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WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Final Report

Jetty Road South of Rail Trail SWMS

APD Projects Pty Ltd

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Project Name	Jetty Road South of Rail Trail SWMS
Client	APD Projects Pty Ltd
Client Project Manager	Vanessa Italiano
Water Technology Project Manager	Lachlan Inglis
Water Technology Project Director	Johanna Theilemann
Authors	Gayani Chandrasena, Lachlan Inglis, Cameron Walsh
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51 Little Fyans Street
Geelong VIC 3220
Telephone (03) 8526 0800
ACN 093 377 283
ABN 60 093 377 283

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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.

Water Technology has prepared previous revisions of this Stormwater Management Strategy (SWMS) for the proposed development land. This revision of the SWMS has been amended in line with the authority discussion associated with the conceptual design, as outlined in Water Technology's letter[1] (dated 1/08/2023).

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 the requirements with 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[2] (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] Jetty Road South of Rail Trail SWMS – Drainage Asset – 22010206_L01v01b – 1 August 2023

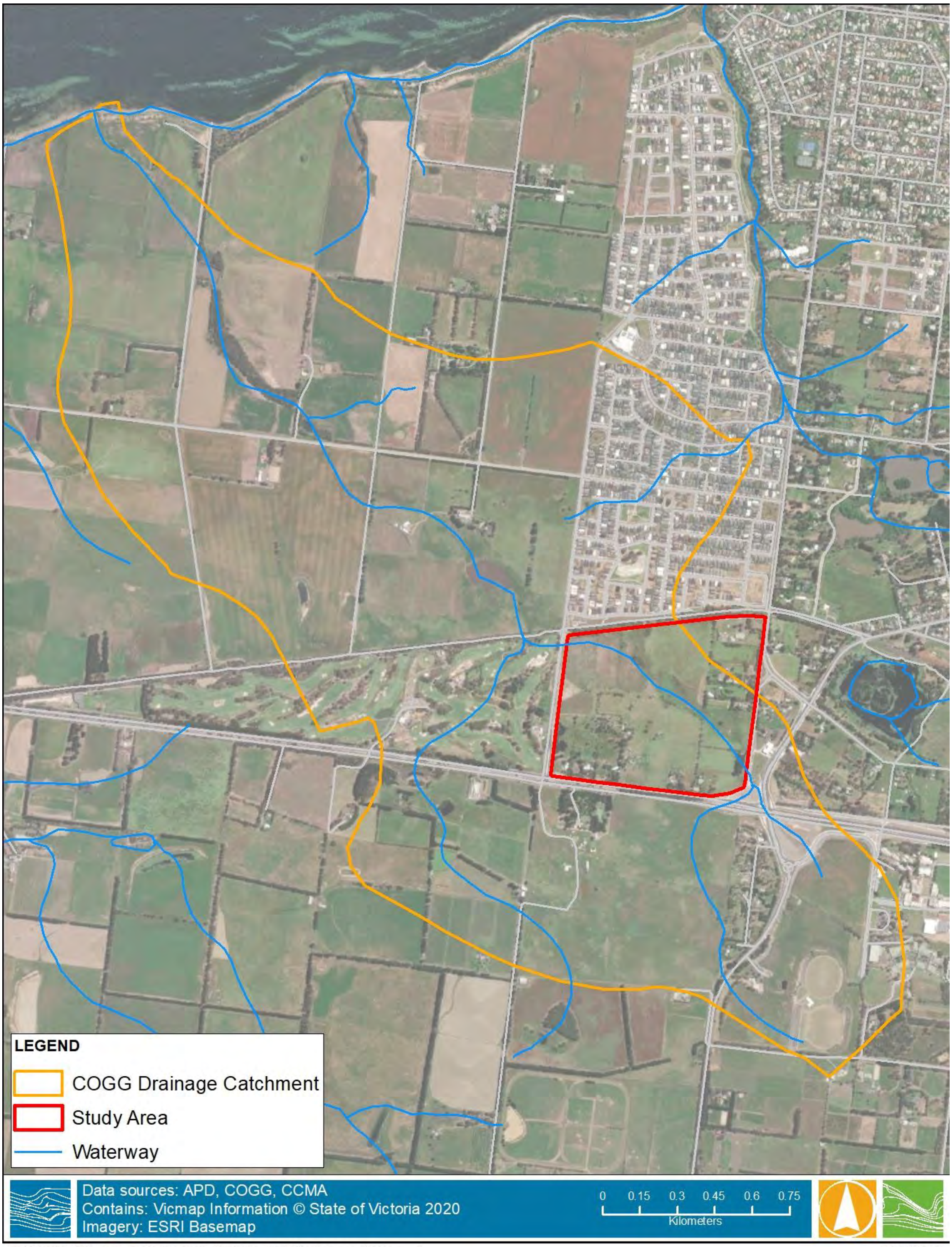

FIGURE 2-1 SUBJECT SITE

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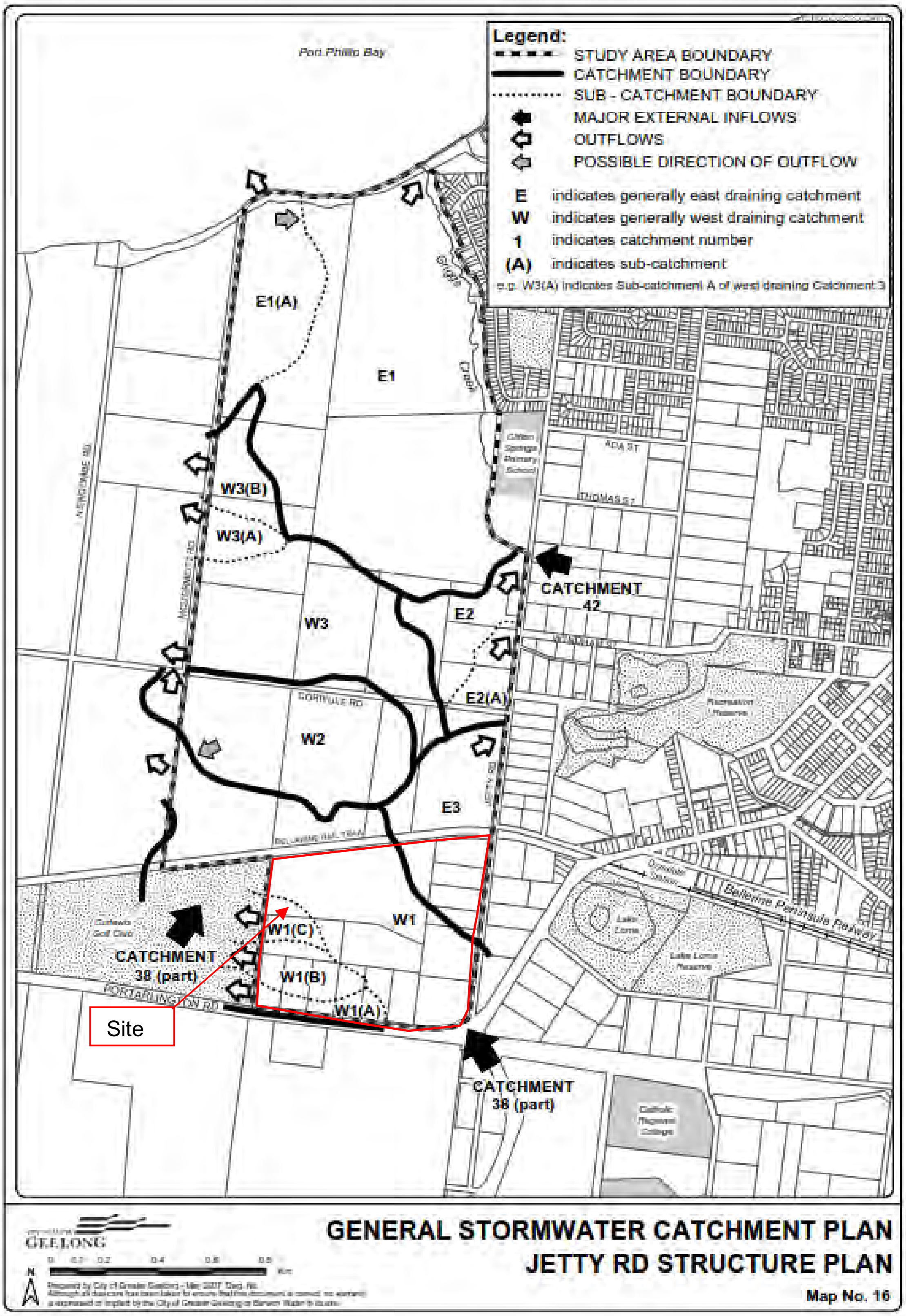

FIGURE 2-2 JETTY ROAD GROWTH AREA CATCHMENT PLAN

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[3] 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.

