in Tivoli Drive, Portarlington Road and Jetty Road. It is expected that the existing electrical distribution network can be augmented and extended to service the Subject Site.

The supply connection point for the site will be confirmed by Powercor at the time of submission of a formal request for supply. It is expected that the supply will come from the existing overhead network located on the south side of Portarlington Road or the west side of Jetty Road, depending on where the first development front is located.

Tivoli Drive contains an existing underground low voltage electrical supply which services the public lighting on the east side of the road. It is expected that relocation of sections of the public lighting will be required to facilitate the widening of Tivoli Drive.

In line with other properties in this area, this development will be classified as an Urban Residential Development (URD) where each lot will be supplied with an underground electrical supply at the developer's cost.

2.7 Telecommunications

Existing telecommunication assets in the vicinity of the Subject Site include Telstra and NBN Co. ducts and pits. NBN Co. assets are present within the road reserve of Portarlington Rd and Jetty Road and are considered to be suitably available to service the Subject Site. Opticomm assists are available within the adjacent Curlewis Parks estate on the north side of the Bellarine Rail Trail.

Under amendments enacted in 2011 to the Federal Telecommunications Act (1997), developers of new residential estates and multi-dwelling units are required to enter into an agreement with a provider to design, build and operate a wholesale telecommunications network. Developers have a choice of network providers as long as they comply with Part 7 & 8 of the Act. If NBN Co is selected as a provider, the developer will be responsible for the design and construction of a "fibre-ready" pit and pipe network for NBN Co to utilise.

2.8 Gas Infrastructure

AusNet Services Pty Ltd is the principal authority responsible for the provision of gas to service the Subject Site.

An existing DN250 gas supply service is located on the north side of Portarlington Road. It is expected that the subject site will have the ability to make connection to this gas main and extend supply mains down Tivoli Drive and Harkness Place to service the development, subject to approval of Ausnet Services.

The existing Ausnet Services gas reticulation in proximity to the subject site is presented in **Figure 11**.

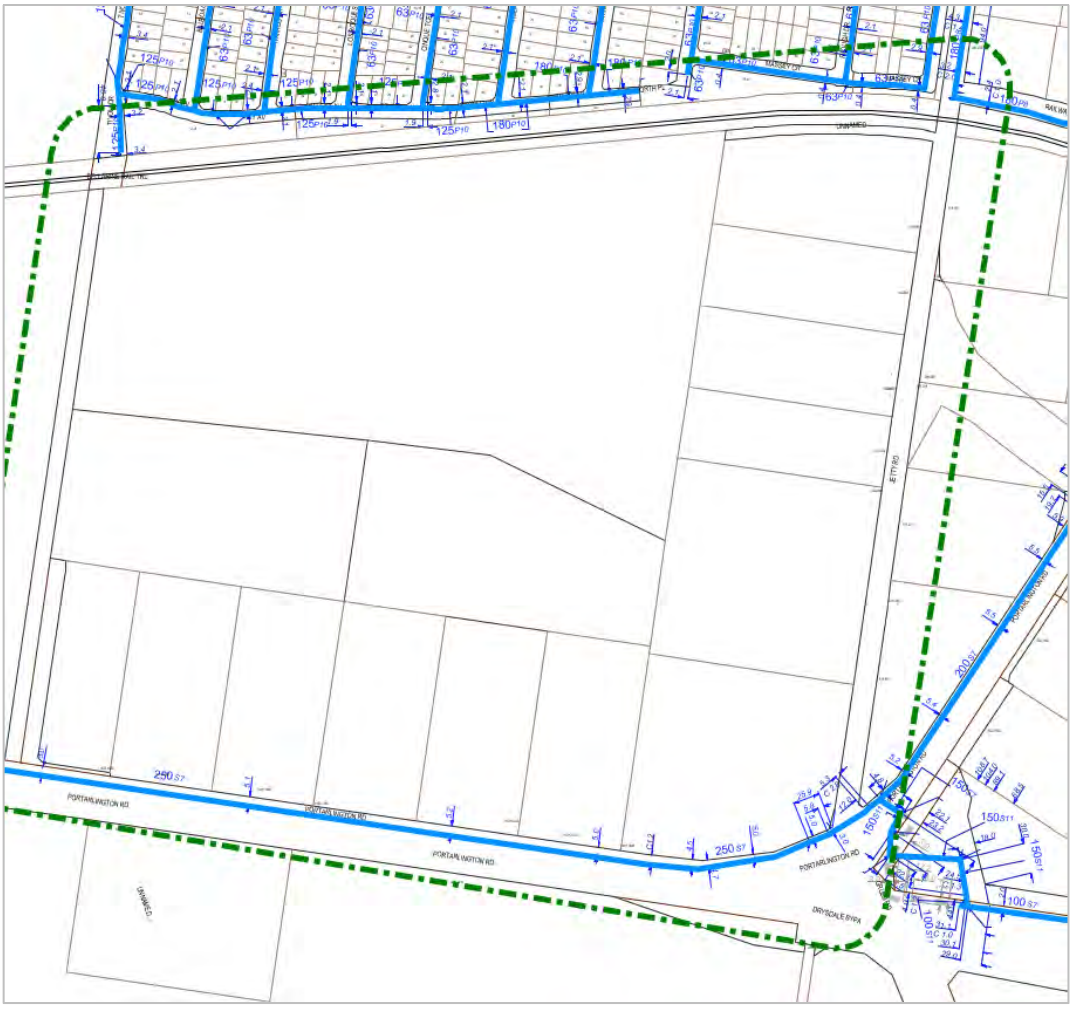

Figure 11: Existing Gas Network

3 Disclosure

Based on SMEC's experience and the investigations carried out, SMEC believes that the Subject Site has availability and access to services required for this type of urban residential development, subject to authority approvals. The extension/augmentation of any services would be in accordance with the standard development process.

The information in this report is preliminary and has been obtained as a result of informal discussions with officers from the relevant authorities and review of MOCS information.

The information supplied by SMEC is subject to change pending official advice from the service authorities, detailed property investigation, detailed design and survey. The information is current to the report date, however, SMEC cannot accept responsibility if any authority changes its requirements after the date of this report.

Appendix A Barwon Water Preliminary Servicing Advice

Our Ref: L013220
Enquiries to: Natalie Clifford - Ph: 1300 656 007

13 July 2021

SMEC
By Email: Tom.Moorfoot@smec.com

Dear Sir,

PRELIMINARY SERVICING ADVICE

RE: PS616244M - 1421-1423 Portarlington Road Curlewis
Servicing advice request as of 07 July 2021

We refer to your request for servicing advice regarding the above specified land.

Please note that this is just preliminary advice based on the information you provided to Barwon Water.

Any information given in this preliminary servicing advice or otherwise by BW is not binding upon BW and you shall not undertake any commitment based on any information given until a formal execution of a Developer Deed or a Private Works Deed.

Preliminary advice is as follows:

Potable Water

The alignment of the proposed DN450 main through this development area can be refined through discussions with CoGG and the developer. Given the pipe size, larger road reserves would be preferable. The DN450 needs to extend back to the Clifton Springs Tank in McKiernan Street for full servicing, so timing and location of the asset needs to be well understood through the development as is reflected in the servicing assessment. It may be that the section of DN450 through the development is best delivered as a developer works project with the rest of the civil works in the estate, like has been done by developers in areas of Armstrong Creek (i.e. Airport Rd). Barwon Water can do the rest outside of the growth area in our own time. It will certainly require further conversation. An indicative alignment is shown below but this is quite flexible and can be determined through discussions with the developers

Sewer

DN300/225 sewer mains are required to be installed remote from the site to service this land. The sewer runs through land identified in the Jetty Road UGP for future residential development. The alignment of this sewer will largely be based on contours, however it should consider future requirements of this area, including drainage, roads etc. Any information available on this site would assist the design of this sewer. The timing of the installation of this asset will need to be refined post amendment.

If you have any questions regarding this letter please contact Barwon Water's representative listed above.

Yours sincerely,

Manager Enterprise Project Delivery

Appendix B **Stormwater Management Strategy (SWMS)**

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Report

Jetty Road South of Rail Trail SWMS

APD Projects Pty Ltd

15 June 2022

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1 INTRODUCTION

This report sets out a recommended Stormwater Management Strategy (SWMS) for a proposed residential subdivision of the land encompassing a number of parcels including 1421-1423 & 1479 Portalington Road and 10 & 12-18 Hackwell Place, Curlewis. A SWMS is required as a part of the rezoning application to meet the requirements of Section 96A of Planning and Environment Act 1987.

The SWMS sets out a concept design to manage stormwater runoff from the proposed development to meet infrastructure needs in accordance with the Infrastructure Design Manual and requirements of the City of Greater Geelong (the City).

The Corangamite CMA (CCMA) have identified the site as having a designated waterway on site which requires an assessment of the waterway including flood modelling.

1.1 Objective

The key objective of the SWMS is to prepare conceptual drainage layout for the site, with concept design of the drainage infrastructure, namely:

- A constructed waterway running from the south-east to north-west of the site;
- Water quality assets to meet Best Practice objectives; and
- Retardation assets, where required, to retard post-development flows to pre-development conditions.
- Opportunities to reduce the overall volume runoff through the integration with the Clifton Springs Drysdale Integrated Water Management Plan.

2 BACKGROUND

The proposed development is located between Portarlington Road to the south, Hackwell Place (formerly Jetty Road) to the east, Tivoli Drive to the West and the Bellarine Rail Trail to the North as shown in Figure 2-1. The site assessed within this project has an area of 52.9 ha and is part of a broader development known as Jetty Road Stage 2 and forms part of the broader Jetty Road Urban Growth Area. The Jetty Road Urban Growth Plan developed in 2008[1] (Figure 2-2) identifies the site for residential development. This report has been prepared at the same time as an overall SWMS for the Jetty Road Stage 2 area.

The main access into the site has been identified as from Tivoli Drive. The site is part of a City of Greater Geelong (The City's) designated catchment (Hermsley Rd/Scarborough Rd catchment) which has been subject to changes in the hydrology and hydraulic flow through the construction of the Drysdale Bypass and other works in the external upstream catchment.

[1] City of Greater Geelong (CoGG)– Jetty Road Master Plan 2008

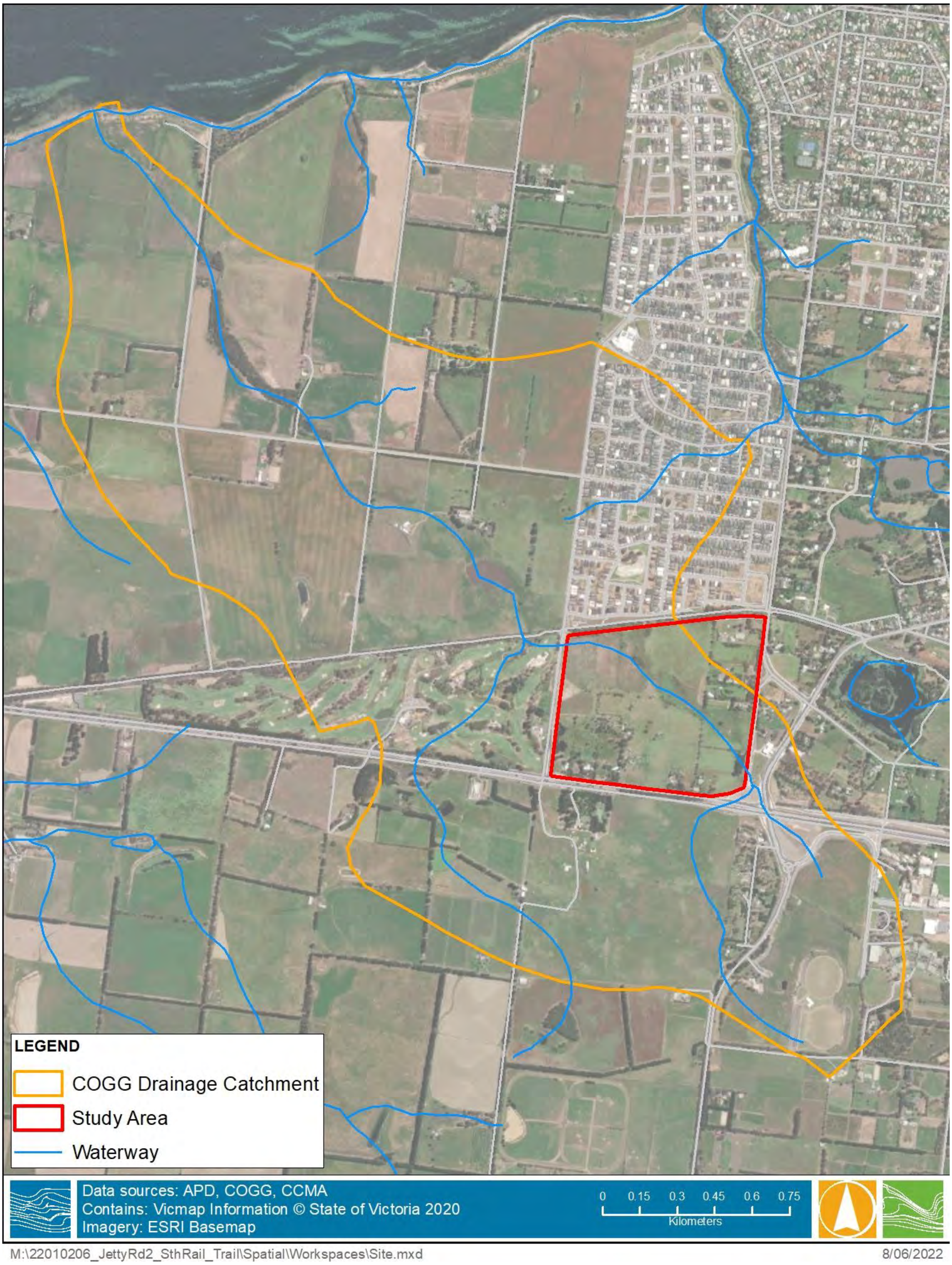

FIGURE 2-1 SUBJECT SITE

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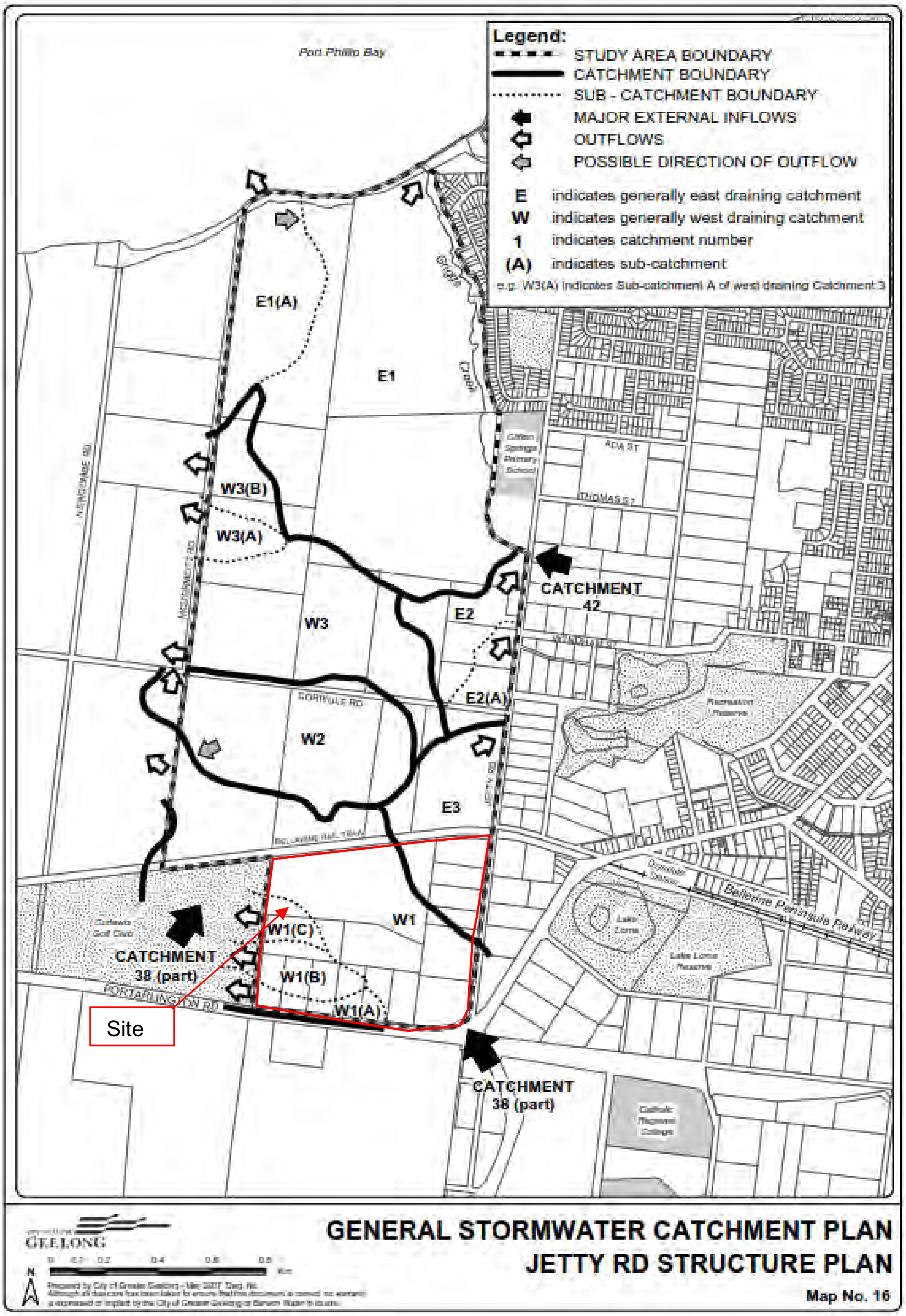

FIGURE 2-2 JETTY ROAD GROWTH AREA CATCHMENT PLAN

2.1 Proposed Development

The proposed development is for a residential subdivision. An earlier development concept plan produced by TGM Cardno is shown in Figure 2-3.