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

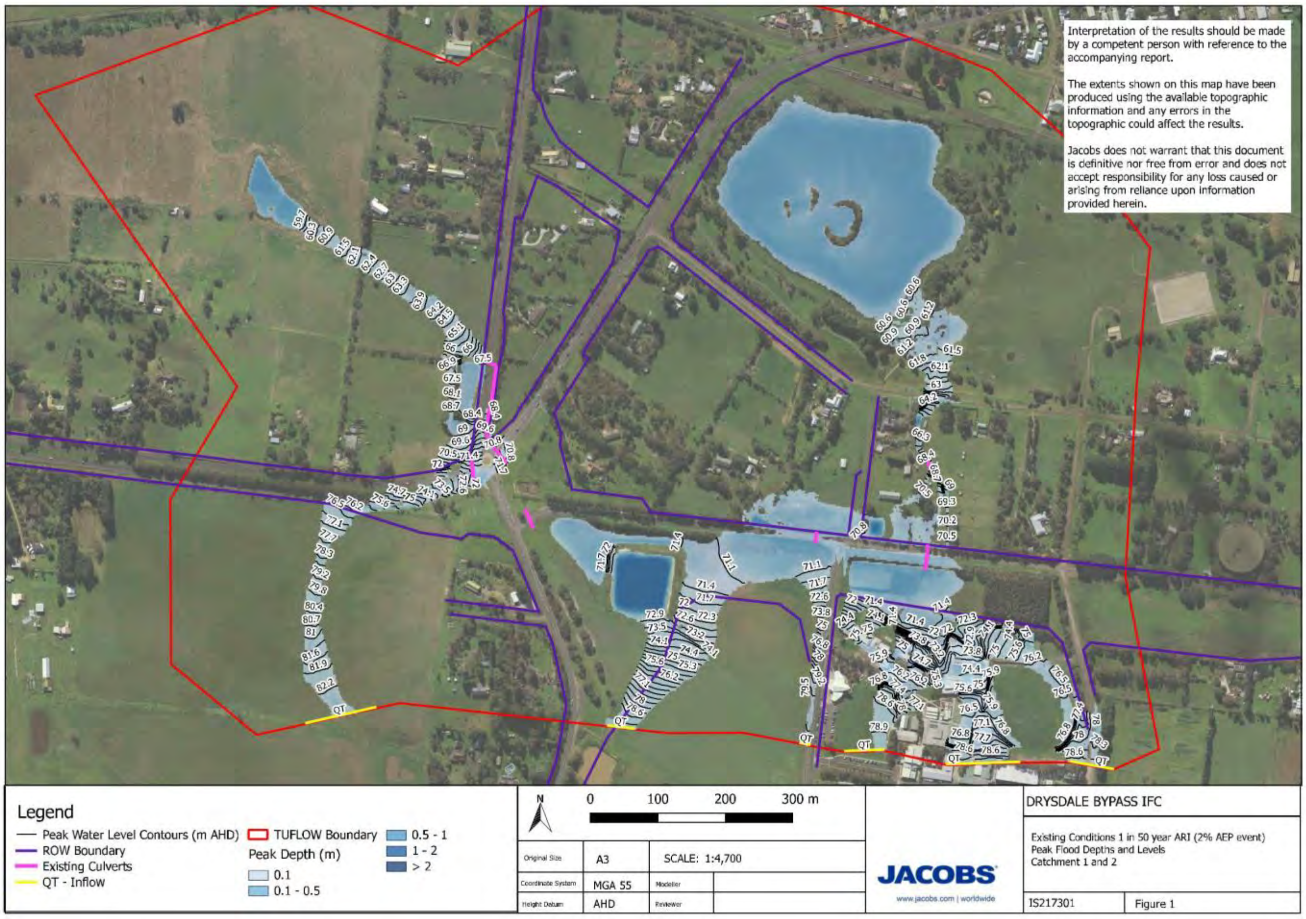

FIGURE 3-1 FLOOD MODELLING – POST DRYSDALE BYPASS WORKS (JACOBS FLOOD MODELLING[2])

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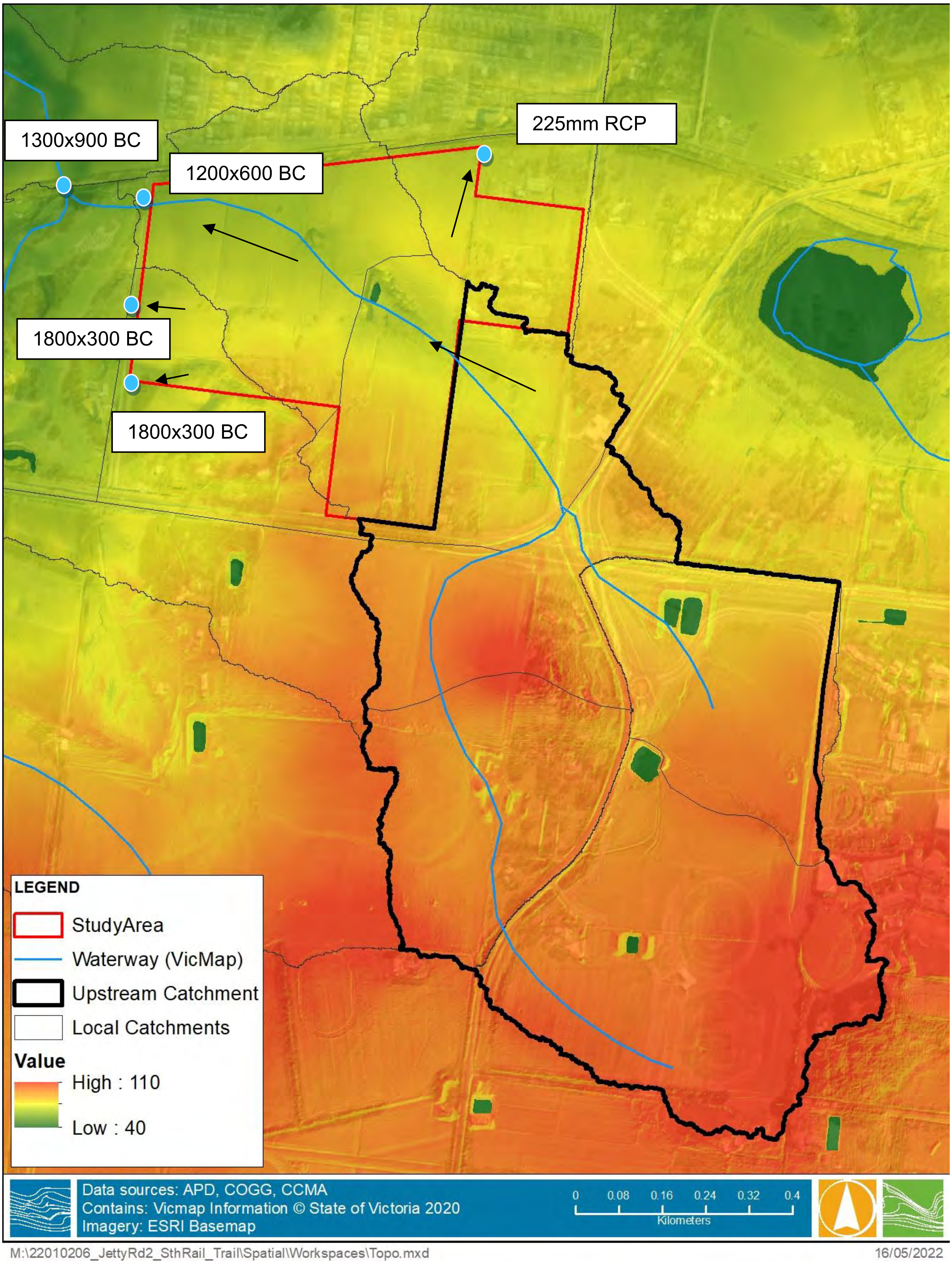

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FIGURE 3-2 UPSTREAM CATCHMENT (LIDAR FLOW PRIOR TO DRYSDALE BYPASS COMPLETION)

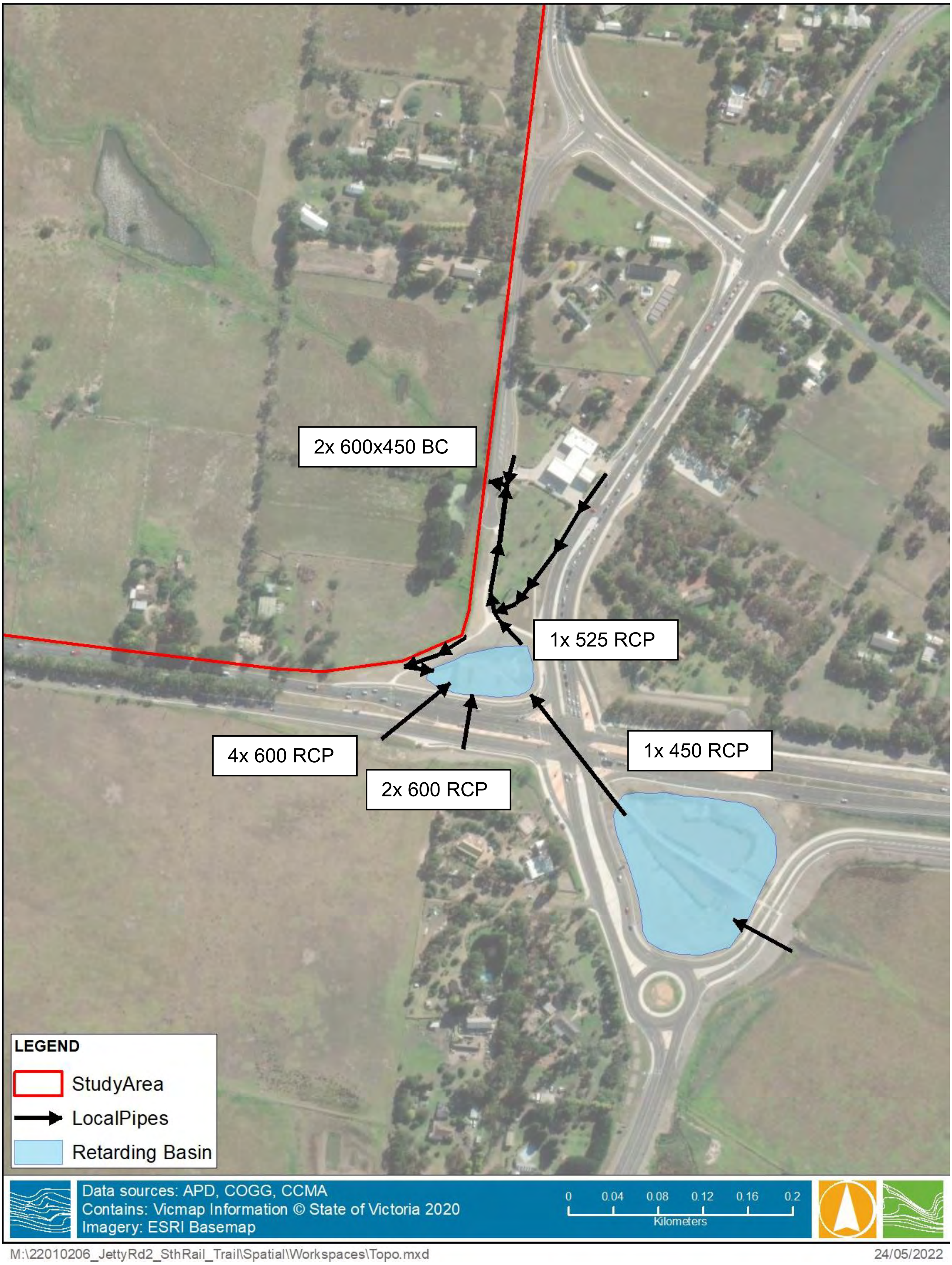

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FIGURE 3-3 DRYSDALE BYPASS CULVERTS AND BASINS

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[4] 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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[4] Jetty Rd Rezoning – Stage 2 Flood Study – TGM, prepared for Curlewis Bellarine Pty Ltd version 6, June 2020.

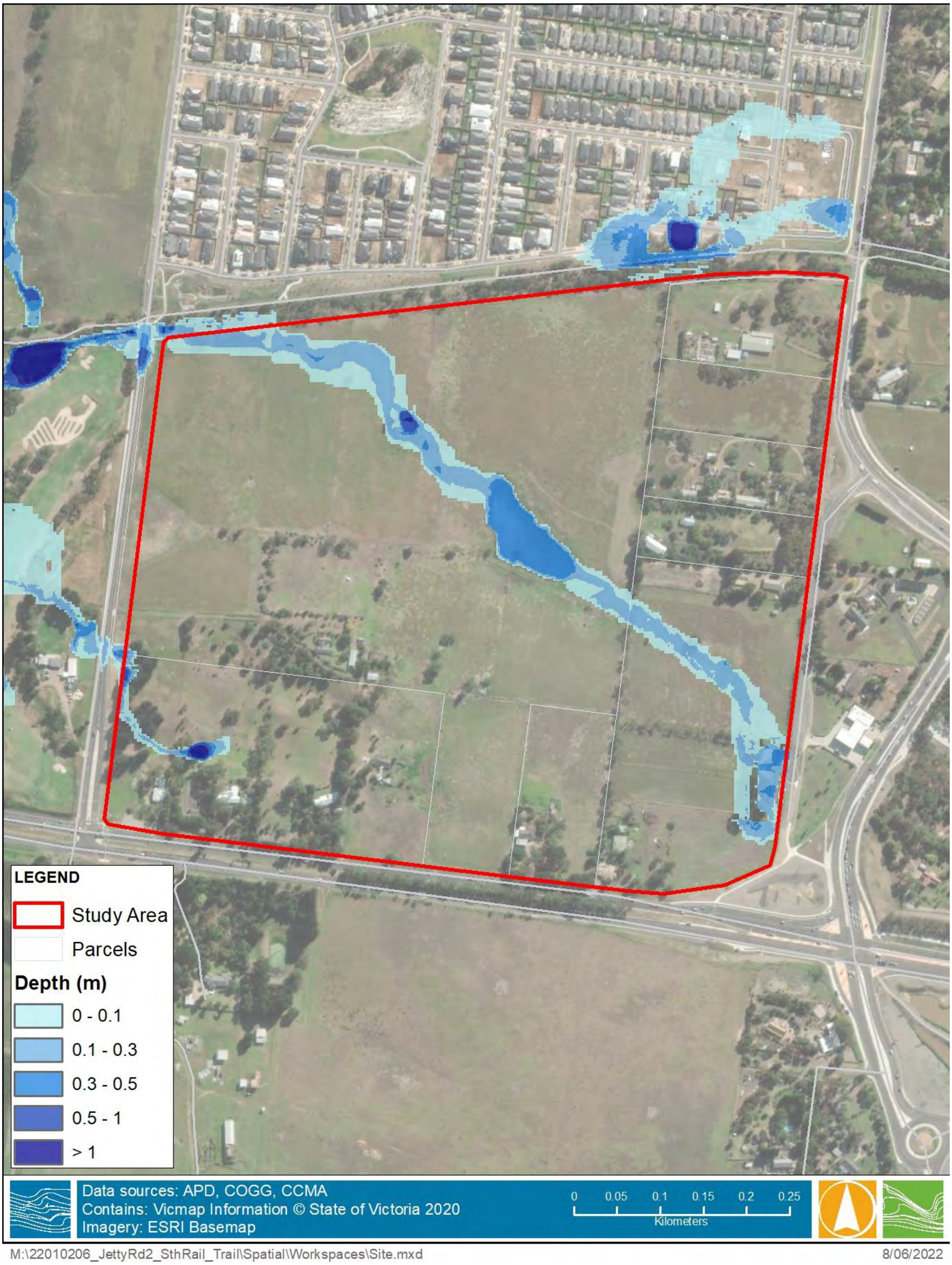

FIGURE 3-5 1% AEP FLOOD DEPTH – EXISTING CONDITIONS

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.

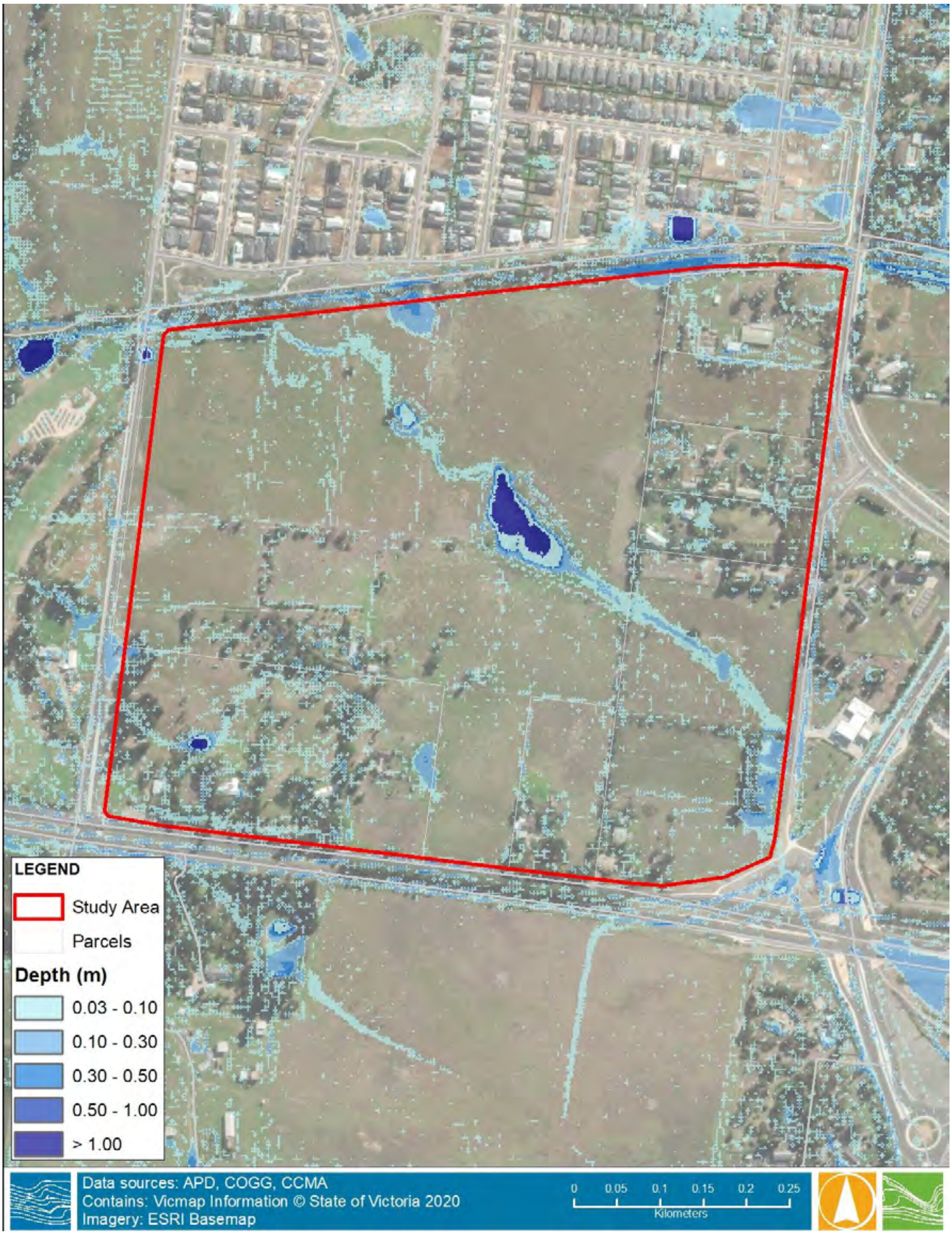

FIGURE 3-7 1% AEP RAIN ON GRID DEPTH RESULTS

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3.6 Developed Conditions

Developed conditions flood modelling is to be undertaken at later stages of the development process. 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.