FIGURE 2-3 A SUPERSEDED CONCEPT PLAN (SUPPLIED BY CARDNO TGM, OCTOBER 2021)

It must be noted that the proposed concept site layout may change as the development progresses. Provided that the overall density and layout are not significantly altered, minor revisions are not likely to impact the drainage and water quality concept design presented in this report.

2.2 Existing Waterways

There is one designated waterway within the site (Figure 2-1). The CCMA requires for this waterway to be protected, and where possible, enhanced as part of the residential development. A request for flood advice provided by the CCMA in February 2022 (CCMA-F-2021-01539), provided the following information on the status of the designated waterway:

- The CMA notes the proposed development plan includes open space around the waterway, which is supported based on the assumption that the waterway itself will remain in a naturalised state (i.e. not piped).
- The stream order and setback requirements will be guided by the Melbourne Water publication Waterway Corridors: Guidelines for greenfield development areas within the Port Philip and Westernport Region. The designated waterway shown passing through 91-125 Coriyule Road has a Strahler Order of 2. In accordance with the guidelines, order 1 and 2 streams require a setback of 20 metres from each bank. As noted during the site visit, the banks of the waterway are not clearly defined. The Authority therefore recommends following the hydraulic width method from the guidelines, whereby the "top of bank", or reference point, is set by the 1% AEP flood extent.; and
- Any works within, above or below the bed and banks of a designated waterway require a Works on Waterways Permit from the CMA prior to commencement.

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3 FLOOD MODELLING ASSESSMENT

3.1 Overview

An existing hydraulic model was developed by TGM as part of the Jetty Road precinct structure plan works. The hydraulic model relied on hydrology also developed as part of the same project. The hydraulic model and associated hydrology inflows were provided to Water Technology for the establishment of existing conditions as part of the flood modelling assessment for this site.

3.2 External Catchment Flows

The City of Greater Geelong drainage catchment delineation suggests the catchment upstream of the site is around 94ha. A revision of the upstream catchment delineation based on detailed LIDAR data suggests the entire catchment is around 82ha. The Portarlington Road acts as a hydraulic control upstream of the site and significant works associated with the Drysdale Bypass have altered the hydrology of the catchment.

The flood modelling report prepared by Jacobs[2] in 2019 details the hydrology and detention calculations regarding the overland flow paths prior to and post construction of the bypass. Based on a review of the Drysdale Bypass Flood Mapping[2] outputs (Figure 3-1), available topography (Figure 3-2), the upstream catchment size has changed and the addition of significant storage within the following areas:

- Wetland at Grubbs Road,
- Basin downstream of St Thomas Catholic School and
- The Basin at Jetty Road.

The inclusion of additional storage into the catchment and a limiting outlet pipe from the Basin at Jetty Road (525mm diameter RCP) has reduced the overland flow being routed through the catchment towards the site in a 1% AEP event. Flow from the Jetty Road basin discharges from the basin outlet through an existing pipe network and under Jetty Road via 2x 600x450mm box culverts adjacent to the existing farm dam at Property 23. The drainage infrastructure upgraded as part of the Bypass works are shown in Figure 3-3.

Ultimately the inflow into the site is controlled by the piped outlet from the recently constructed retarding basins, where a 2 x 600mm x 450mm box culvert conveys flow from the basin into the waterway alignment upstream of the site (Jetty Road) shown in Figure 3-4. The Jacobs report[2] indicates a peak flow from the Jetty Road Basin outlet pipe of 0.23 m^3/s in the 1% AEP event with a 50% blockage and 0.50 m^3/s with 0% blockage. A constant inflow of 0.50 m^3/s (for the 1% AEP event) and 0.2 m^3/s (10% AEP event) at the piped outlet under Jetty Road adjacent to the existing farm dam has been included in the hydraulic.

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[2] Drysdale Bypass Detailed Design – DP03 Drainage Design Report prepared by Jacobs for Decmil and MRPV, 2019

FIGURE 3-1 FLOOD MODELLING – POST DRYSDALE BYPASS WORKS (JACOBS FLOOD MODELLING[2])

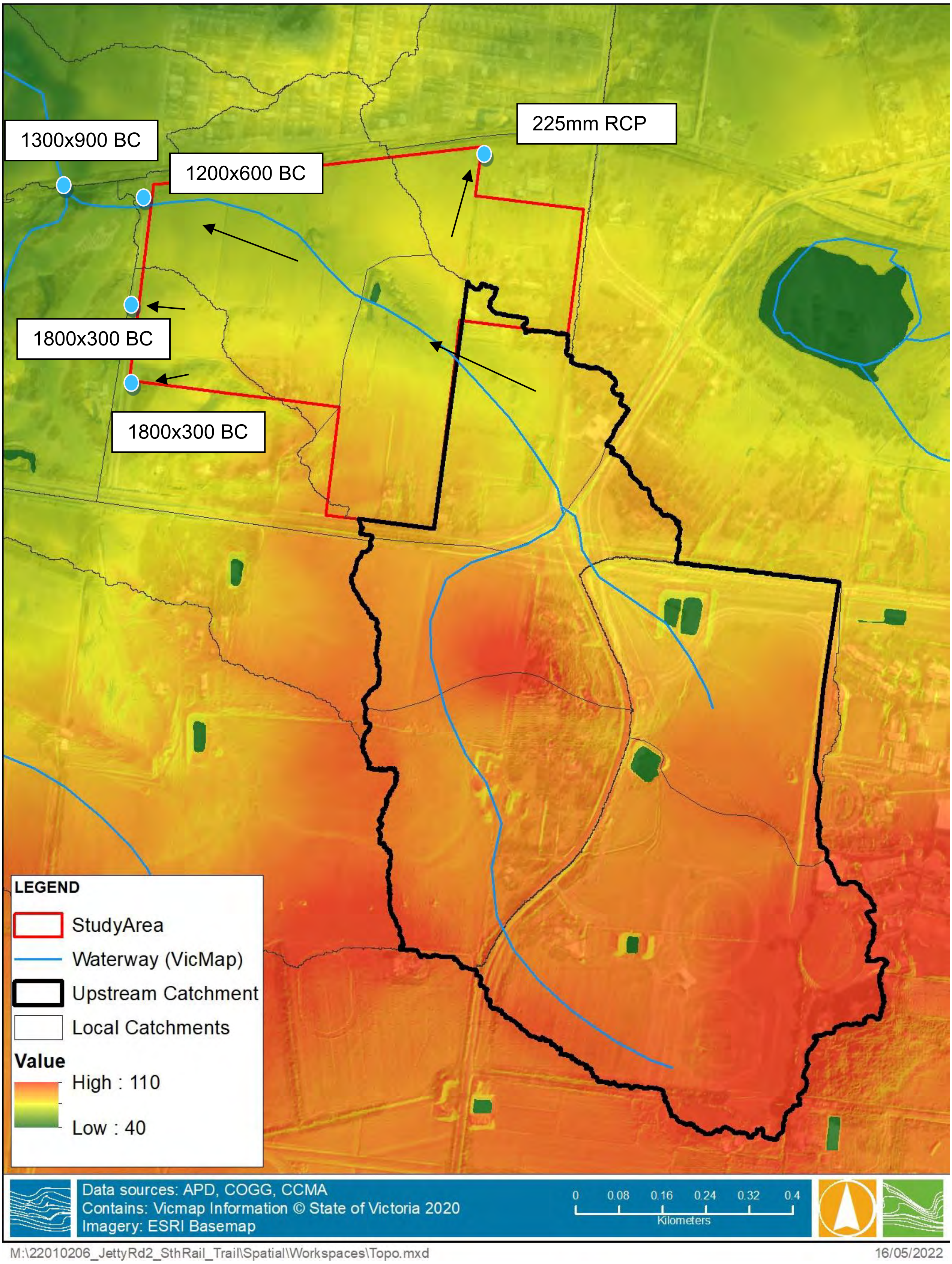

FIGURE 3-2 UPSTREAM CATCHMENT (LIDAR FLOW PRIOR TO DRYSDALE BYPASS COMPLETION)

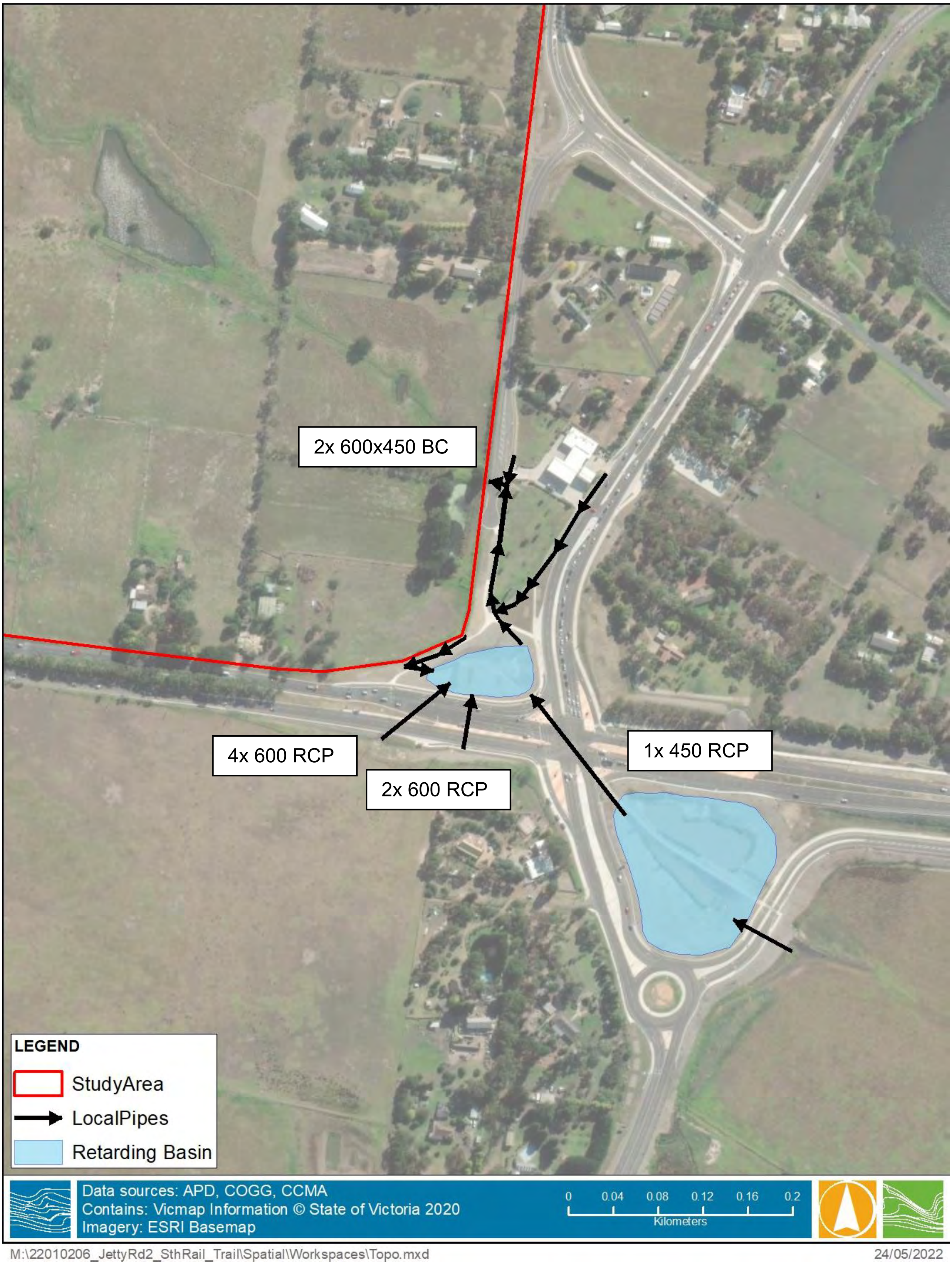

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FIGURE 3-3 DRYSDALE BYPASS CULVERTS AND BASINS

FIGURE 3-4 BYPASS BASIN OUTLET INTO THE WATERWAY AT JETTY ROAD

3.3 TUFLOW Modifications

The TUFLOW model developed as part of the Jetty Rd Rezoning – Stage 2 Flood Study[3] was modified to include feature survey of the site capture by SMEC in 2022. Other changes included the direct inflow representing external catchment upstream of Jetty Road as discussed above as well as changes to culverts at Tivoli Drive and under the rail-trail downstream of Tivoli Drive.

3.4 Model Simulation

The hydraulic model was simulated for the 10% and 1% AEP event for a range of durations from 15-minute to 6-hour. The results were then spliced to produce the maximum depth for existing conditions. The maximum depth results (Figure 3-5) show the 1% AEP depths generally less than 300mm through the site, with the exception of the location of the two existing dams. The flow path along the waterway is relatively confined due to the topography of the site.

Floodwaters are shown as overtopping Tivoli Drive in two locations in the 1% AEP flood event and minor flows break out of the existing open channel located downstream of Tivoli Drive before flows are conveyed under the rail reserve via a single 1200x900mm box culvert adjacent to a dam within the Curlewis Golf Course.

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[3] Jetty Rd Rezoning – Stage 2 Flood Study – TGM, prepared for Curlewis Bellarine Pty Ltd version 6, June 2020.