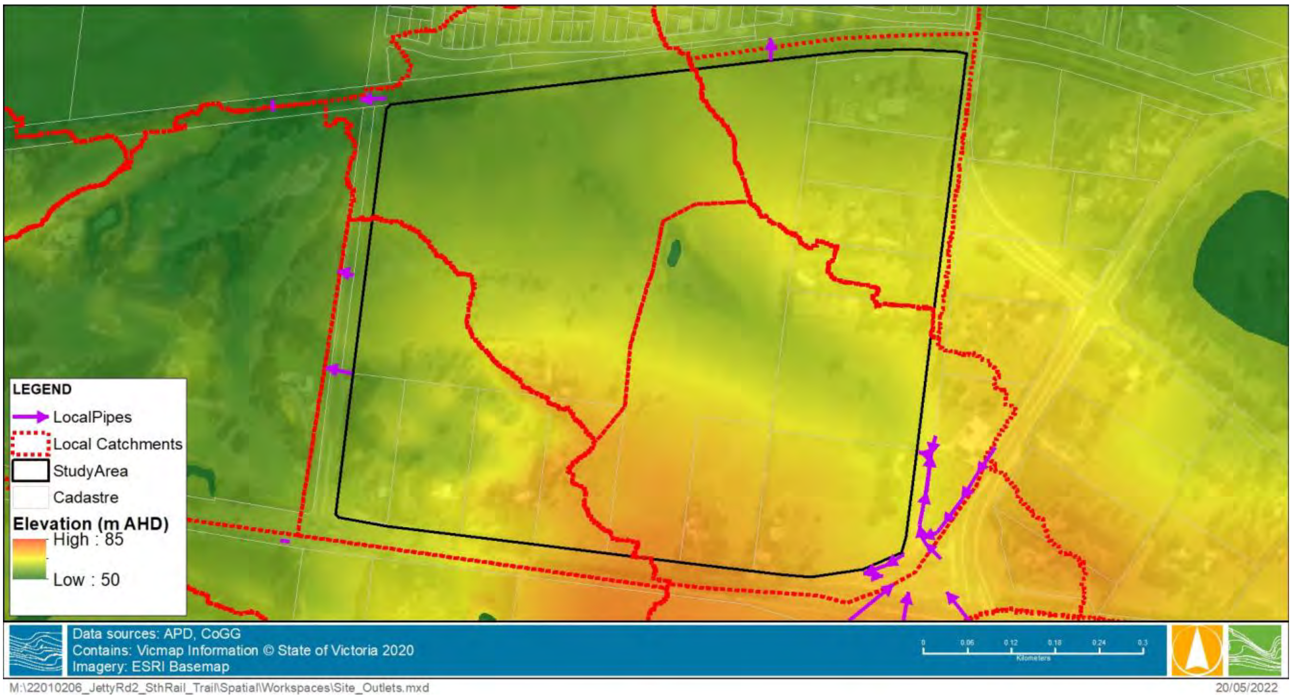

FIGURE 4-1 DRAINAGE OUTFALLS

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.

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)[5]. 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[6]. 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

[5] Jetty Road Rezoning - Stage 2 SWMS, prepared by Water Technology for Curlewis Bellarine Pty Ltd
[6] 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 & 7] 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.

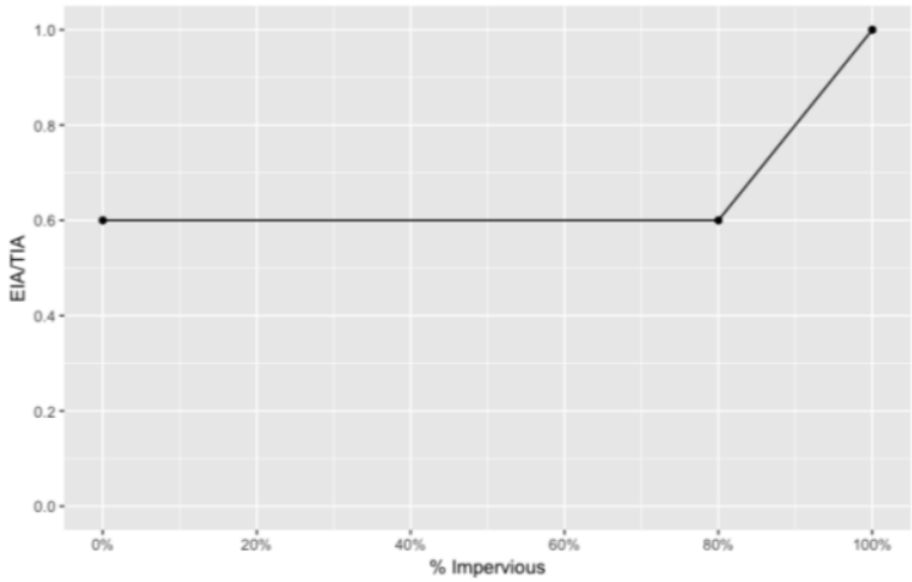

FIGURE 4-3 EIA/TIA RATIO INCREASE FOR HIGHLY IMPERVIOUS CATCHMENT

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

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[7] 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)[8] 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)[9]
  - A sensitivity check was conducted upon receiving the as-constructed plans provided by the Council. The 1% AEP flood storage of the basins B1 and B2 were modelled as 15,987 m^3 and 8,000 m^3 respectively. Outlet pipe lengths and slopes were also adjusted accordingly. Refer to Appendix A-3 for further details. A summary of modelling outcomes is presented below.
    - Modelling results indicate the basin B1 (Grubbs Rd wetland/retarding basin) flood storage exceeds the allocated volume at the 1% AEP event. It was assumed any runoff passing through the spillway would enter the downstream basin (B2).
    - The flood level within the downstream basin (B2) does not overtop in the 1% AEP event and all flow leaves the basin via the outlet pipe.
    - Nevertheless, the 1% AEP peak flow rate at the external catchment inflow for the site was estimated as 0.82 m^3/s whereas the original model results indicated a peak flow rate of 0.81 m^3/s. Since the change in inflows are insignificant, original model results were considered appropriate for the stormwater management plan development.
- 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 the external catchment flow inflow point at Jetty Road was compared with previous modelling. The previous study estimated the existing peak flow at this location to be 1.1 m^3/s (MRPV, 2019). The current RORB model result indicated a peak flow rate of 0.82 m^3/s at this location.

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

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 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 basins were included in the RORB model