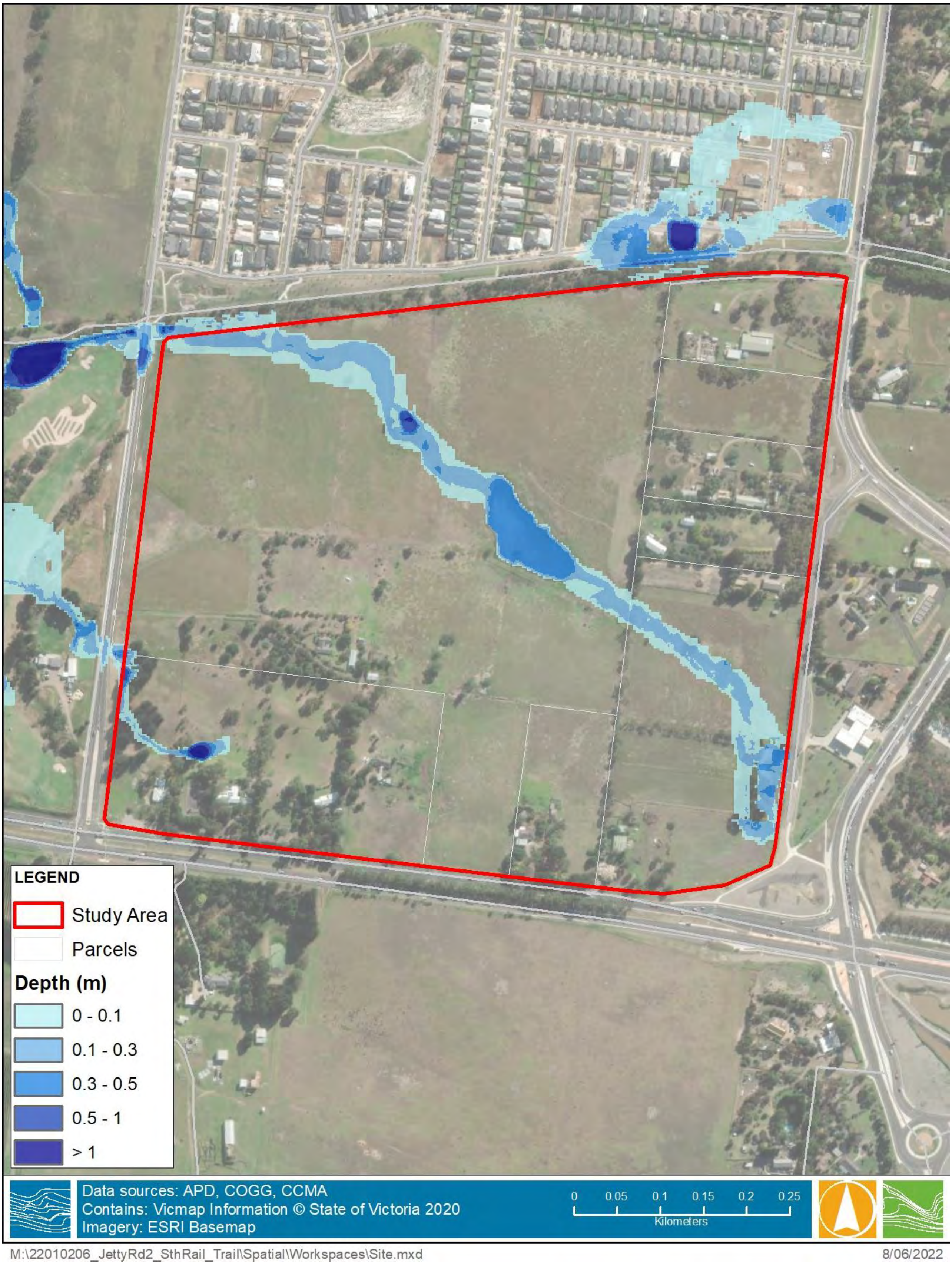

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FIGURE 3-5 1% AEP FLOOD DEPTH – EXISTING CONDITIONS

FIGURE 3-6 10% AEP FLOOD DEPTH – EXISTING CONDITIONS

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3.5 Existing Overland Flow Paths

As the current Cardno Hydraulic model was constructed using a lumped hydrology methodology, an additional rain-on-grid model was constructed as a validation of overland flow paths. The 1% AEP, 2-Hour duration event were simulated with the depth results shown below (Figure 3-7). This highlights there are no major overland flow paths that are not included in the existing TGM modelling.

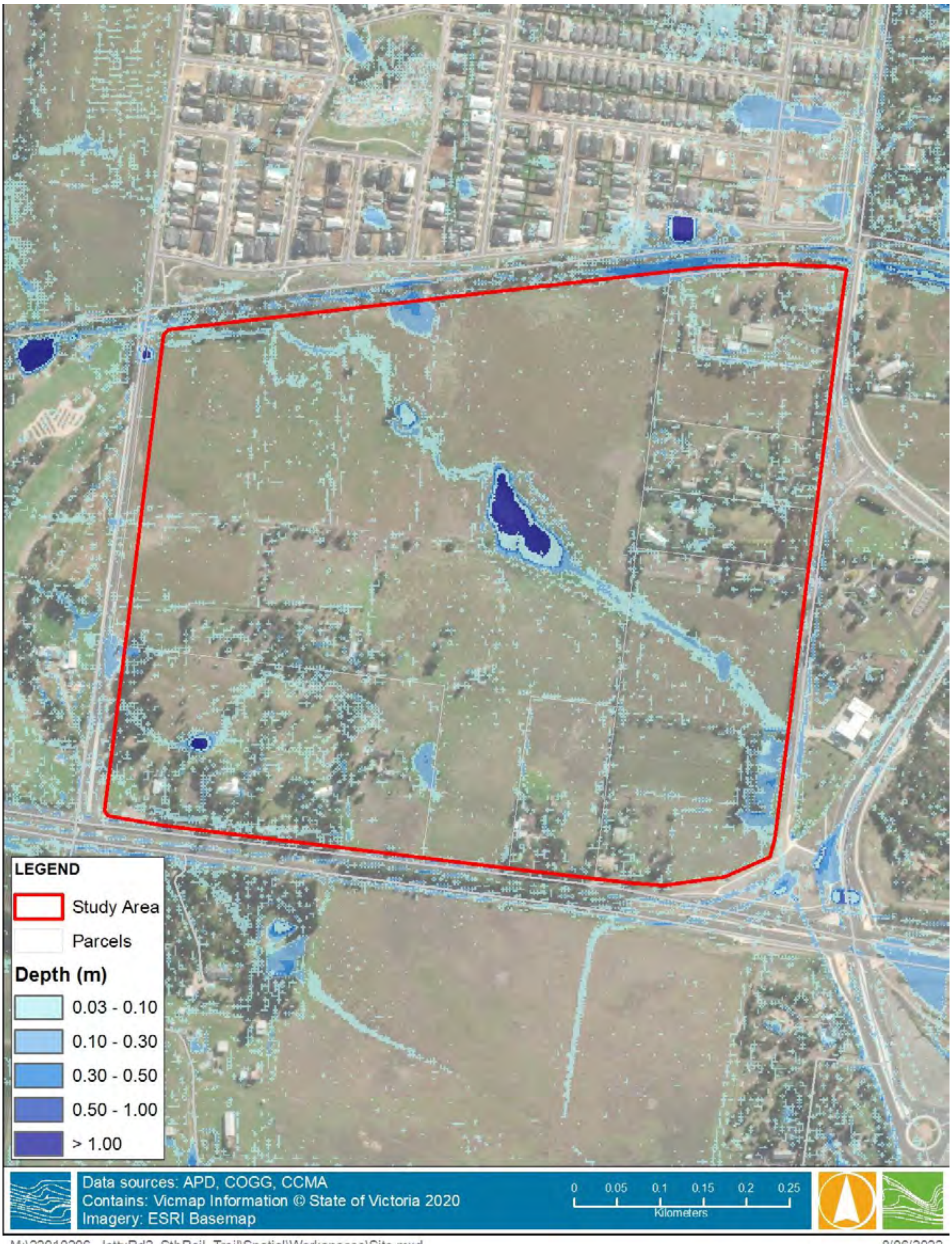

FIGURE 3-7 1% AEP RAIN ON GRID DEPTH RESULTS

3.6 Developed Conditions

Developed conditions flood modelling is to be undertaken at a later stage of design once a developed surface becomes available. This will assess the overall impact and provide further checks on flood depths and velocities within the site.

As per the CCMA flood advice:

"Post developed flood mapping has not been provided. The proposed removal of the dam on site has the potential to influence flood levels, velocities and depths in the waterway which traverses private property downstream."

As an interim, the RORB modelling undertaken has shown that the peak flow rate leaving the site can be controlled for the 1% AEP event and a staged outlet option to consider flow rates in a lower magnitude event (10% AEP).

4 STORMWATER MANAGEMENT

This section of the SWMS details internal drainage infrastructure. Its objective is to guide drainage design for the site to provide for the collection, treatment and disposal of stormwater runoff in an environmentally acceptable manner within the subdivision layout, consistent with applicable guidelines and standards and including the implementation of best practice water quality measures.

4.1 Legal Points of Discharge

Under existing conditions, the Subject site has four natural outfalls. The largest of these is in the north-west of the site where the waterway is conveyed under Tivoli Drive into an open drain within the Bellarine Rail Trail adjacent to the Curlewis Golf Course. This discharge then continues north under the Bellarine Rail Trail via a large grated pit and twin box culvert arrangement. The other drainage outfalls are relatively minor with a catchment delineation plan shown in Figure 4-1. The majority of the site area (70%) drains through the outfall located at the north-western corner of the subject site. The remaining outfalls drain the remaining area to Tivoli Drive via two culverts into the Curlewis Golf Course or to the Bellarine Rail Trail (north-east portion of the site). The strategy proposed is to convey all stormwater runoff and external catchment flow via culverts under Tivoli Drive in the northwest of the site to an existing drain along the Bellarine Rail Trail. This in essence removes the need for the smaller outlets.

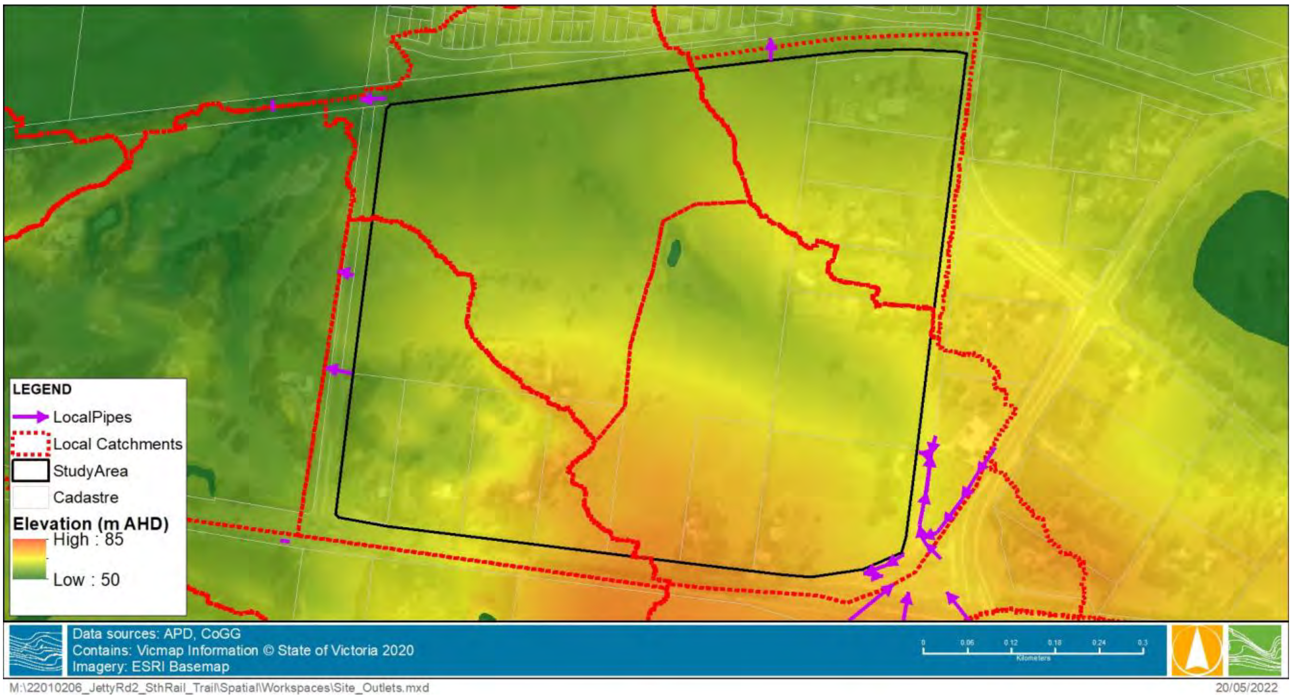

FIGURE 4-1 DRAINAGE OUTFALLS

4.2 Hydrological Analysis

The catchment runoff routing model, RORB, has been used for the hydrologic analysis of this study. RORB is a non-linear rainfall runoff and stream flow routing model for calculation of flow hydrographs in drainage and stream networks. The model was developed in accordance with the latest Australian Rainfall and Runoff (ARR2019) rainfall datasets and guidelines. The RORB modelling assesses the impacts of the proposed site development on the peak flows flow leaving the site and associated volume of runoff.

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4.2.1 Pre-Development Validation

A pre-development RORB model was constructed as part of the Jetty Road Rezoning - Stage 2 SWMS Project (Water Technology 2022a)[4]. The pre-developed condition represents 100% natural/rural catchment. This scenario was modelled to validate initial and continuing losses and the RORB routing parameter, Kc against the Regional Flood Frequency Estimate (RFFE). The total catchment area is 16.1 km^2. Fraction Impervious (FI) of all sub-catchments was set to zero and all reaches in RORB model was modelled as natural to reflect the 100% rural nature for the catchment.

- The initial and continuing losses as in Table 4-1 were used in the modelling, in accordance with ARR2019 (Book 5, Chapter 3.5.3). The pervious area losses were extracted from the ARR2019 Data Hub.

TABLE 4-1 INITIAL AND CONTINUING LOSSES

	Initial Loss (mm)	Continuing Loss (mm/hr)
PA	16.80	3.00

  - The Data Hub rural losses (19 mm and 3 mm/hour) were selected for the initial and continuing loss respectively for the catchment area. The initial loss from the Data Hub is for complete storms rather than bursts. As the rainfall modelled in RORB are bursts, rural initial losses are reduced by the pre-burst rainfall corresponding to the median pre-burst rainfall depth (mm) from the Data Hub. The median pre-burst rainfall depth (mm) varies across the storm durations 2.2 mm pre-burst rainfall depth was selected and subtracted from the rural initial loss (19 mm) to derive the burst PA initial loss (19 mm – 2.2 mm = 16.8 mm).
  - The rural continuing loss (3 mm/hour) is based on a 1-hour timestep. In accordance with ARR 2019 Book 5, Chapter 3.5.3.2.2, continuous losses should typically range from 0-4 mm/h in south-eastern Australia. As a result, unmodified continuous loss of 3 mm/hour was adopted for PA.
- The Kc routing parameter value was derived using regional equations in accordance with the Melbourne Water Flood Mapping Projects Guidelines and Technical specifications[5]. Total catchment area (A) was 16.08 km^2 and d_{av} (average routing distance) was calculated as 2.94 km.
- The pre-developed conditions RORB model was run using the ARR2019 Intensity-Frequency-Duration (IFD) data and the selected Kc (routing parameter) values for the 1% AEP event across the ensemble of temporal patterns and a range of storm durations from 10 min to 72-hour.
- The 1% AEP median peak flow at the catchment outlet was compared against the RFFE (Table 4-2). Equation No. 4 in was chosen as the method for deriving routing parameter as it provides the closest flow rate to the RFFE.

TABLE 4-2 KC VALUES AND COMPARISON OF FLOW ESTIMATES

Equation No.	Regional Equation	Kc	1% AEP Flow
1	$k_c = 0.49 \times A^{0.65}$	2.98	33.5
3	$k_c = 2.2 \times A^{0.5}$	8.82	15.2
4	$k_c = 1.25 \times d_{av}$	3.68	26.9
RFFE			29.1

[4] Jetty Road Rezoning - Stage 2 SWMS, prepared by Water Technology for Curlewis Bellarine Pty Ltd
[5] Melbourne Water Flood Mapping Projects Guidelines and Technical specifications (July 2020)

Models were developed and run for existing and developed conditions based on these parameters to determine the flooding mechanisms across the site and to size the retarding basin. Details of the RORB modelling are provided in the following sections.

4.2.2 Existing Conditions

The hydrology model was then used to represent the existing conditions across the catchment and determine the pre-development flow rate leaving the site. The sub-catchments were delineated using 2012/13 LiDAR data. The catchment plan of the existing condition RORB model is shown in (Figure 4-2).

FIGURE 4-2 EXISTING CONDITIONS RORB CATCHMENT PLAN

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To reflect the existing conditions of the site, the following approach was adopted:

- The initial and continuing losses as in Table 4-1 were used in the modelling, in accordance with ARR2019 (Book 5, Chapter 3.5.3). The pervious area losses were extracted from the ARR2019 Data Hub.

TABLE 4-3 INITIAL AND CONTINUING LOSSES

	Initial Loss (mm)	Continuing Loss (mm/hr)
EIA	1.50	0.00
ICA	13.30	2.50
PA	16.80	3.00

- The catchment impervious fractions for different surface types were determined. Firstly, Total Impervious Area (TIA) fraction values were determined for each land use type based on typical fraction impervious values outlined in the Melbourne Water MUSIC Guidelines (2018).
- Following this the catchment fractions were determined for the following three urban surface[5 & 6] types:
  - Effective Impervious Area (EIA)
    - EIA = 0.6 * TIA (ARR2016, Book 5, Chapter 3.4.2.2.2) when TIA ≤ 80%
    - EIA = 0.6 to 1.0 * TIA when TIA >80% as per EIA/TIA ration relationship presented in Figure 4-3.

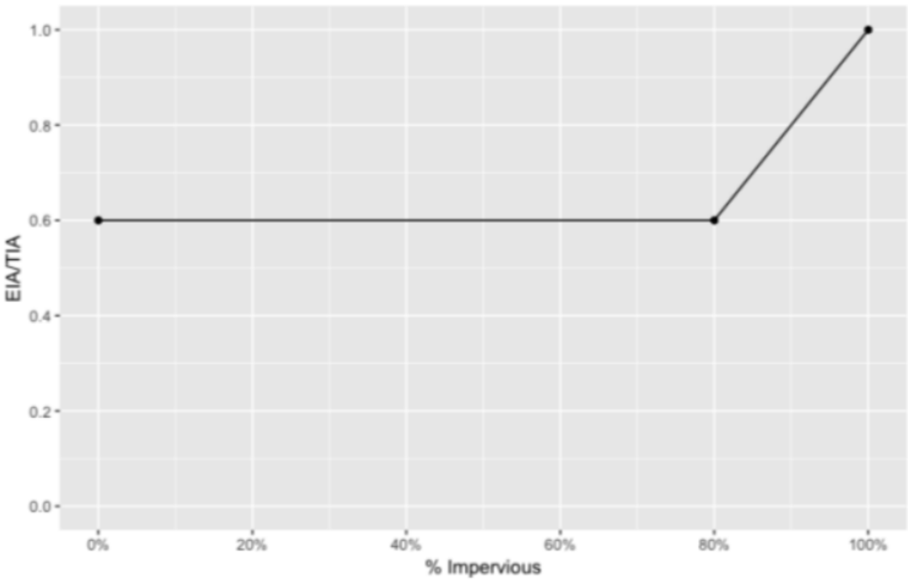

FIGURE 4-3 EIA/TIA RATIO INCREASE FOR HIGHLY IMPERVIOUS CATCHMENT

  - Pervious Area (PA)
    - PA = 1 – TIA
  - Indirectly Connected Area (ICA)
    - ICA = 1 – PA - EIA

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[6] Australian Rainfall and Runoff Guidelines, 2019.

- Five different reach types are available in RORB (1 = natural, 2= excavated & unlined, 3= lined channel or pipe, 4= drowned reach, 5= dummy reach). The reach types in the RORB model were set as followed:
  - "natural" for runoff being conveyed through non-formalised drainage paths (i.e., grassed slope without channel);
  - "excavated & unlined" for runoff being conveyed via open channel (grassed or earthen); and
  - "Lined channel or pipe" for runoff being conveyed from the residential lots subarea (roof/gutter/downpipes).
- The existing dam located within the site was not included in the existing condition RORB model.
- The catchment upstream of the Jetty Road is around 81 hectares (0.3581km^2).
- Five existing retarding basins within the Bellaview Estate and Curlewis Parks Estate along with two upstream basins next to the of Drysdale bypass were included in the existing conditions RORB model.
  - The stage-storage-discharge relationships of the five basins within Bellaview Estate and Curlewis Parks Estate were derived using the basin parameters extracted from the existing conditions flood study (Cardno TGM, 2020)[7] XP-STORM model.
  - In absence of stage-storage information of the two basins controlling the flow from the external catchment upstream of the Outfall 1 (Drysdale Bypass Basins), aerial imagery and the Drysdale Bypass Report (MRPV, 2019[5]) were used to estimate the storage volume. The estimated volume of two basins (modelled as storages B1 and B2) was estimated as 59,000 m^3 and 18,000 m^3. The diameter of the outlet pipes of the two basins were set as 450 mm and 525 mm for B1 and B2 respectively (MRPV, 2019)[8]
- The existing conditions RORB model was run using the ARR2019 IFD (Intensity Frequency Duration) data and the selected Kc routing parameter (3.60 with new d_{av} of 2.88 under existing conditions) value for the 1% AEP event across the ensemble of temporal patterns and a range of durations (from 10 min to 72-hour duration).
- The 1% AEP median peak flow at locations throughout the site were compared against the previous studies (Table 4-4). Modelled results are generally comparable at the existing basin outlets. A notable difference was observed downstream of Outfall 1 (Tivoli Drive). It should be noted that the previous study (Cardno TGM, 2020), did not include the two upstream basins controlling the external catchment in their modelling (basin construction was completed after the modelling) and is likely the reason for observed discrepancy in modelled 1% AEP flows.

TABLE 4-4 EXISTING CONDITION 1% AEP PEAK FLOW COMPARISON (MEDIAN Q1%AEP)

Location	1% AEP Peak Flow		Reference
	Current Study	Previous Studies	
External catchment Inflow at Jetty Road (downstream of B1 &B2)	0.81	0.5 – 1.0	(Jacobs 2019)
Outfall at Tivoli Dr	1.88	3.29*	(Cardno TGM, 2020)
Existing SMEC North basin (outside study area)	1.00	0.84	
Existing Greenvale basin outflow (outside study area)	1.36	1.39	

[7] Cardno TGM (June 2020), Jetty Rd Rezoning - Stage 2, Flood Study, Existing Conditions Report (Version 6)
[8] Drysdale Bypass Detailed Design – DP03 Drainage Design Report prepared by Jacobs for Decmil and MRPV, 2019

* Peak flow of 3.29 m3/s was based on the catchment with no retarding basins at the Drysdale Bypass.2

4.2.3 Developed Conditions

To create the developed conditions RORB model, the overall site stormwater drainage strategy was incorporated into the RORB model. This involved identifying likely post-development drainage catchments and indicative drainage alignments which conveyed all stormwater runoff from the site back to a single outfall located in the northwest of the site.

Changes to the RORB model included:

- New TIA values were updated to account for the increase in imperviousness proposed by the development, in line with Section 4.2.2. EIA and ICA are calculated from the same approach outlined in the existing condition scenario (section 4.2.2).
- Additional sub-catchments in the south-west and north-east of the site were changed to drain back to the constructed waterway.
- The catchment area draining to Outfall 1 (excluding the controlled catchment upstream of Jetty Road) is around 52 hectares ($0.52km^2$).
- Reach types have also been adjusted to the "piped" for all reaches within the development except for reaches along the proposed constructed waterway which were set as "excavated & unlined".
- The Kc (routing parameter) value was updated to 3.59) as the d_{av} changed slightly (2.87) due to addition of new basins in PSP scale RORB model between the existing and developed conditions models.
- The developed conditions RORB model was run using the ARR2019 IFD data (ensemble of temporal patterns and a range of durations (from 10 min to 72-hour duration)).
- Post-development flows were evaluated at Outfall 1, to estimate permissible site discharge and, subsequently, the required 1% AEP retardation storage.
- Following this, the additional retarding basin was included in the RORB model

The catchment plan of the development condition RORB model is shown in Figure 4-4.

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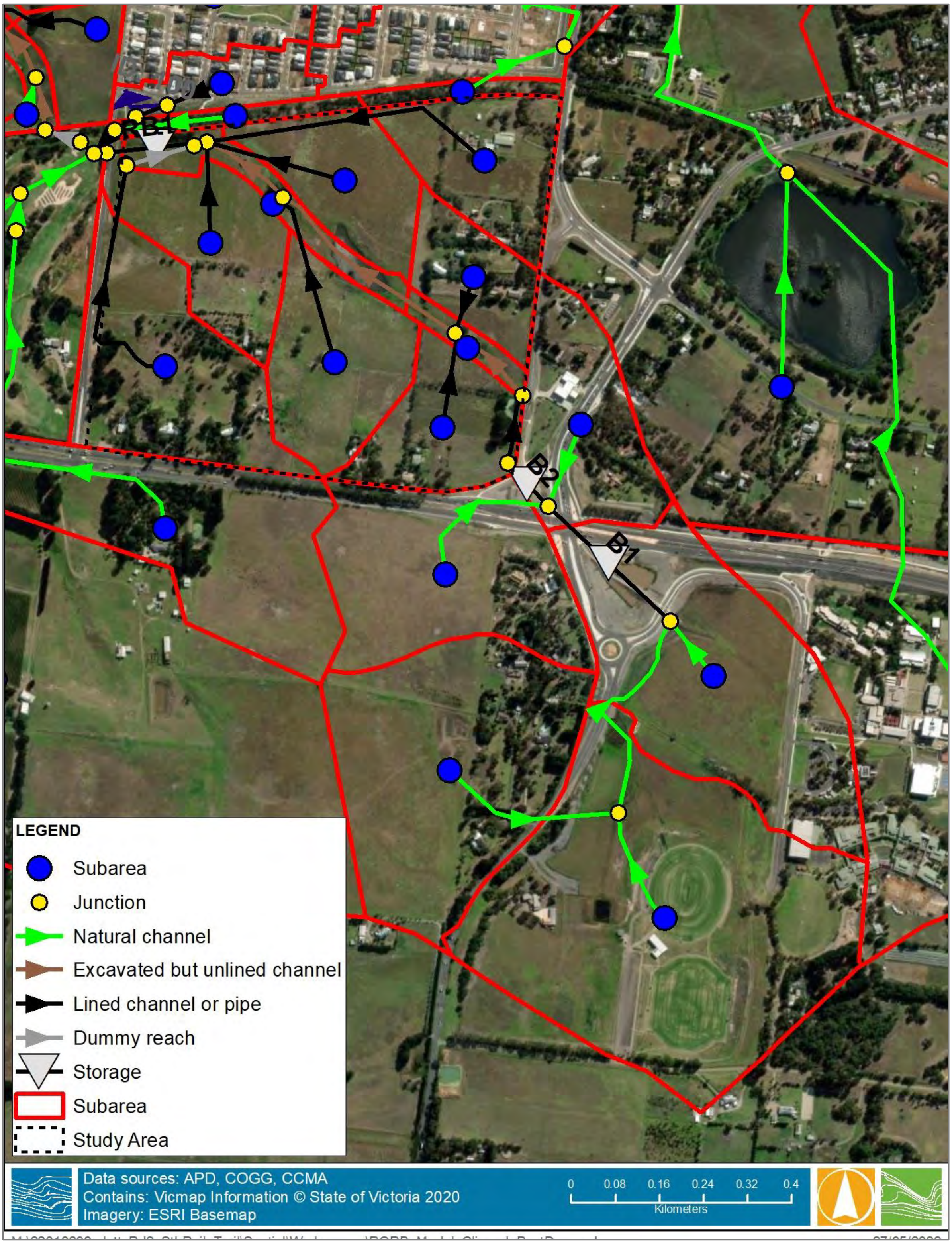

FIGURE 4-4 DEVELOPMENT CONDITIONS RORB MODEL CATCHMENT PLAN

4.3 Retarding Basin

A retarding basin is proposed to service the site and retard development peak flows leaving the site back to existing conditions. Drainage flows will be conveyed to the RB along a constructed waterway through the site. The configurations of the proposed retarding basin required to limit downstream flows to existing pre-development flow rates are shown in Table 4-5. A schematic of the layout is shown in Figure 4-5 and involves a staged outlet incorporating:

- A single 525mm diameter RCP to convey the 10% AEP flow rate;
- A grated overflow pit to provide the balance of flow between the 10% and 1% AEP flow rates
- A single 750mm diameter RCP to convey the 1% AEP flow rate
- A spillway located at a height of 2.00m

TABLE 4-5 RETARDING BASIN DETAILS

Item	Details
Upstream Area (ha)	**134***
Storage Volume (m^3)	12,800
Pre-development 1% rate at RB (m^3/s)	1.88
Peak 1% AEP RB Inflow (m^3/s)	8.67
Peak 1% AEP Outflow (m^3/s)	1.70
Peak 1% AEP Water Depth (m)	1.99
Pre-development 10% rate at RB (m^3/s)	0.83
Peak 10% AEP RB Inflow (m^3/s)	4.14
Peak 10% AEP Outflow (m^3/s)	0.83
Peak 10% AEP Water Depth (m)	1.56

* Catchment significantly controlled due to upstream basins at Jetty Road and the Drysdale Bypass

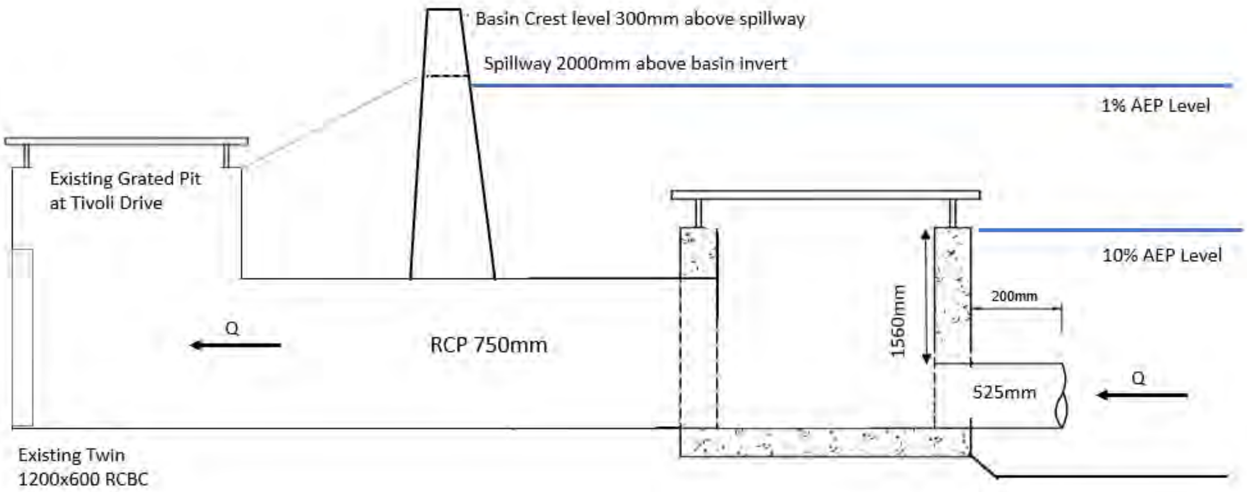

FIGURE 4-5 SCHEMATIC OF BASIN OUTFALL AND PIPED CONNECTION AT TIVOLI DRIVE

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5 WATER QUALITY AND VOLUME MANAGEMENT

The following section of the SWMS details the Water Sensitive Urban Design (WSUD) assets proposed to treat runoff from the development. The water quality treatment targets established by the Urban Stormwater Best Practice Guidelines (CSIRO, 1999) should be achieved as a minimum to protect ecological values within Port Philip Bay.

The load reduction targets for key pollutants are as follows:

- 80% of total suspended sediments (TSS);
- 45% of total nitrogen (TN);
- 45% total phosphorous (TP); and,
- 70% gross pollutants.

Additionally, the post development runoff volume should not exceed the pre-development runoff volume as per the Jetty Road Urban Growth Plan (CoGG, 2008).

The proposed WSUD strategy informed by the current best practice industry methods, City of Greater Geelong MUSIC guidelines[9], and Melbourne Water MUSIC guidelines[10]. In addition to typical stormwater treatment technologies, lot and precinct scale stormwater harvesting and infiltration opportunities to meet the runoff volume management targets were investigated.

MUSIC modelling (Version 6.3) was undertaken to estimate the WSUD asset sizing and investigate runoff volume reduction opportunities. Geelong North 20 year (1971 – 1990) 6-minute MUSIC climate template (available from the City of Greater Geelong MUSIC modelling guidelines) was adopted for the stormwater quality modelling. The MUSIC model schematic is shown in Figure 5-1.