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

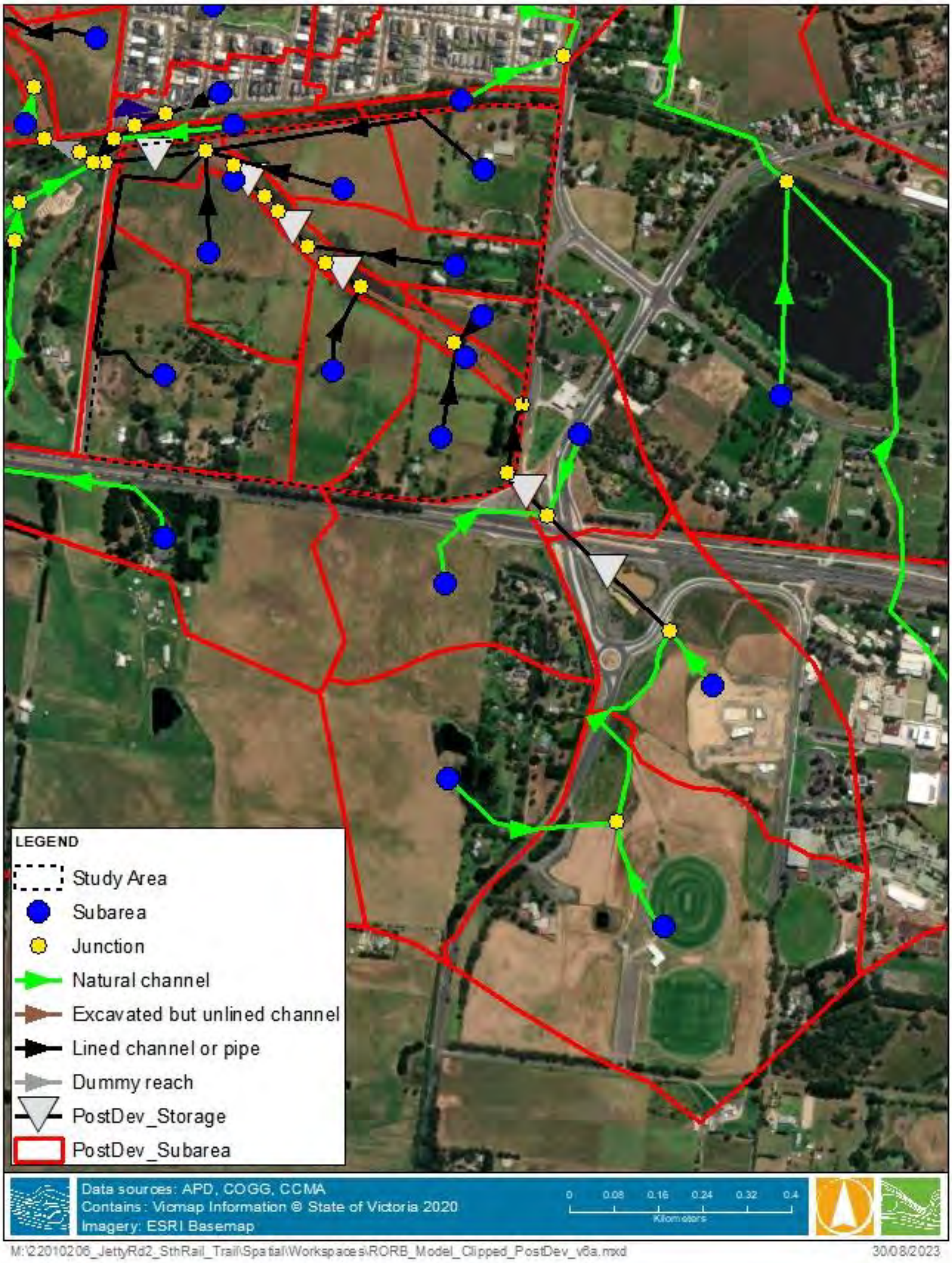

FIGURE 4-4 DEVELOPMENT CONDITIONS RORB MODEL CATCHMENT PLAN

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4.3 Asset and Catchment Strategy

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

The asset strategy was developed based on the SMEC development catchment strategy as per Figure 4-4.

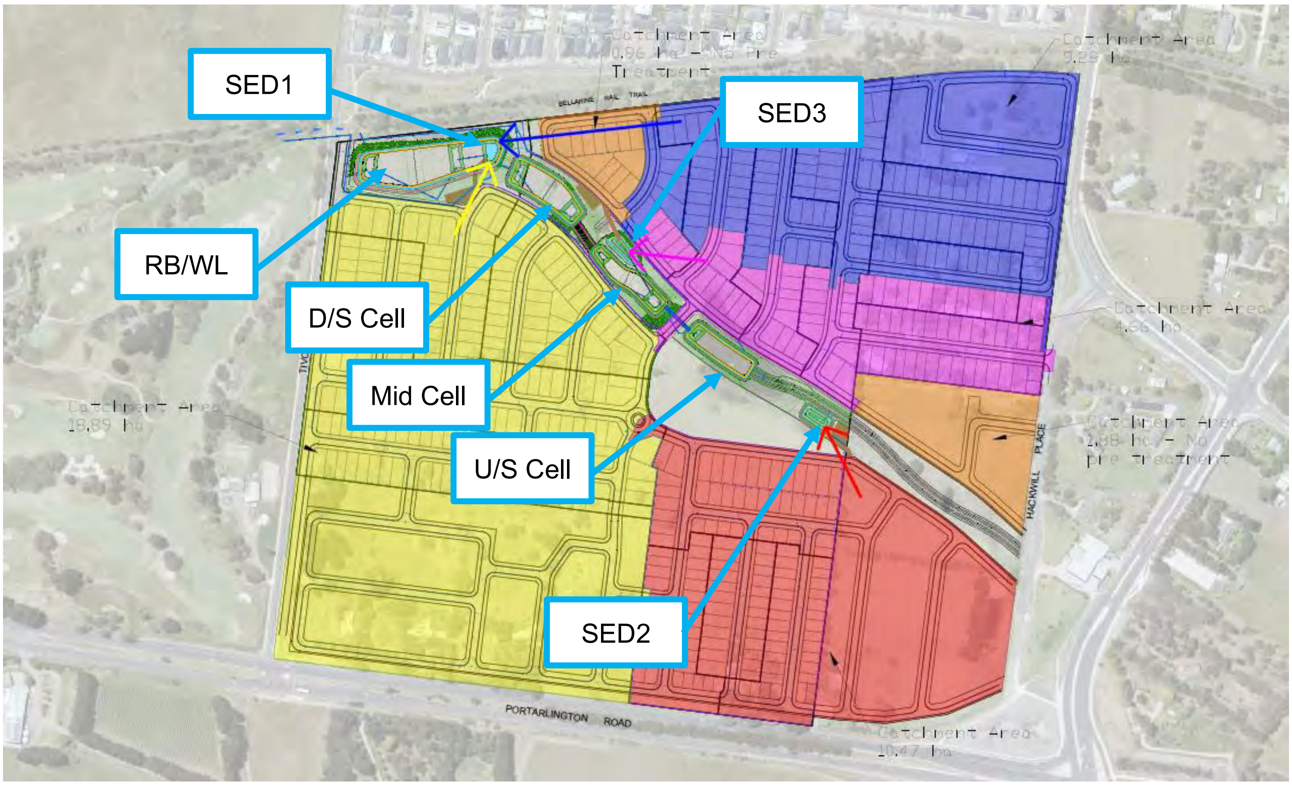

FIGURE 4-5 SMEC OVERALL DRAINAGE CATCHMENT EXTRACT

4.4 Asset Arrangement

The proposed stormwater management strategy for the site consists of:

- A retarding basin and wetland located at the northwest corner of the site:
  - Inclusive of a sedimentation basin
- A three-cell linear wetland located along the waterway alignment:
  - Inclusive of detention volume provided above NWL within the wetland cells.
  - Inclusive of sedimentation basin associated with the Mid Cell
- A sedimentation basin for the southeast development catchment

Table 4-5 presents a schematic layout of the proposed stormwater asset arrangement. Refer to appendices for SMEC conceptual plans.

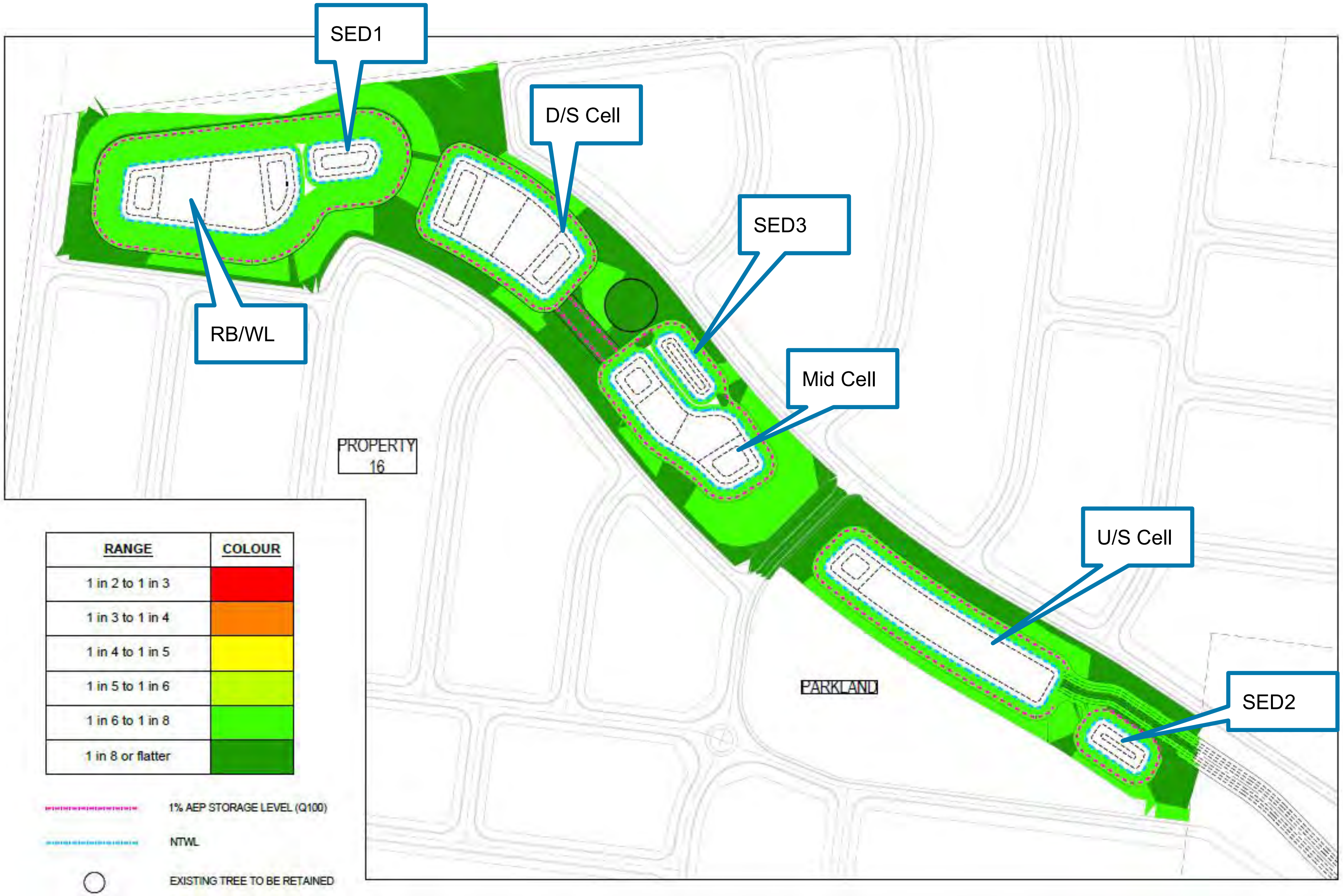

FIGURE 4-6 CONCEPT ASSET ARRANGEMENT SCHEMATIC (EXTRACT OF SMEC BATTER SLOPE PLAN)

4.5 Retarding Basin

Four retarding basins are proposed within the development to retard development peak flows leaving the site back to existing conditions peak flow rates. Consisting of one RB/WL located at the northwest corner of the site, and three additional detention volume storage areas are provided within each of the linear wetland cells.