FIGURE 5-1 MUSIC MODEL SCHEMATIC

[9] City of Greater Geelong MUSIC Modelling Guidelines. Available at https://www.geelongaustralia.com.au/idm/documents/item/8cf4f273fe1120f.aspx
[10] Melbourne Water MUSIC modelling guidelines (2018). https://www.melbournewater.com.au/sites/default/files/2018-03/Music-tool-guidelines.pdf
Melbourne Water Constructed Wetland Manual https://www.melbournewater.com.au/planning-and-building/developer-guides-and-resources/standards-and-specifications/constructed-0
Melbourne Water Biofiltration systems in Development Services Schemes guidelines https://www.melbournewater.com.au/media/14586/download

5.1 Sub-catchments

The MUSIC model incorporates the 52.9 ha of land proposed for development south of the rail trail. MUSIC sub-catchment delineation was informed by post-development RORB modelling. Similar to RORB modelling, it was assumed that parts of the Jetty Road Stage 2 development that currently drain through the minor LPoD will be diverted to the main LPoD outfall at the northwest of the stie once fully developed.

Since no hydrologic routing was applied in MUSIC modelling, source nodes (sub-catchments) were represented using a simplified/lumped approach. Catchment FI values were set as per Melbourne Water MUSIC guidelines (i.e., taken as TIA of RORB model subareas). Pervious area soil parameters were adopted as follows:

- Soil Store Capacity = 120 millimetres
- Field Capacity = 50 millimetres

A summary of MUSIC sub-catchments areas and FI values are presented in Table 5-1.

TABLE 5-1 WATER QUALITY CAGTCHMENT SUMMARY

Catchment	Area (ha)	FI (%)
Existing	46.5	20%
Post Developed	52.9	67%[1]

[1]The typical FI for a standard residential development is 75%. A lower FI was used in MUSIC after taking into account waterway corridor and drainage reserves, etc.

5.2 WSUD Assets

12,000 m^2 wetland surface area and 1,800 m^3 of sediment pond volume was found to be meet BPEM guidelines in the Jetty Road Stage 2 SWMS. Using these values, the general layout was then prepared to incorporate three sediment ponds located upstream of the wetland to provide pre-treatment (Refer to Appendix C for sediment pond sizing calculations). WSUD asset performance summary is presented in Table 5-2.

TABLE 5-2 TREATMENT TRAIN EFFECTIVENESS

Pollutant	Inflow Load (kg/y)	Outflow Load (kg/y)	% Load reduction	BPEM met?
Total Suspended Solids	31,700	6,450	80	Yes
Total Phosphorus	64.9	19.7	70	Yes
Total Nitrogen	459	226	51	Yes
Gross Pollutants	7,150	336	95	Yes

5.2.1 Runoff Volume Management

The post development stormwater runoff volume (143 ML/year) is higher than the existing conditions stormwater runoff volume leaving the site (57 ML/year). Therefore, additional measures will be required to reduce post development runoff volume by 86 ML/year. Four options have been assessed as part of the overall Jetty Road Stage 2 SWMS.

Option 1 – Lot Scale Intervention (Rainwater Tanks)

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An investigation to test the contribution for lot-scale rainwater harvesting was undertaken using MUSIC modelling. It was assumed each residential lot will be connected to a 2-kL rainwater tank for toilet flushing as per the Jetty Road Urban Growth Plan Flooding, Drainage and Utility Services Principles and Objectives (Objective 27.1). For modelling purposes, it was assumed that only 50% of the residential roof is connected to the rainwater tank (Refer to Appendix D for more details on rainwater harvesting calculations). Modelling results indicate an average of 13 ML/year could be retained through lot-scale rainwater harvesting.

Option 2 - Lot Scale Intervention (Rainwater Tanks) & Precinct Scale Harvesting & Reuse

As identified in Option 1, there is still significant runoff volume to be managed through other interventions. If the excess runoff volume to be used for open space irrigation, this will be equivalent to irrigating 19 – 27 ha of turfed areas[11]. This option will also require substantial storage to be provided. Curlewis Golf Course could be a potential re-use site. However, it is understood that currently the Curlewis Golf Course use reclaimed water for irrigation of the course. Therefore, this option is not likely practical to manage post-development runoff volume.

Option 3 – Ocean Outfall

Alternatively, there is potential to consider an ocean outfall through the existing council pipe along the Coriyule Road. The feasibility of this option is subject to the ability to pass the excess runoff volume west of Tivoli Drive and the capacity of the council pipe to receive additional flows form the Jetty Road Stage 2 Area. It is noted the given peak flow rates are maintained to existing conditions as assessed in the overall Jetty Road Stage 2 SWMS.

Option 4 – Evaporation Pond

If the excess runoff volume of 86 ML/year to be lost via evaporation, MUISC modelling results indicated that an evaporative basin with ~8 ha surface area is required. This does not appear to be a viable option in the overall development strategy.

It is recommended to pursue Option 3 as the suitable volume management option along with the potential to investigate Options 1 & 2 at a later stage of design.

[11] Based on typical reuse demand of 3.2 and 4.5 ML/ha/year for Warm season turf and Cool season turf (CoGG MUSIC modelling guidelines).

6 WATERWAY/LINEAR WETLAND DESIGN

This section outlines the concept design for the constructed waterway and linear wetland through the site. The existing designated waterway is an ephemeral waterway running from the south-east of the site through to the north-western boundary. The design for this reach of the waterway has been undertaken in line with Melbourne Water's *Constructed Waterway Design Manual* (2019)[12], *Wetland Design Manual*[13] and in response to site constraints.

6.1 Overview

The opportunity to incorporate an improved waterway outcome has resulted in the incorporation of a constructed wetland and waterway within the waterway corridor reserve. This is in line with advice received form the CCMA in February 2022 stating:

"The CMA notes that the proposed development plan includes open space adjacent to the waterway and recommends stormwater reuse opportunities are explored at this location (for example co-locating a basin with open space may allow for reuse of stormwater)"

Based on requirements of the water quality modelling in the Jetty Road Stage 2 SWMS, the following assets are required to meet best practice:

- 12,000 m^2 wetland surface area
- 1,800 m^3 sedimentation pond volume
- 12,800 m^3 retarding basin volume

The proposed layout incorporates a constructed waterway within the upper section of the site before transitioning to a constructed wetland. This is supplemented by three sediment ponds along the waterway corridor to provide pre-treatment of stormwater at runoff outfalls (piped) before entering the wetland (Figure 6-1).

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[12] Melbourne Water, 2019, Constructed Waterway Design Manual
[13] Melbourne Water, 2019, Wetland Design Manual

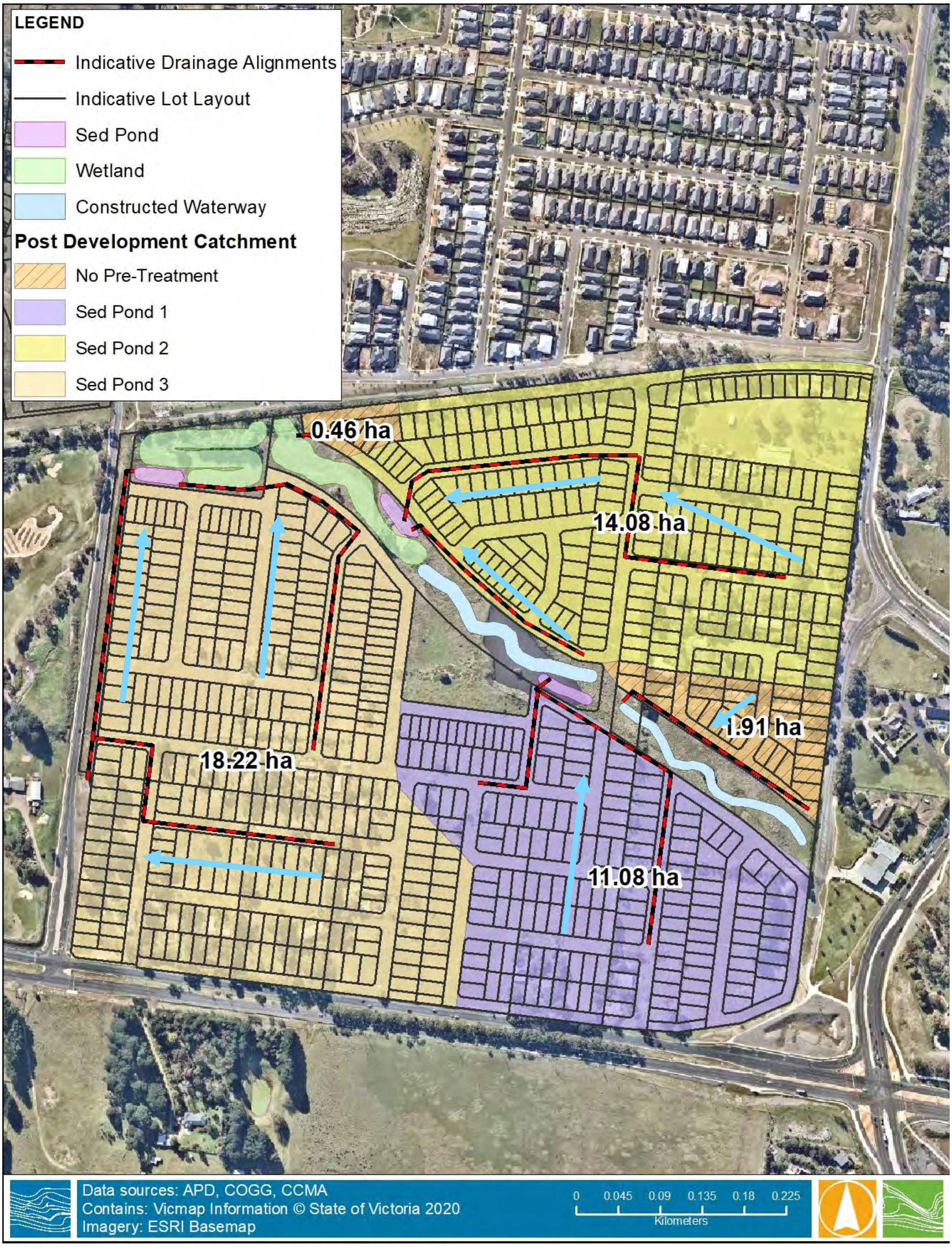

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FIGURE 6-1 WSUD ASSET LOCATION AND INDICATIVE DRAINAGE ALIGNMENT

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6.2 Waterway Corridor

The waterway within the site is designated under the Victorian Water Act (1989) and is listed as Designated Waterway ID 33-30. Designated waterways with Order 1 and 2 streams require a setback of at least 20 metres from each bank. As noted during the site visit, the banks of the waterway are not clearly defined. The Authority recommends following the hydraulic width method from the guidelines, whereby the "top of bank", or reference point, is set by the 1% AEP flood extent. Since the development proposes to provide a constructed waterway and decommission the large dam on site, the hydraulic width at that location will be determined by post-development flood modelling.

6.3 Constructed Waterway

The waterway reach has been designed as a compound waterway (i.e., a low flow channel within a high flow channel), generally following the alignment of the existing waterway. Melbourne Water's Waterway Corridors guidelines (Melbourne Water 2013)[14] provides guidance on the corridor widths for constructed waterways, which are based on the 1% AEP hydraulic width and the availability of active edges (roads) on both sides of the corridor (Figure 6-2).

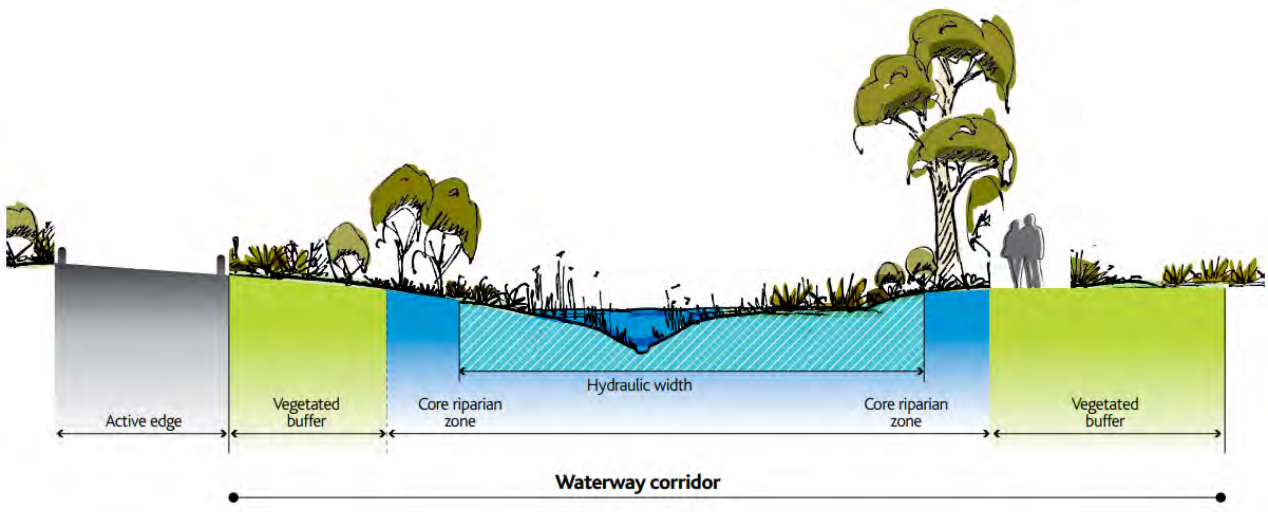

FIGURE 6-2 EXAMPLE OF SETBACK SUBZONES FOR CONSTRUCTED WATERWAYS (MELBOURNE WATER, 2013)

The waterway corridor widths in this concept design exceed the sliding scale minimum waterway corridor requirements outlined in Melbourne Water's *Waterway Corridors guidelines* (2013) and is sufficient to meet the reserves batter slope requirements:

- The 1% peak flow for the site in developed conditions flowing into the basin is 6.4 m^3/s (refer to RORB modelling in section 0);
  - This would result in a hydraulic width of 13.6 m (based on a 1 in 200 slope);
- A 40 m wide (minimum) waterway corridor would be required based on the hydraulic width.

The proposed concept plan is consistent with the Melbourne Water's *Waterway Corridors guidelines* (2013).

[14] Melbourne Water, 2013, Waterway Corridors: guidelines for greenfield development areas within the Port Phillip and Westernport region

6.4 Typical Cross-section and Longitudinal Grade

Figure 6-3 shows a typical cross section of the proposed compound waterway system. Based on the Melbourne Water's *Waterway Corridors guidelines* (2013) batter slopes along the low flow channel (LFC) should be no steeper than 1 in 3. The LFC will meander through the corridor. Batter slopes along the high flow channel (HFC) area and the edges of the reserve vary from the standard 1 in 6 batter and can be up to 1 in 3 using a stepped rock outcome based on Melbourne Water's *Constructed Wetlands guidelines*. This differs slightly from the 1 in 6 example section provided in the overall Jetty Road Stage 2 SWMS. In areas where it is not feasible to achieve the recommended batter slopes, it is feasible to restrict access by providing dense vegetation or retaining/gabion walls and appropriate fencing.

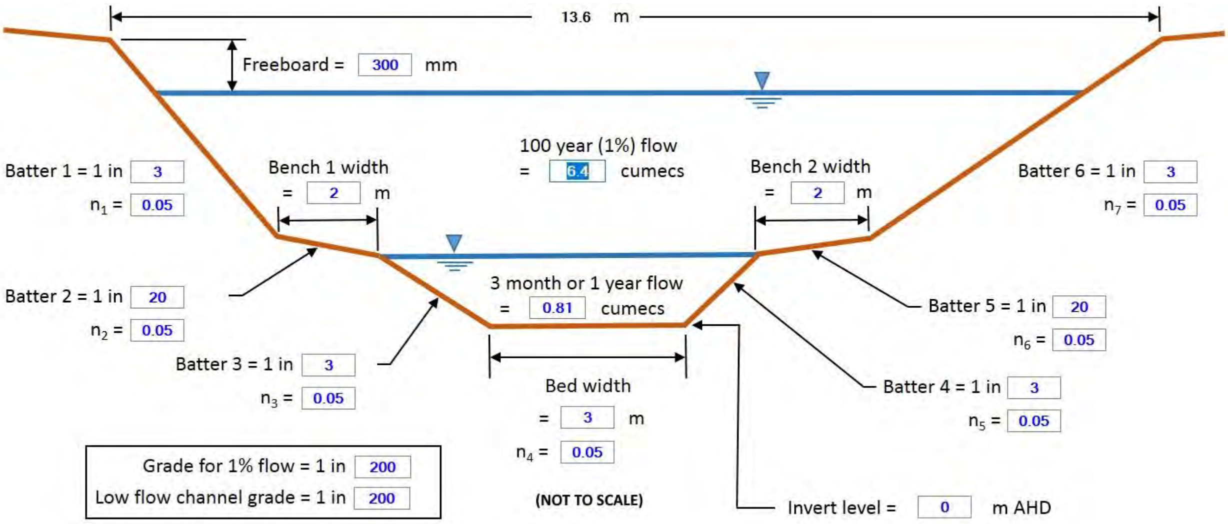

FIGURE 6-3 COMPOUND WATERWAY CROSS-SECTION (PC-CONVEY)

The existing longitudinal slope across the site is around 1 in 140 however, it is proposed to use pool-riffle and pool-run sequences to:

- Undertake earthworks in the upper section of the constructed waterway (pending detailed design of where the constructed waterway will start)
- Have a flatter slope across the majority of the reach, bar steeper rock chute/riffle arrangements:
  - Design grades should be within the acceptable 'stable' range, being flatter than 1 in 200;
- Create a range of habitat along the reach:
  - The waterway will be planted with instream and riparian vegetation.
- Facilitate connection to existing upstream and downstream reaches and adjacent properties;
- Manage steeper section of the reach via rock chutes and graded rocks.