The configurations of the proposed retarding basins to limit flows to target existing pre-development flow rates at the development site outlet are shown in Table 4-4 below.

TABLE 4-4 RETARDING BASIN DETAILS

Retarding Basin Details	RB/WL	D/S Cell	Mid Cell	U/S Cell
Storage Volume Available in SMEC conceptual design (m^3) (NWL to Q100)	11,966	2,489	1,873	2,581

The proposed retardation asset(s) function is accordance with the summaries below:

- Minor and major flows will enter the linear wetland retarding basin cells as piped flows and overland flows from the respective contributing development catchment and external catchments upstream of the retarding basins.
- Minor and major flows will enter the final RB/WL asset as piped flows and overland flows respectively from the contributing development catchment and linear wetland system upstream of the retarding basins.
  - The minor treated flow from the U/S linear wetland system will be bypassed around the final RB/WL system.
- The retarding basin has been designed to ensure it can cater for flows up to and including the 1% AEP.

Refer to Appendices for H-S-Q relationship, Controlling outlet pit computations, and SMEC conceptual design plans.

The overall detention performance is summarised in Table 4-5.

TABLE 4-5 RETARDING BASIN DETAILS

Item	Details
Upstream Area (ha)	**134**
Storage Volume (m^3)	Refer Table 4-4
Pre-development 1% AEP rate at RB/WL (m^3/s)	1.88
Peak 1% AEP RB/WL Inflow (m^3/s)	4.14
Peak 1% AEP RB/WL Outflow (m^3/s)	1.771
Peak 1% AEP Site Outflow (m^3/s)	1.777
Peak 1% AEP Water Depth in RB/WL (m)	2.03
Pre-development 10% rate at RB (m^3/s)	0.83
Peak 10% AEP RB/WL Inflow (m^3/s)	1.066
Peak 10% AEP RB/WL Outflow (m^3/s)	0.822

Item	Details
Peak 10% AEP Site Outflow (m^3/s)	0.828
Peak 10% AEP Water Depth in RB/WL (m)	1.26

* Upstream catchment flow significantly controlled due to basins at Jetty Road and the Drysdale Bypass

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[10], and Melbourne Water MUSIC guidelines[11]. 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.

5.1 Volume Management

The MUSIC model schematic utilized for investigate of runoff volume reduction opportunities is shown in Figure 5-1.

[10] City of Greater Geelong MUSIC Modelling Guidelines. Available at https://www.geelongaustralia.com.au/idm/documents/item/8cf4f273fe1120f.aspx
[11] 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

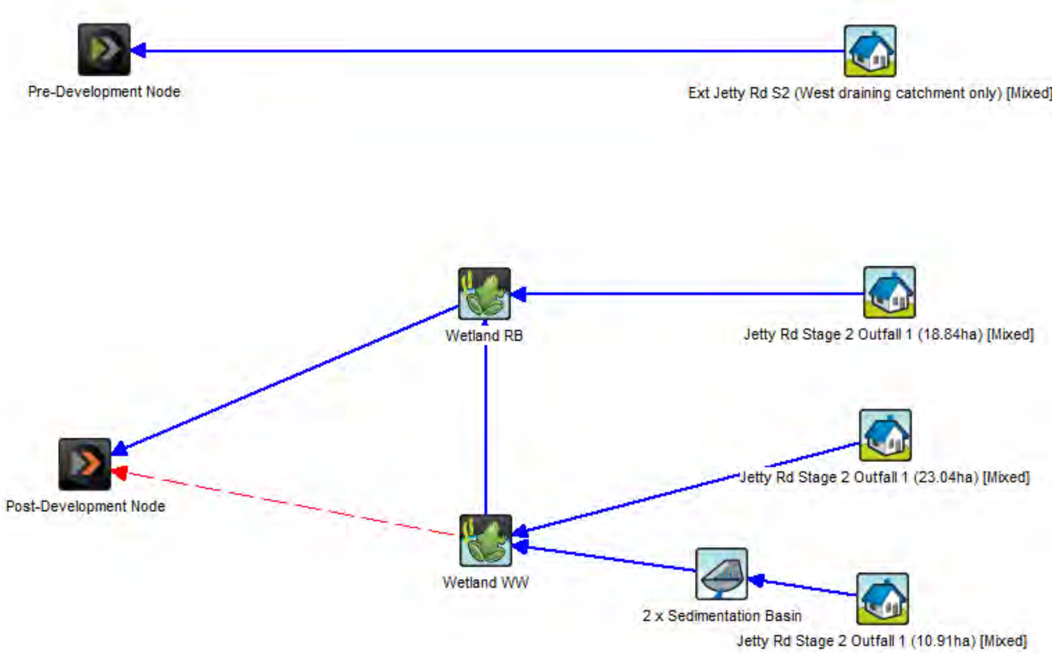

FIGURE 5-1 MUSIC MODEL VOLUME ANALYSIS SCHEMATIC

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.

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5.1.1 Runoff Volume Management

The post development stormwater runoff volume of 145 ML/year is higher than the existing conditions stormwater runoff volume leaving the site of 57 ML/year.. Therefore, additional measures will be required to reduce post development runoff volume by 88 ML/year. Four options have been assessed as part of the overall Jetty Road Stage 2 SWMS.

Option 1 – Lot Scale Intervention (Rainwater Tanks)

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 17 – 23 ha of turfed areas[12]. 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 88 ML/year to be lost via evaporation, MUSIC modelling results indicated that an evaporative basin with ~8 ha surface area is required to manage excess volumes. This does not appear to be a viable option in the overall development strategy.

Option 3 as a suitable volume management option along with the potential to investigate Options 1 & 2 at a later stage in the development design process.

5.2 WSUD Assets

The MUSIC model schematic utilised for designing and analysing the WSUD asset function is shown in Figure 5-1. This model was developed in line with the SMEC conceptual asset design plans (refer Appendix C).

[12] 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).

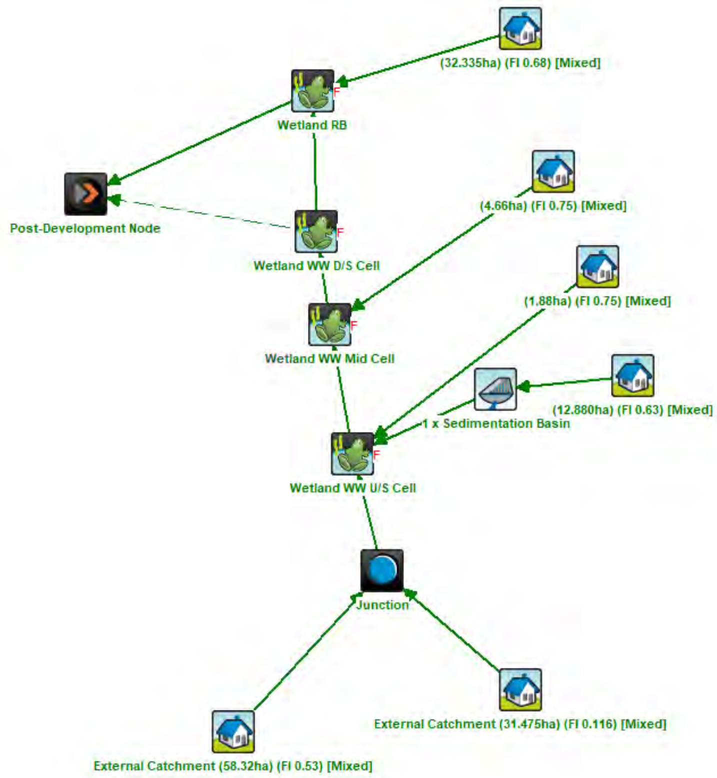

FIGURE 5-2 MUSIC MODEL WSUD ASSET SCHEMATIC

The linear wetland system in conjunction with sediment ponds was demonstrated to be meet BPEM guidelines. Refer to the Water Technology Memorandum[13] within the Appendix E for the detailed analysis of the asset functionality, inclusive of:

- Inundation frequency analysis
- Residency time analysis
- Controlling outlet structures
- H-S-Q relationships
- Sedimentation basin sizing
- Velocity computations
- MUSIC Auditor analysis
- Wetland Analysis Results

Asset size parameters are presented in Table 5-2.

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[13] Water Technology - 22010206_M01v04 – 19 July 2023

TABLE 5-2 KEY MODELLING PARAMETERS RELATED TO SEDIMENTATION PONDS AND WETLAND

Catchment	System	Surface Area at the NWL (m^2)	Extended Detention Depth (m)	Low Flow Bypass (m^3/s)	High Flow Bypass (m^3/s)	Permanent Pool Volume (PPV) (m^3)	Notional Detention Time (hrs)
1	RL/WL	2,915	0.35	0.0*	n/a	1,040	Na**
2a	D/S Cell	2,815	0.35	0.0	n/a	1,085	Na**
2a	Mid Cell	2,090	0.35	0.0	n/a	1,005	Na**
2b	U/S Cell	2,775	0.35	0.0	n/a	680	Na**
2b	Sed Basin (Sed2)	420	0.35	0.0	0.13	205	~12hrs

*The Linear Wetland Cell bypasses the treated 'pipe' flows around the final RB/WL node
**Refer to appendix for residence time analysis.