The proposed pool-riffle and pool-run sequences comprises of large pools connected by riffle sections, as per the longitudinal section shown in Figure 6-4. The length of the waterway is likely to be in the order of 300-500m (pending the area required to provide suitable wetland treatment area. The assumed longitudinal slope (for conveyance calculations) was assumed to be about 1 in 200, i.e., within the acceptable 'stable' range. Rock chutes are likely to be required for sections with longitudinal grades steeper than 1 in 200.

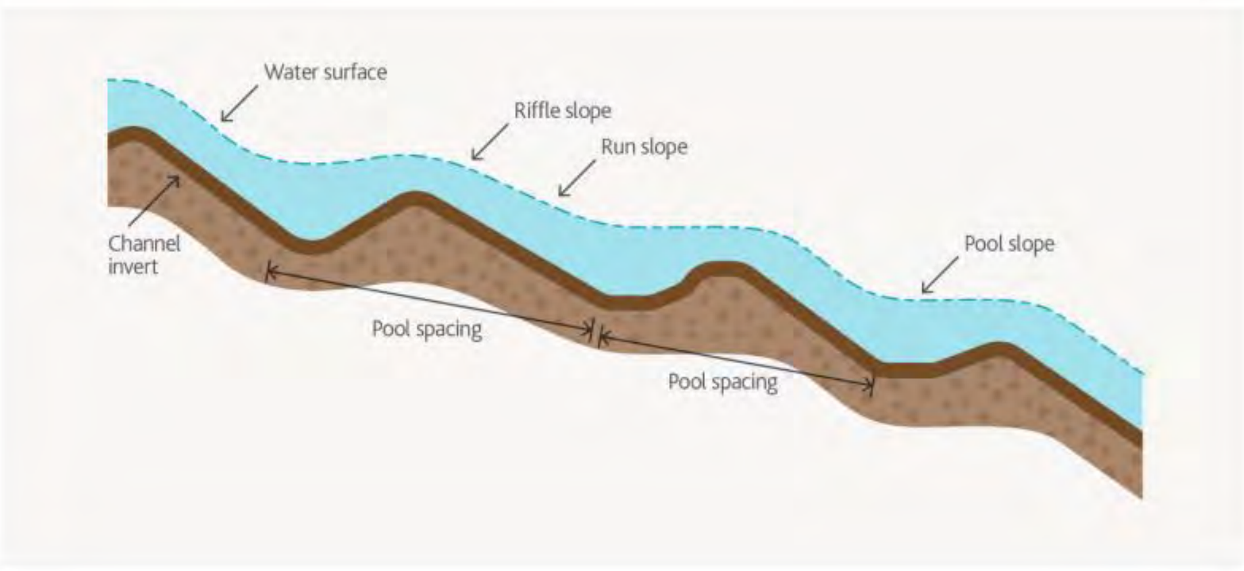

FIGURE 6-4 TYPICAL RIFFLE-POOL SEQUENCE (MW CONSTRUCTED WATERWAY DESIGN MANUAL, 2019)

6.5 Flow Capacity and Velocity Analysis

The hydraulic width along the waterway was determined through PC-convey analysis of representative waterway cross sections at upstream and downstream ends of the site:

- The HFC 13.6 m width would be sufficient to convey the 1% AEP peak flow of 6.4 m^3/s (the unmitigated post-development runoff for the site), as shown in Figure 6-5:
  - This incorporates a 300mm freeboard above the 1% AEP flood level
  - The overall waterway corridor (40 m) provides ample width to ensure 300mm freeboard and complies with the corridor width expected based on the Melbourne Water Constructed Waterway Guidelines.
- The LFC, with a minimum width of 3 m, would have ample capacity to convey the 1 year-equivalent flow of 1.05 m^3/s, even when assuming a longitudinal grade of 1 in 200, to allow for increased sinuosity (Figure 6-5).

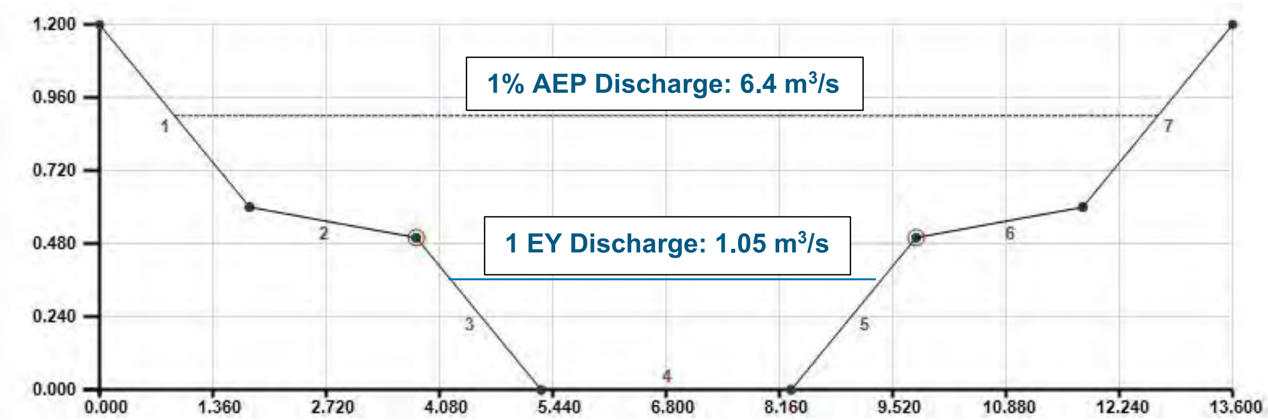

FIGURE 6-5 HFC & LFC PC CONVEY ANALYSIS (1% AEP & 1 EY FLOW)

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PC-Convey analysis shows that the 1% AEP velocities are less than 1.0 m/s in the waterway (excluding riffle sections) and the 1 YE velocities less than 0.5 m/s in the LFC. Velocities will be higher along rock chutes however, shear stresses and further detailed analysis across the waterway and these sections will be checked during the functional design stage.

- PC Convey analysis demonstrated that the proposed compound channel can safely convey the 1% AEP flow event and LFC has a bankfull capacity to convey flows the 1EY (maximum) event (3 m minimum base), noting that:
  - Adopted roughness parameters (0.05) was in accordance with Melbourne Water's guidelines;
  - Peak 1% AEP average velocity of 1.00 m/s based on a longitudinal grade of 1 in 200 based on the proposed constructed waterway slope.
  - Peak 1% AEP average velocity of 1.13 m/s based on a longitudinal grade of 1 in 140 based on the current waterway slope through the site.

It is appropriate for other elements of the design to be considered and confirmed at the functional and/or detailed design stages.

6.5.1 Waterway Crossings

Based on the current indicative lot layout and previous Masterplan, there is the need to provide a waterway crossing within the site (Figure 6-6). The crossings should be designed in accordance with CCMA Works on waterway and waterway crossing guidelines. The indicative location proposed in Figure 6-6 show that stormwater runoff from some parts of the site will have entered the constructed waterway. Culvert sizing was calculated at this location using flows from a rational calculation. The catchment areas used have been based on the WSUD asset location and indicative drainage alignment.

FIGURE 6-6 POTENTIAL WATERWAY CROSSINGS

Design Flows

The Crossing is required to convey the external catchment inflow (0.5 m^3/s – assumed to be constant inflow for this assessment), 2ha of untreated flows located at the eastern end of the development and 12ha of the southern portion site at the which outfalls to a sedimentation basin = 4.2 m^3/s in a 1% AEP event.

Culvert Sizing

HY-8 was used to calculate an appropriate culvert size to ensure flows can be passed without the waterway crossing overtopping. A culvert length of 8 metres at a slope of 1 in 100 grade and Mannings n roughness value of 0.05 was adopted for the calculation. A Mannings roughness value of 0.013 was adopted for the proposed culverts.

At the crossing location, the estimated waterway width is based on a bottom width of 5m in the LFC as the waterway transitions to the wider wetland. Three 1200mm x 900mm high box culverts convey the 1% AEP design flow with a depth above the obvert of the box culvert (~200mm). The culvert rating curve is shown in Figure 6-7.

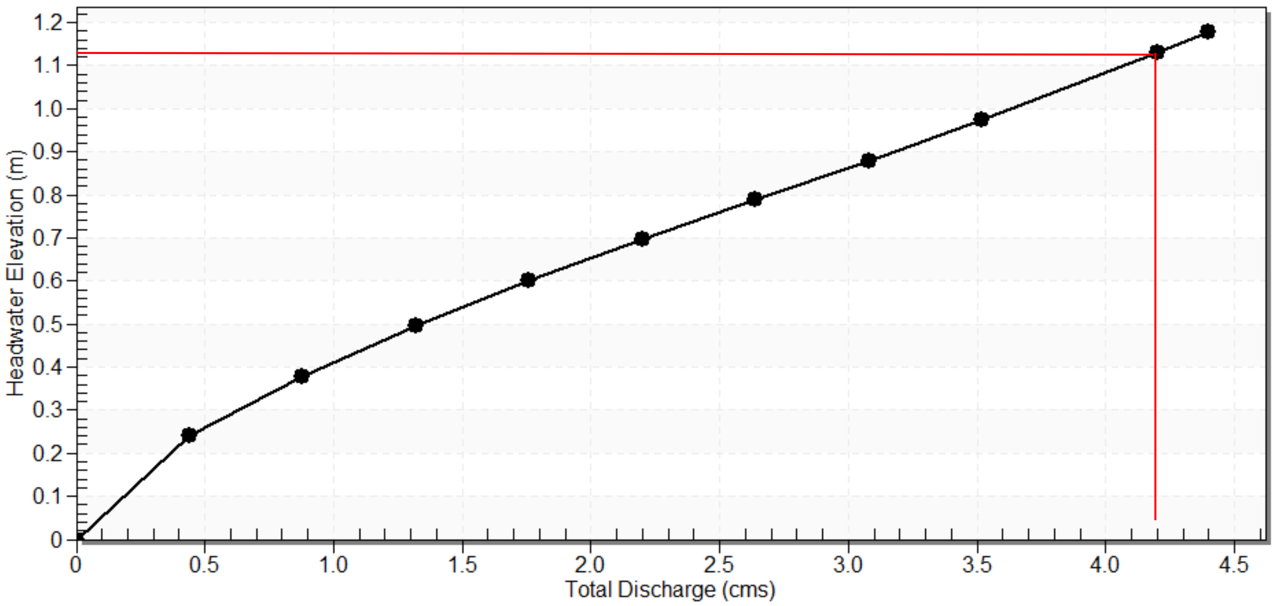

FIGURE 6-7 CROSSING 2 RATING CURVE

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6.6 Wetland and Sedimentation Ponds Design

The total estimated treatment area of wetland and sediment pond is 13,800 m^2. However, the total asset footprint needed to allow minimum offsets, batter slopes, access routes and pipe connections etc is likely to be considerably larger. A preliminary asset footprint estimate was made by increasing the treatment area by a factor of 2 as a conservative arrangement. This yielded a total water quality asset footprint of 27,600 m^2 but is likely to be refined at a later stage of development.

Based on the current development layout, it is proposed to incorporate a linear wetland and sedimentation ponds within the waterway corridor with the wetland system located at the downstream end of the constructed waterway. While the linear wetland system will provide opportunities to improve biodiversity, measures to protect wetland vegetation from extended inundation and scouring needs to be incorporated in the waterway design during the concept and functional design stages.

There are numerous examples of this arrangement including within the City of Greater Geelong and the inclusion of this design provides the opportunity to improve the waterway outcome. Two examples of existing linear wetland systems are shown in Figure 6-8 (Armstrong Creek) and Figure 6-9 (Marriot Waters Estate – Lynbrook). These examples show the use of multiple sedimentation ponds to provide pre-treatment of stormwater runoff from the site prior to entering the wetlands.

A typical cross section of linear wetland system using 1:3 batter slopes is shown in Figure 6-10. 1 % AEP hydraulic width was estimated to be ~35 m using Manning's calculation (1 in 200 slope and n = 0.05 similar to compound waterway calculations). Figure 6-10 shows a HFC width of 35m which fits within the minimum 40m waterway corridor width. Additional measures to stabilise batters, such as stepped rock batters and matting will be required if steeper batter slopes are adopted for the linear wetland for sections with steeper batters.

FIGURE 6-8 LINEAR WETLAND SYSTEM AT ARMSTRONG CREEK (SOURCE: METROMAP IMAGERY)

FIGURE 6-9 LINEAR WETLAND SYSTEM AT MARRIOTT WATERS (SOURCE: METROMAP IMAGERY)

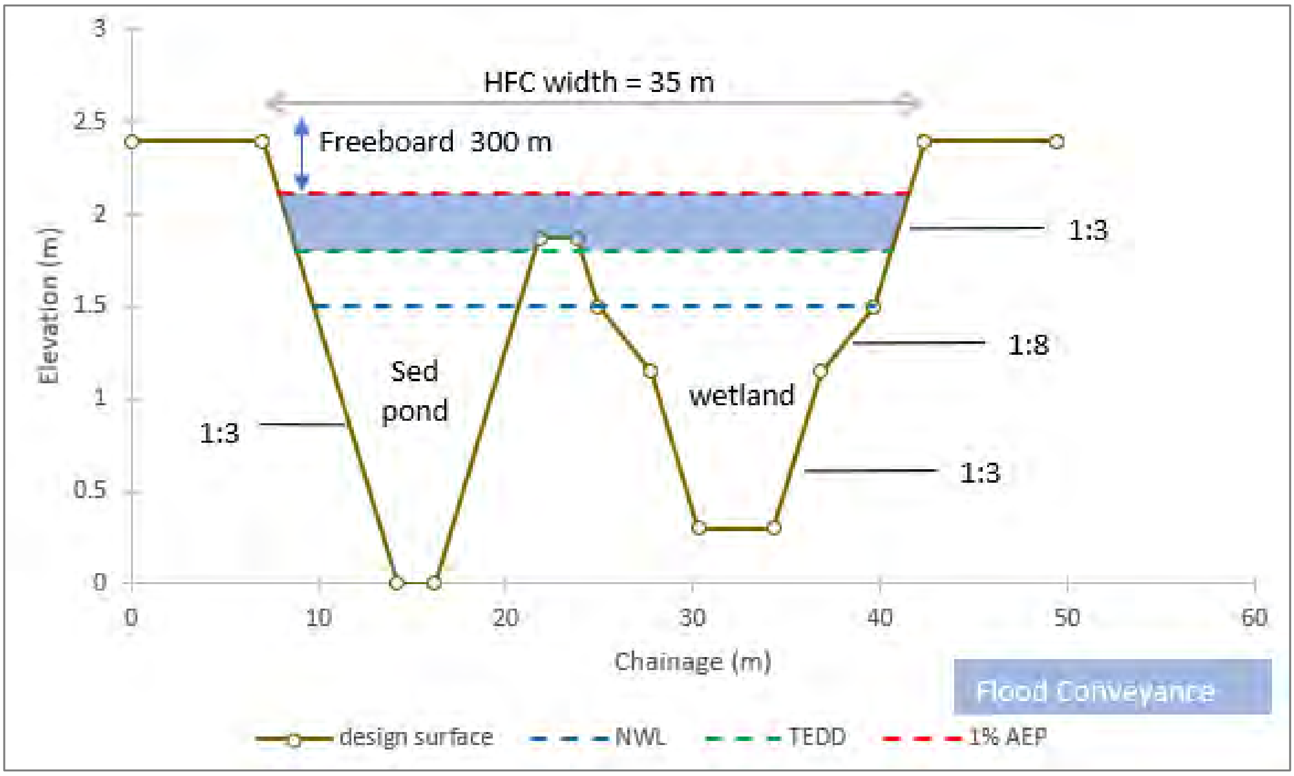

FIGURE 6-10 TYPICAL LINEAR WETLAND SECTION WITH SEDIMENT POND AND WETLAND MACROPHTYE ZONES

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6.6.1 Vegetation Establishment and Velocity Analysis

As outlined in Section 6.5, PC-Convey analysis shows that the 1% AEP velocities are less than 1.0 m/s in the waterway (excluding riffle sections) and the 1 YE velocities less than 0.5 m/s in the LFC. The velocities over the wetland component are likely to be considerably lower as the overall hydraulic width is in the order of 30-35m and flow depths will be typically shallower.