WSUD asset performance summary is presented in Table 5-3, this analysis represents the pollutant load reduction performance assuming the external catchment zoned for development is treated to BPEM

TABLE 5-3 TREATMENT TRAIN EFFECTIVENESS (ASSUMING BPEM EXTERNAL CATCHMENT TREATMENT)

Component	Total Inflow Loads (kg/yr)	Inflow Loads Produced by the Development (kg/yr)	Total Load Removed (kg/yr)	External BPEM Asset Load Removed (kg/yr)	Load Removed attributed to development (kg/yr)	Percentage Removal of Development Load
Total Suspended Solids	62,800	31,500	53,230	21,630	31,600	100%
Total Phosphorus	130	64.1	95	38.5	56	87%
Total Nitrogen	938	453	453	197	256	57%

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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)[15], *Wetland Design Manual*[16] 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)"

As outlined in section 4.3 of this report the proposed layout incorporates a constructed waterway within the upper section of the site before transitioning to a linear constructed wetland.

6.2 Waterway Corridor

The CCMA typically expects either a setback from the waterway or the delivery of a constructed waterway corridor in line with Melbourne Water's Waterway Corridors guidelines (Melbourne Water 2013)[17].

A constructed waterway approach is proposed for the development site.

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 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 6-1).

[15] Melbourne Water, 2019, Constructed Waterway Design Manual
[16] Melbourne Water, 2019, Wetland Design Manual
[17] Melbourne Water, 2013, Waterway Corridors: guidelines for greenfield development areas within the Port Phillip and Westernport region

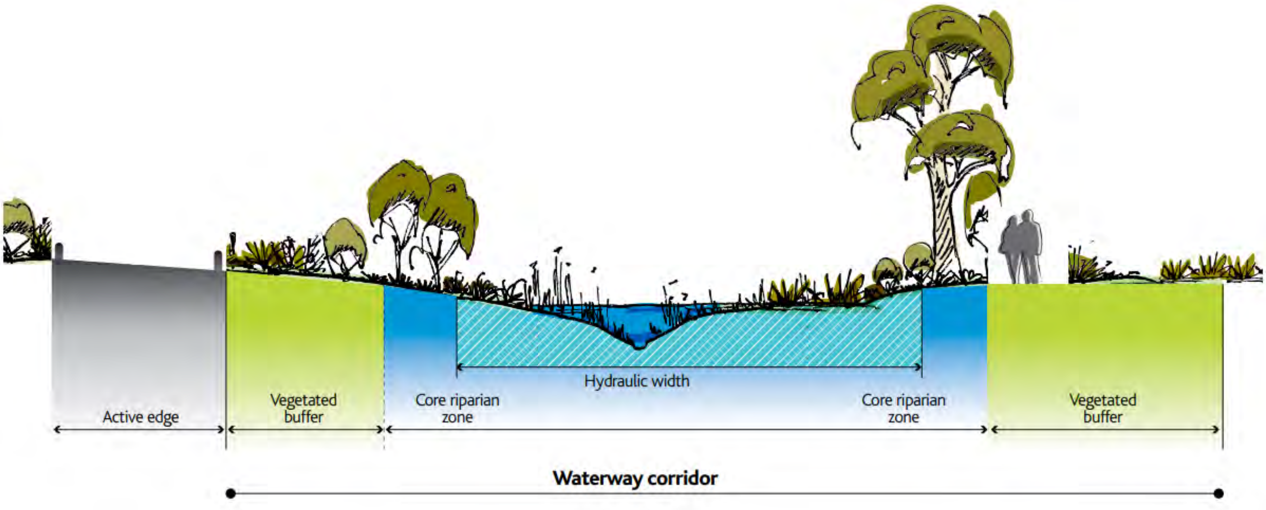

FIGURE 6-1 EXAMPLE OF SETBACK SUBZONES FOR CONSTRUCTED WATERWAYS (MELBOURNE WATER, 2013)

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 2.8 m^3/s;
  - This would result in a hydraulic width of 17.3 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).

6.4 Typical Cross-section and Longitudinal Grade

Figure 6-2 shows a typical cross section of a 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*.

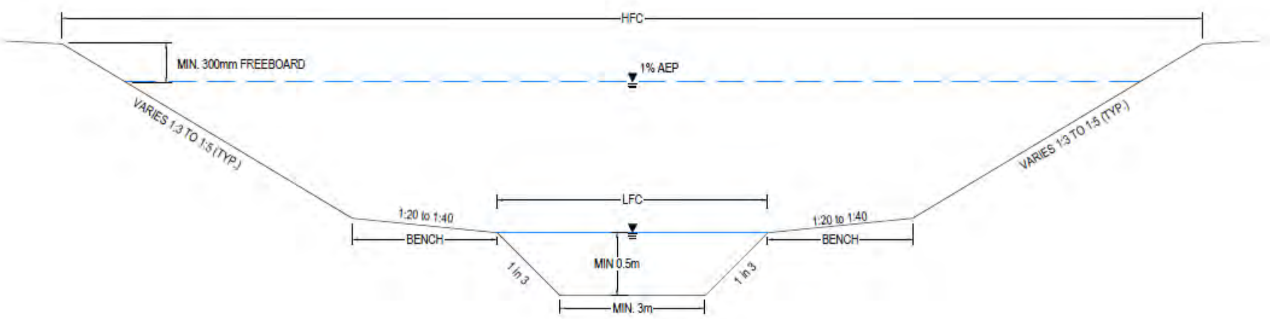

FIGURE 6-2 COMPOUND WATERWAY TYPICAL CROSS-SECTION

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 6-3. 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.

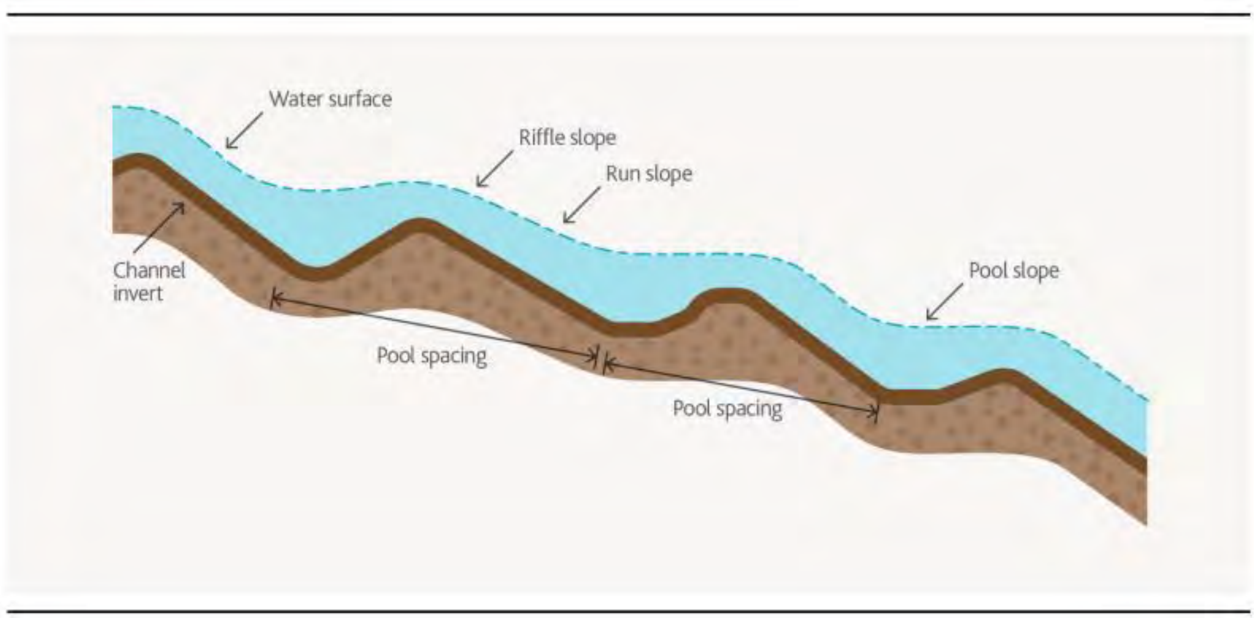

FIGURE 6-3 TYPICAL RIFFLE-POOL SEQUENCE (MW CONSTRUCTED WATERWAY DESIGN MANUAL, 2019)

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 17.3 m width would be sufficient to convey the 1% AEP peak flow of 2.8 m^3/s (the unmitigated post-development runoff entering the upstream detention asset), as shown in Figure 6-4:
  - 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 6-4).