Other options to ensure vegetation can establish and avoid scour/erosion include the use of gypsum ameliorated into the topsoil to support plantation establishment and limit its dispersive characteristics as has been undertaken throughout the Warralily development.

6.6.2 Sedimentation Pond Maintenance

Clean out frequency of the sedimentation ponds sized has been determined and provided in Appendix C.

The design of the sedimentation ponds should allow for the ability to draw down and divert individual sedimentation ponds given they are 'online assets'. This should include the ability to 'block off' flows into the sedimentation drawing the draw down and dewatering process through sandbagging of piped inflows around the sedimentation pond. It is recommended that maintenance work be scheduled during a dry period (summer) to avoid the volume of stormwater diverted around the sedimentation pond during the maintenance works.

An overall stormwater asset maintenance program should also be prepared at a later stage of design incorporating the clean out frequency, recommended time of the year to carry out maintenance and the procedure to undertake the works.

6.7 Summary

The constructed waterway concept design incorporates the following:

- A compound waterway (i.e., a low flow channel within a high flow channel) that begins within Property No. 23 at the eastern end of the site.
- The waterway corridor alignment is proposed with an overall longitudinal grade of 1 in 200. This is to be confirmed at a later date and will incorporate the existing low point at the downstream property boundary (grated pit outfall);
- The existing dam located on site is considered to be decommissioned as part of the constructed waterway works;
- The constructed waterway alignment mostly follows existing waterway alignment and considers the upstream catchment
- LFC will allow for sinuosity, noting that it is appropriate to confirm this during functional design stage;
- Overall longitudinal grade will be no steeper than 1 in 200 noting that:
  - Steeper sections may be required in place, to tie-in with upstream and downstream levels.
- The waterway will transition into a constructed linear wetland located within the waterway corridor;
- Three sedimentation basins are required as pre-treatment along the waterway corridor prior to flows entering the wetland.

7 OUTFALL FROM SITE

Flows discharging from the retarding basin within the site will leave the site via the existing grated pit and twin box culverts located under Tivoli Drive. Currently the flows leaving the site are conveyed into a grated pit located with the rail reserve and under Tivoli Drive via twin box culverts (1200x600mm) shown in Figure 7-1.

The flows then travel along an existing drain which appears to be located between the former rail embankment and the Curlewis Golf Course within the Crown land reserve. It appears the majority of flows are then captured in a dam located in the golf course (Figure 7-1) and into the rail reserve corridor. Once the capacity of the dam is exceeded, flows travel north under the rail embankment via a single 1300x900mm box culvert.

Current flood modelling shows that some flow in the open channel spills out into the golf course. As part of the SWMS, peak flows leaving the site will be managed to pre-development "existing conditions" rates. It is likely that the frequency of flow and overall volume leaving the site would increase should stormwater harvesting, or an evaporation pond not be implemented as part of the overall strategy.

Minor earthworks and erosion protection (vegetation/rock beaching) to improve the open channel would ensure that flows are maintained within the rail trail reserve and would not impact the golf course may be required as part of the overall Jetty Road Stage 2 drainage works. Further to this, it is recommended an easement over the existing dam located within the golf course is proposed to ensure ongoing maintenance of the open channel and culvert under the rail embankment can be maintained.

FIGURE 7-1 LEFT: GRATED PIT UPSTREAM OF TIVOLI DRIVE & RIGHT: DAM LOCATED UPSTREAM OF THE CULVERT UNDER RAIL TRAIL EMBANKMENT

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8 SUMMARY

This report sets out a recommended Stormwater Management Strategy (SWMS) for a proposed residential subdivision of the land 1421-1423 & 1479 Portalington Road and 10 & 12-18 Hackwell Place, Curlewis. The SWMS sets out a concept design to manage stormwater runoff discharge (flow rate) and quality from the proposed development to meet infrastructure needs in accordance with the Infrastructure Design Manual and the requirements of the City of Greater Geelong.

Flood modelling of the site incorporating the inflow into the site at the south-east corner (from the retarding basin located within the Drysdale Bypass works) has been carried out. Based on the 1% AEP flows generated from the site and upstream catchment (under existing conditions), a peak flow rate of 1.88 m^3/s was determined as the existing conditions flow rate leaving the site.

A single retarding basin asset is proposed at the end of the proposed constructed waterway at the north-west end of the proposed development to maintain post-development flow rate at pre-development rates at the main outfall. No detention strategies are currently proposed for the remaining two outfalls into Tivoli Drive.

The existing waterway immediately downstream of the proposed constructed waterway has sufficient capacity to convey post-development flows with the recommendation for minor earthworks within the rail reserve adjacent to the golf course to maintain the flow within the reserve.

The proposed WSUD strategy to meet the BPEM target involved assets along the constructed waterway. MUSIC modelling indicates a sediment pond and wetland are required for the proposed development. Additional measures including 2kL tank on each residential lot within the development, stormwater harvesting for irrigation will be required to maintain the post-development runoff volume at the existing levels.

Proposed SWMS assets are shown in Figure 8-1 on top of the previous Jetty Road Master Plan which provides an indicative layout.

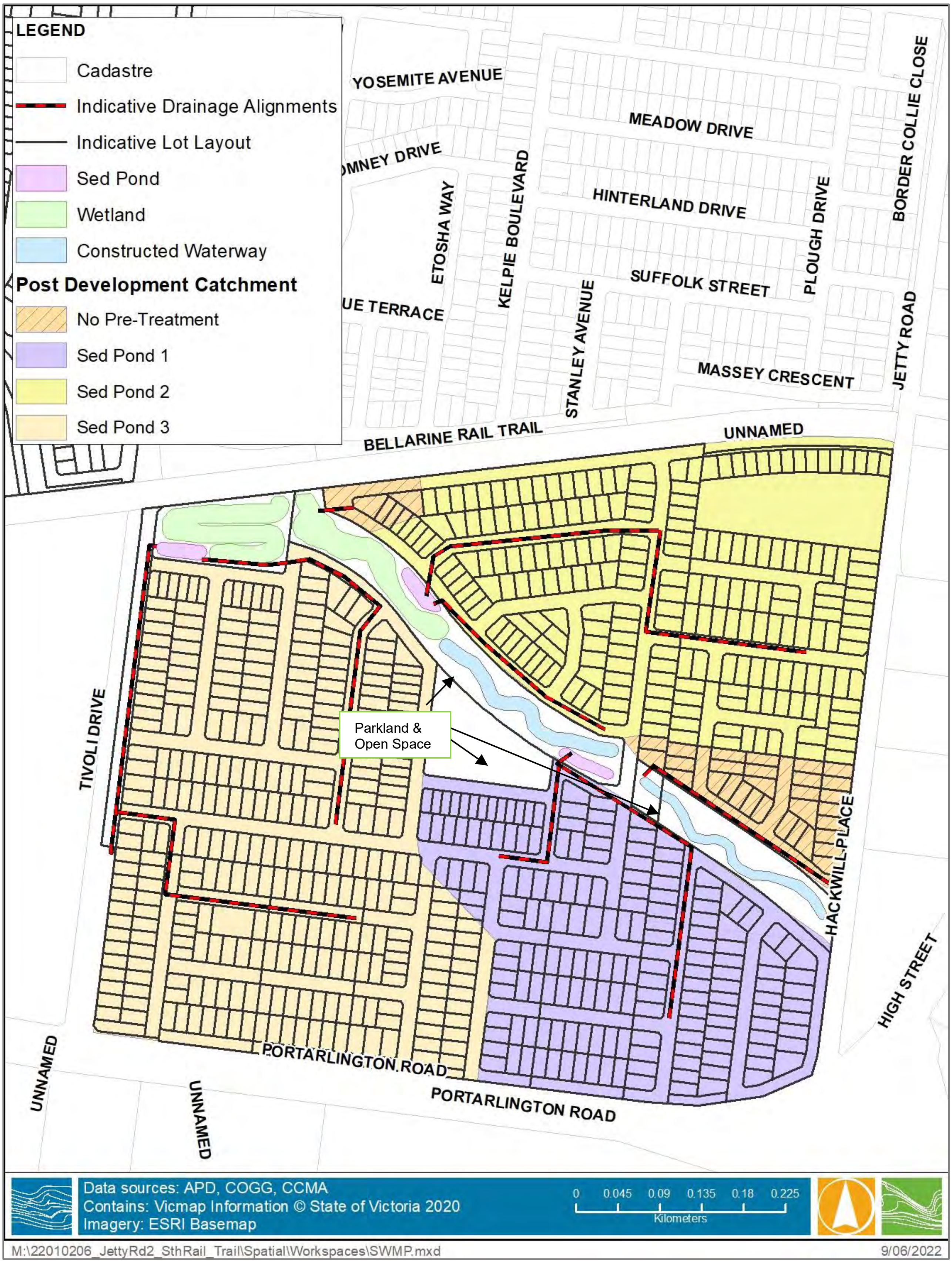

FIGURE 8-1 SWMS ASSET LOCATIONS

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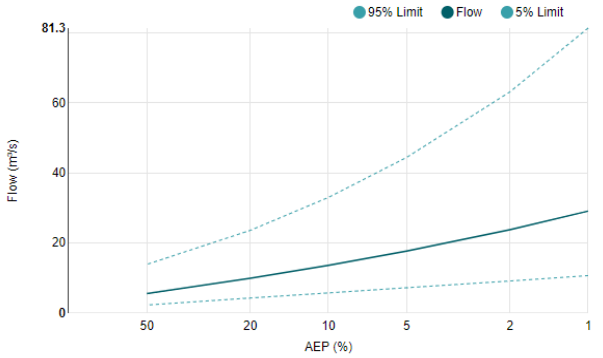
APPENDIX A
RORB MODEL PRE-VALIDATION

A-1 Regional Flood Frequency Estimate

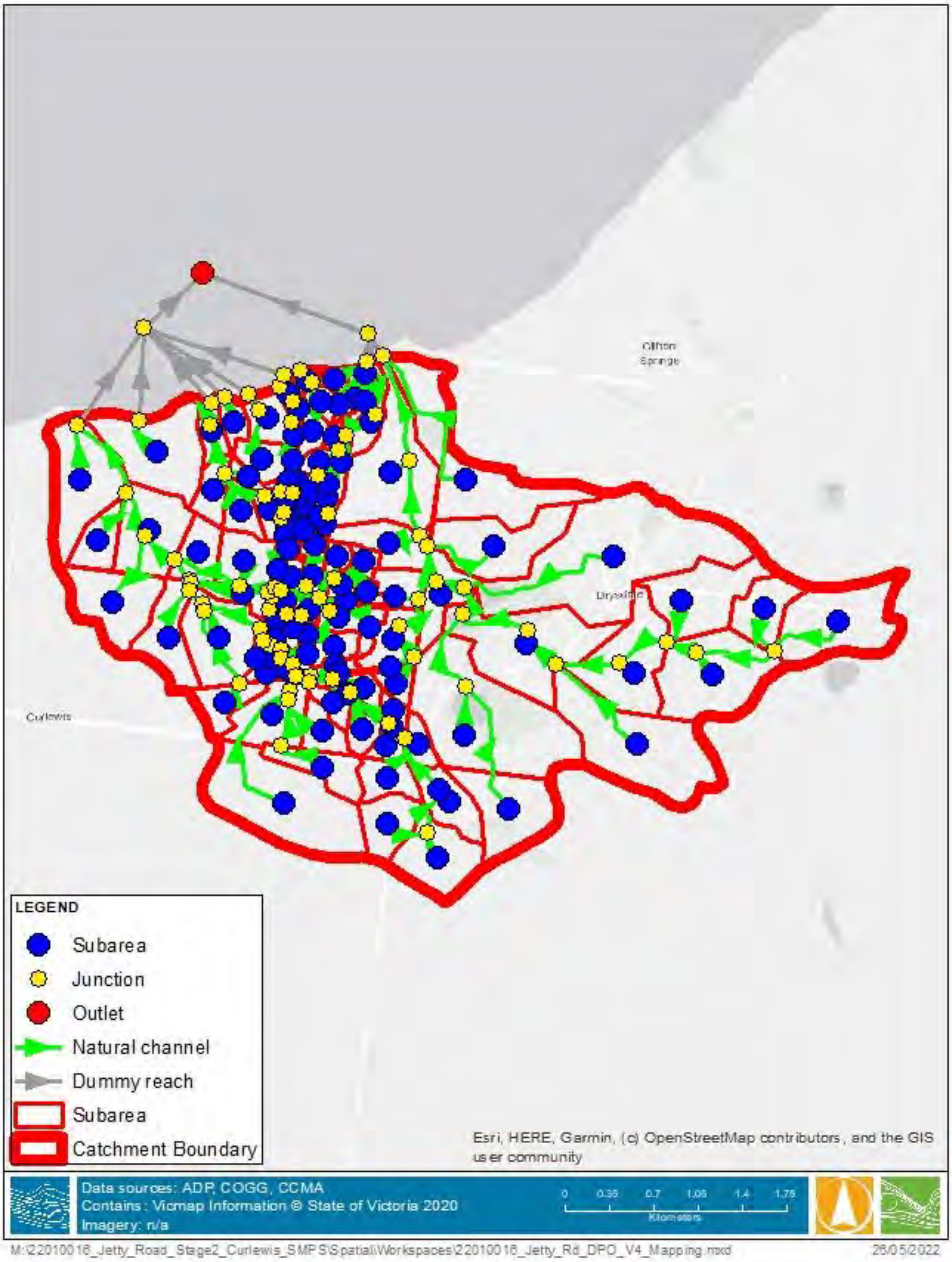
Results | Regional Flood Frequency Estimation Model



AEP (%)	Discharge (m^3/s)	Lower Confidence Limit (5%) (m^3/s)	Upper Confidence Limit (95%) (m^3/s)
50	5.58	2.30	13.9
20	9.96	4.31	23.6
10	13.6	5.76	33.0
5	17.7	7.24	44.4
2	23.8	9.16	63.1
1	29.1	10.7	81.3

Input Data	
Date/Time	2022-05-19 07:00
Catchment Name	Catchment1
Latitude (Outlet)	-38.16032
Longitude (Outlet)	144.51944
Latitude (Centroid)	-38.17485
Longitude (Centroid)	144.5518
Catchment Area (km^2)	16.085
Distance to Nearest Gauged Catchment (km)	28.34
50% AEP 6 Hour Rainfall Intensity (mm/h)	4.513329
2% AEP 6 Hour Rainfall Intensity (mm/h)	9.873671
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 v1
Region Source (User/Auto)	Auto

A-2 Pre-Development Catchment Plan



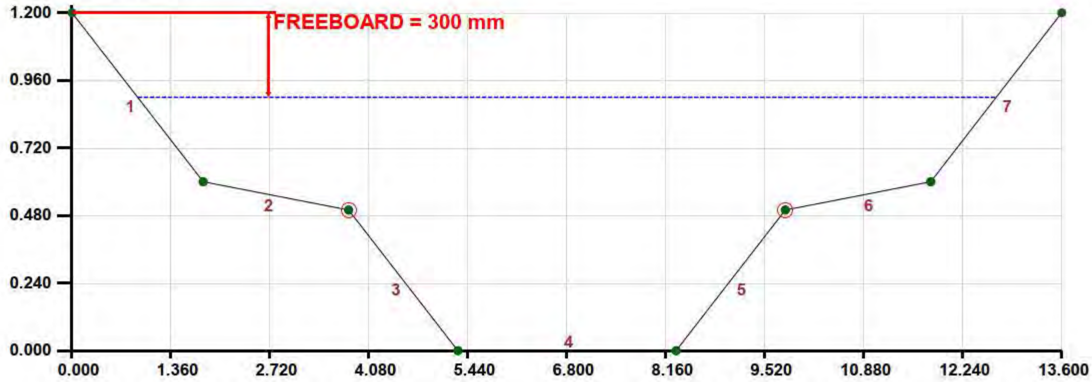
Catchment area = 16.08 km^2

d_{av} = 2.94 km

APPENDIX B
PC CONVEY ANALYSIS - WATERWAY

PROJECT: Jetty Road Stage 2
CWD XS (Outfall 1)_SteeperSlopes
Print-out date: 10/06/2022 - Time: 11:04
Data File: Constructed_Waterway_Outfall1_Alternative_SteeperSloper.dat

1. CROSS-SECTION



2. DISCHARGE INFORMATION

1% AEP storm event
Design discharge after construction of retarding basin

Required overland / channel / watercourse discharge = 6,400 litres/second

3. RESULTS Water surface elevation = 0.900 m

High Flow Channel grade = 1 in 200, Main Channel / Low Flow Channel grade = 1 in 200.

	LEFT OVERBANK	MAIN CHANNEL	RIGHT OVERBANK	TOTAL CROSS-SECTION
Discharge (litres/second):	508.934	5,450.576	508.934	6,468.445
D(Max) = Max. Depth (m):	0.400	0.900	0.400	0.900
D(Ave) = Ave. Depth (m):	0.288	0.775	0.288	0.775
V = Ave. Velocity (m/s):	0.610	1.172	0.610	1.023
D(Max) x V (cumecs/m):	0.244	1.055	0.244	0.921
D(Ave) x V (cumecs/m):	0.175	0.908	0.175	0.793
Froude Number:	0.363	0.425	0.363	0.401
Area (m^2):	0.835	4.650	0.835	6.320
Wetted Perimeter (m):	2.951	6.162	2.951	12.065
Flow Width (m):	2.900	6.000	2.900	11.800
Hydraulic Radius (m):	0.283	0.755	0.283	0.524
Composite Manning's n:	0.050	0.050	0.050	0.050
Split Flow?	-	-	-	No

4. CROSS-SECTION DATA

	LEFT HAND POINT		RIGHT HAND POINT		
SEGMENT NO.	CHAINAGE (m)	R.L. (m)	CHAINAGE (m)	R.L. (m)	MANNING'S N
1	0.000	1.200	1.800	0.600	0.050
2	1.800	0.600	3.800	0.500	0.050
3	3.800	0.500	5.300	0.000	0.050
4	5.300	0.000	8.300	0.000	0.050
5	8.300	0.000	9.800	0.500	0.050
6	9.800	0.500	11.800	0.600	0.050
7	11.800	0.600	13.600	1.200	0.050

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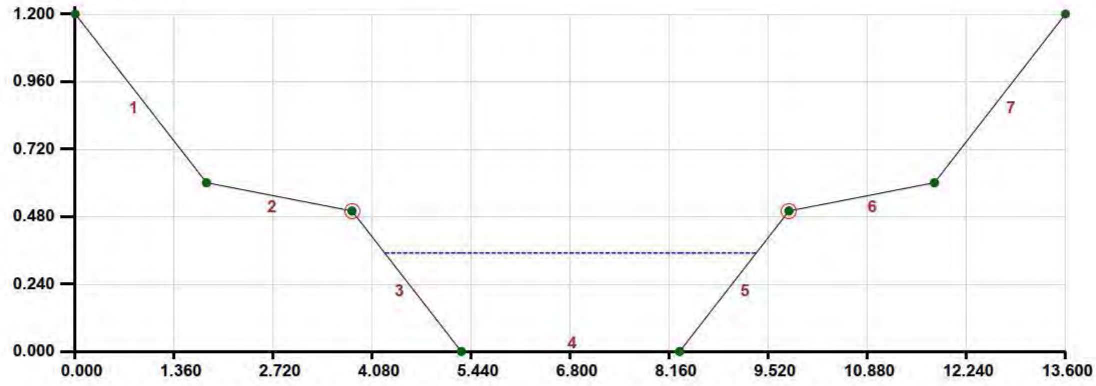
PROJECT: Jetty Road Stage 2
CWD XS (Outfall 1)_SteeperSlopes
Print-out date: 10/06/2022 - Time: 11:11
Data File: Constructed_Waterway_Outfall1_Alternative_SteeperSloper.dat

1. CROSS-SECTION



2. DISCHARGE INFORMATION

1% AEP storm event
Design discharge after construction of retarding basin

Required overland / channel / watercourse discharge = 810 litres/second

3. RESULTS Water surface elevation = 0.350 m

High Flow Channel grade = 1 in 200, Main Channel / Low Flow Channel grade = 1 in 200.

	LEFT OVERBANK	MAIN CHANNEL	RIGHT OVERBANK	TOTAL CROSS-SECTION
Discharge (litres/second):	0.000	841.321	0.000	841.321
D(Max) = Max. Depth (m):	0.000	0.350	0.000	0.350
D(Ave) = Ave. Depth (m):	0.000	0.278	0.000	0.278
V = Ave. Velocity (m/s):	0.000	0.594	0.000	0.594
D(Max) x V (cumecs/m):	0.000	0.208	0.000	0.208
D(Ave) x V (cumecs/m):	0.000	0.165	0.000	0.165
Froude Number:	0.000	0.359	0.000	0.359
Area (m^2):	0.000	1.418	0.000	1.418
Wetted Perimeter (m):	0.000	5.214	0.000	5.214
Flow Width (m):	0.000	5.100	0.000	5.100
Hydraulic Radius (m):	0.000	0.272	0.000	0.272
Composite Manning's n:	0.000	0.050	0.000	0.050
Split Flow?	-	-	-	No

4. CROSS-SECTION DATA

	LEFT HAND POINT		RIGHT HAND POINT		
SEGMENT NO.	CHAINAGE (m)	R.L. (m)	CHAINAGE (m)	R.L. (m)	MANNING'S N
1	0.000	1.200	1.800	0.600	0.050
2	1.800	0.600	3.800	0.500	0.050
3	3.800	0.500	5.300	0.000	0.050
4	5.300	0.000	8.300	0.000	0.050
5	8.300	0.000	9.800	0.500	0.050
6	9.800	0.500	11.800	0.600	0.050
7	11.800	0.600	13.600	1.200	0.050

APPENDIX C
SEDIMENT POND SIZING

Fair and Geyer Equation – Equ 10.3 WSUD Stormwater Technical Manual (2005)

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n} \qquad \lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$$

R = fraction of Initial Solids Removed = 80 - 90 % typ.

R	= fraction of Initial Solids Removed = 80 - 90 % typ.	
d_p	= Depth of permanent pool	
d_e	= Extended detention depth above permanent pool	
d^*	= depth below permanent pool sufficient to retain particles (lower of 1.0m or d_p)	
Q	= design flow (Typically 3 month, 6 month or 1 year flow)	
A	= Basin Surface Area	
n	= turbulence parameter (see above)	= 1 for significant short circuiting and turbulence = 5 for insignificant short circuiting and turbulence
v_s	= setting velocity for particles	

Table 7.2 Settling velocities under ideal conditions (Maryland Department of Environment, 1987)

Classification of Particle size range	Particle diameter (μm)	Settling velocities (mm/s)
Very coarse sand	2000	200
Coarse sand	1000	100
Medium sand	500	53
Fine sand	250	26
Very fine sand	125	11
Coarse silt	62	2.3
Medium silt	31	0.66
Fine silt	16	0.18
Very fine silt	8	0.04
Clay	4	0.011

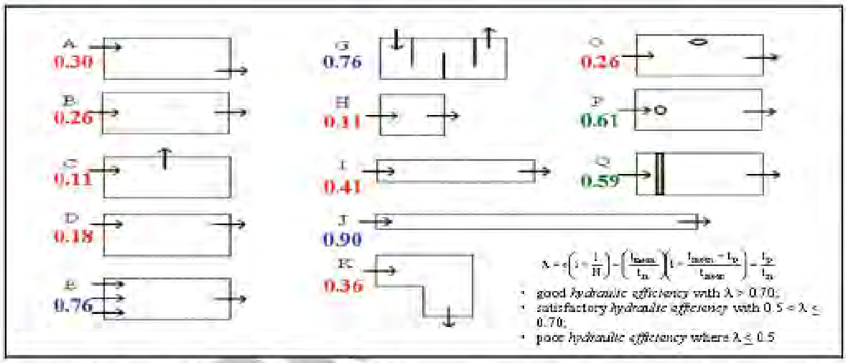


Figure 10.5 Hydraulic Efficiency – A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds; Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment [Source: Australian Runoff Quality, 2003]

Source: WSUD Engineering Procedures: Stormwater Technical Manual DRAFT 2004

C-1 Overall SWMS – Sediment Basin Sizing

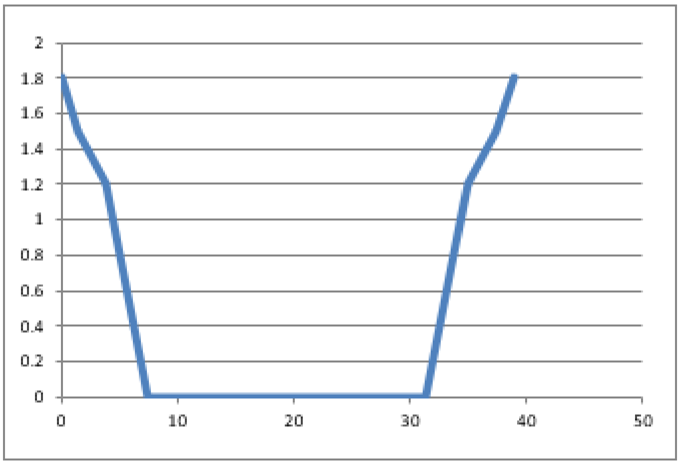
The overall SWMS for the entire Jetty Road 2 estimated the size of a single sediment basin that is required to treat the whole development south of rail trail. A summary of calculations are as below.

Trapezoid Calculator

Total Depth		**1.8** m			
Extended Detention	0.30	1 in	5	0	1.8
Depth of 1:8	0.3	1 in	8	1.5	1.50
Depth of 1:3	1.20	1 in	3	3.9	1.20
NWL Length	50.20	m		7.5	0
NWL Width	35.86	m		31.36	0
NWL Area	**1800.00**	**m²**		34.96	1.2
				37.36	1.5
				38.86	1.8

AREA

Bottom Length	38.20	m
Bottom Width	23.86	m
Bottom Area	911.32	m²
Bench Length	45.40	m
Bench Width	31.06	m
Bench Area	1409.97	m²
NWL Length	50.20	m
NWL Width	35.86	m
NWL Area	1800.0	m²
TED Length	53.20	m
TED Width	38.86	m
TED Area	2067.17	m²



STORAGE

Increments	0.1
Side length per increment 1:3	0.3
Side length per increment 1:8	0.8
Side length per increment 1:5	0.5

Stage	Storage (m	Area (m2)	Length	Width
0	0.0	911.3	38.20	23.86
0.1	93.0	948.9	38.80	24.46
0.2	189.8	987.2	39.40	25.06
0.3	290.4	1026.3	40.00	25.66
0.4	395.0	1066.0	40.60	26.26
0.5	503.6	1106.5	41.20	26.86
0.6	616.3	1147.7	41.80	27.46
0.7	733.2	1189.6	42.40	28.06
0.8	854.3	1232.2	43.00	28.66
0.9	979.6	1275.6	43.60	29.26
1	1109.4	1319.7	44.20	29.86
1.8	2424.6	2067.2	53.20	38.86

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Calculations

Sediment Target = **Very fine sand** *Very fine saand for standard residential develpoments*

Vs =	0.011	m/s	This value changes for different particle size target
d_e =	0.30	m	Extended Detention Depth *max 0.35 for MW*
d_p =	1.5	m	Permanent Pool Vollume Depth *1.5 m is a common depth for standard residential developments*
d^* =	1	m	(lower of 1 m and dp)
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.38		
Q =	1.40	m^3/s	1 EY flows from RORB modelling
A =	1800	m^2	Area of the sediment basin at NWL
L/W =	1.4		Length/Width Ratio (assuming rectangular shape)
$\frac{V_s}{Q/A}$ =	14.13		
λ =	0.11		Pond shape assumption (see figure 10.5 above)
n =	1.12		

Fraction of Initial Solids Removed

R = **96.21%**

Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle

Cleanout Frequency

Catchment area =	47.6	ha	Just urban catchment (excluding 5.32 ha reserve area)
Sediment load =	1.60	$m^3/ha/yr$	1.6 - Willing and Partners 1992 - urban load
Gross Pollutant Load =	0.40	$m^3/ha/yr$	0.4 - Alison et al 1998

Option 1 Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)

Actual basin depth =	1.00	m	500 mm below the NWL (i.e. dp-0.5)
Actual Basin volume =	503.64	m^3	Basin volume at 500 mm below the NWL

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3)}{\text{ActualBasinVolume } (m^3)}$ 0.19 per year — Clean out every 5.29 years

Try to minimise cleanouts - ideally, once every 5 years

Dewatering Area

Dewatering depth =	0.50	m	Max depostion heigh *## max 0.5 m for MW; 0.3 m good practice among some Councils*
Sediment volume collected every 5 years =	503.64	m^3	volume of sediment acumulated up to 0.5 m below the NWL
Required Dewatering area =	1007.29	m^2	

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C-2 Multiple Sediment Ponds

A review of development layout suggested that multiple sediment basins will be required to capture sediment from the development. Local catchment boundaries were derived based on the elevation and an indicative road and lot layout. Based on the existing elevation, two minor catchment with a cumulative area of 2.6 ha (6% of developed catchment treated by the wetland) will not drain to any of the sediment ponds. Nearly 2 ha of these catchment will pass through the constructed waterway, and is likely to receive some pre-treatment before reaching to wetland. Currently, the proposed three sediment ponds exceeds the target sediment capture rate, therefore, the overall sediment removal target is met at the Outfall 1.

FIGURE C-1 INTERNAL CATCHMENTS DRAINING TO LOCAL SEDIMENT PONDS

Sediment removal efficiency and the required clean out frequency was estimated using a similar approach to the overall strategy. 1 EY flows were estimated using a Rational Method calculation.

TABLE C-1 SEDIMENT BASIN SIZING SUMMARY

	Sed Basin 1	Sed Basin 2	Sed Basin 3
Catchment Area (ha)	11.1	14.1	13.9
1 EY Flow (m^3/s)[1]	0.14	0.18	0.18
Surface area at NWL (m^2)	650	750	750
Sediment Capture Efficiency	99%	99%	99%
Sediment Accumulation Volume (m^3)[2]	109	139	139
Clean out frequency (1 in x years)	5.0	5.0	5.0

1. Estimated using the Rational Method
2. Volume 500 mm below the normal water level

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APPENDIX D
LOT-SCALE RAINWATER HARVESTING CALCULATIONS

This section provides a summary of rainwater harvesting calculations.

D-1 Catchment

- Net developable area = 46.35 ha (Jetty Road Urban Growth Plan)
- Lot density = 15 lots/ha
- No. of lots = 695
- Average lot size = 400 m^2
- % of roof area in each lot = 60% (Planning Practice Note 27 | Understanding the Residential Development Standards (ResCode))
- % of roof connected to rainwater tank = 50%
- Roof catchment = 60% x 50% x 695 x 400 m^2 = 8.34 ha

D-2 Reuse Demand

- Individual tank volume = 2 kL
- No. of tanks = 695
- Reuse demand type = daily demand
- Reuse application = toilet flushing
- Reuse demand per person = 20 L/day (Melbourne Water MUSIC modelling guidelines, 2018)
- Average household size = 2.6 persons/dwelling (Jetty Road Urban Growth Plan)
- Total reuse demand = 695 x 2.6 x 20 L/day = 36.14 kL/day

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