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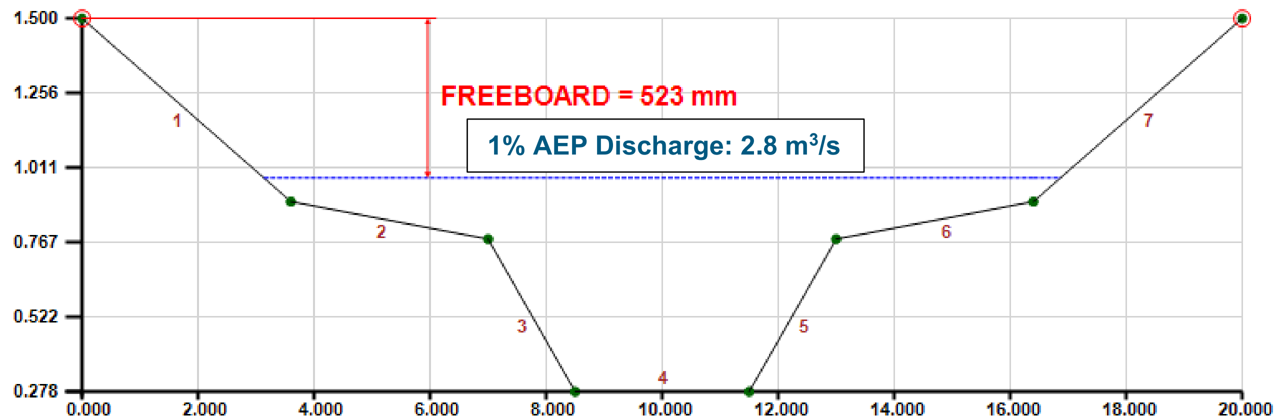

FIGURE 6-4 HFC & LFC PC CONVEY ANALYSIS (1% AEP & 1 EY FLOW)

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 0.66 m/s based on a longitudinal grade of 1 in 200 based on the proposed constructed waterway slope.

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 lot layout and previous Masterplan, there is the need to provide a waterway crossing within the site (Figure 6-5). The crossings will be incorporated into the linear wetland asset system and should be designed in accordance with associated CCMA works on waterway and waterway crossing guidelines. The indicative location proposed in Figure 6-5 shows that stormwater runoff from some parts of the site will have entered the constructed waterway and linear wetland system. Culvert preliminary 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.

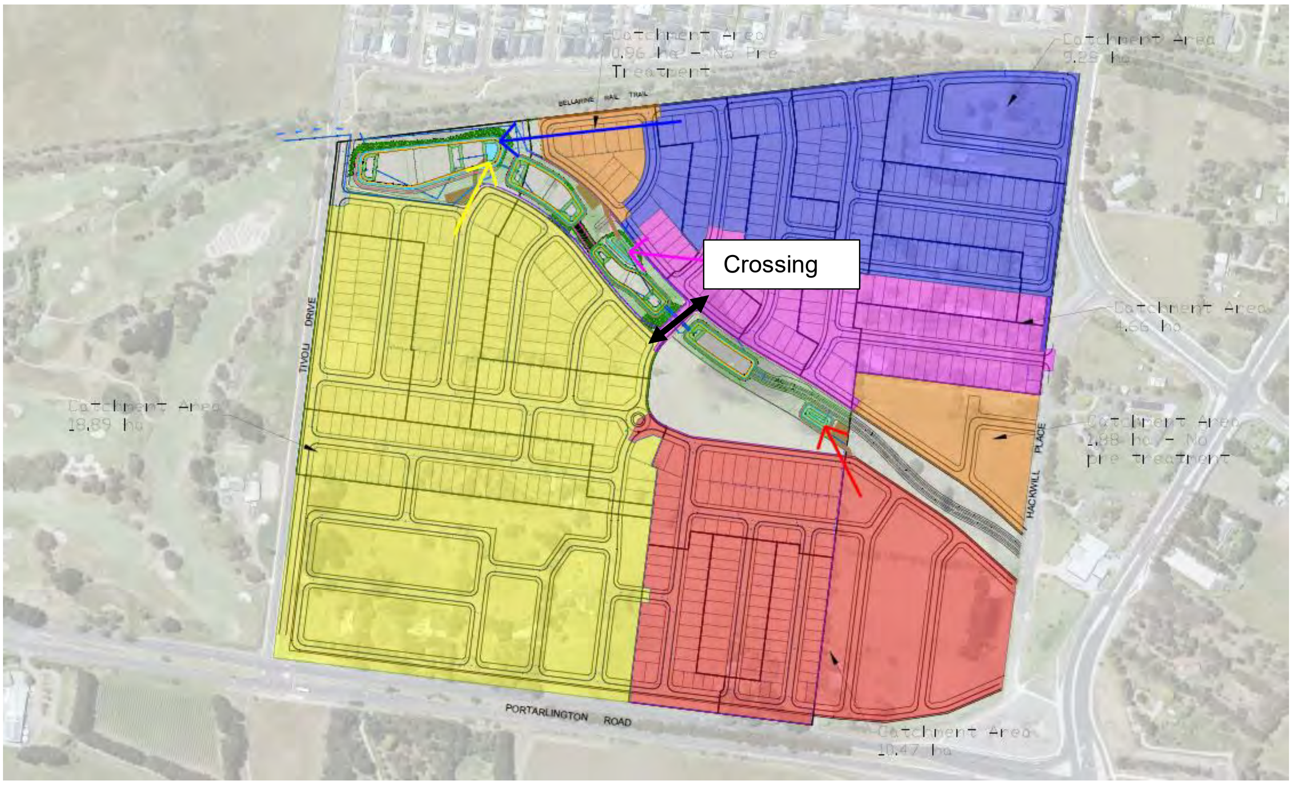

FIGURE 6-5 WATERWAY CROSSINGS

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. This conservative analysis does not account for detention of peak flows which is provided in the linear wetland cell directly upstream of the crossing location.

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.

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 6-6.

It is suitable for this preliminary culvert sizing to be amended as part of the linear wetland asset detail design process.

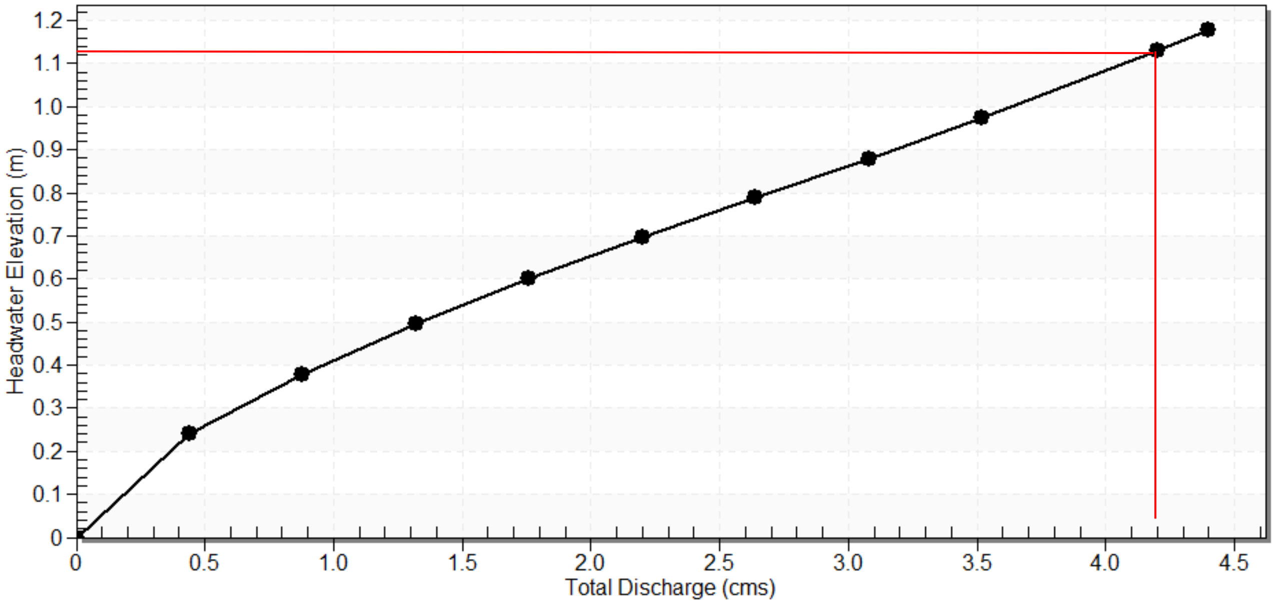

FIGURE 6-6 CROSSING 2 RATING CURVE

6.6 Wetland and Sedimentation Ponds Design

In line with discussions with the City of Great Geelong and the CCMA, it is proposed to incorporate a linear wetland and three sedimentation ponds within the waterway corridor. A linear wetland concept has been developed incorporating the following additional operational and maintenance considerations:

- Ability to isolate sediment ponds from the main wetland cells to draw-down for maintenance without having to empty the macrophyte zone.
- Ability to drain each macrophyte zone without having to empty the other macrophyte zones and sediment ponds.
- Maintain maximum velocity below 0.5m/s within the macrophyte zone and sediment ponds during the 1% AEP storm event.

The Extended detention depth of sediment ponds and macrophyte zone were set to 350 mm. The concept design plans and velocity checks calculations are provided in the Appendix E. The Functional Design Report & Melbourne Water Deemed to Comply Checklist to be supplied as part of future engineering approvals.

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 and meet the Melbourne Water target flow velocities for with the 1%AEP and 3 month events.

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. Additionally, the use of the maintenance bypass pipe can be adopted throughout the establishment phase in order to minimise flow volumes and velocities through the wetland system if desired.

6.6.2 Sedimentation Pond Maintenance

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

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. SMEC have undertaken preliminary design work to undertake an upgrade of the existing drainage along the southern side of the rail embankment adjacent to the golf course located on crown land, shown in (Figure 7-2).

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

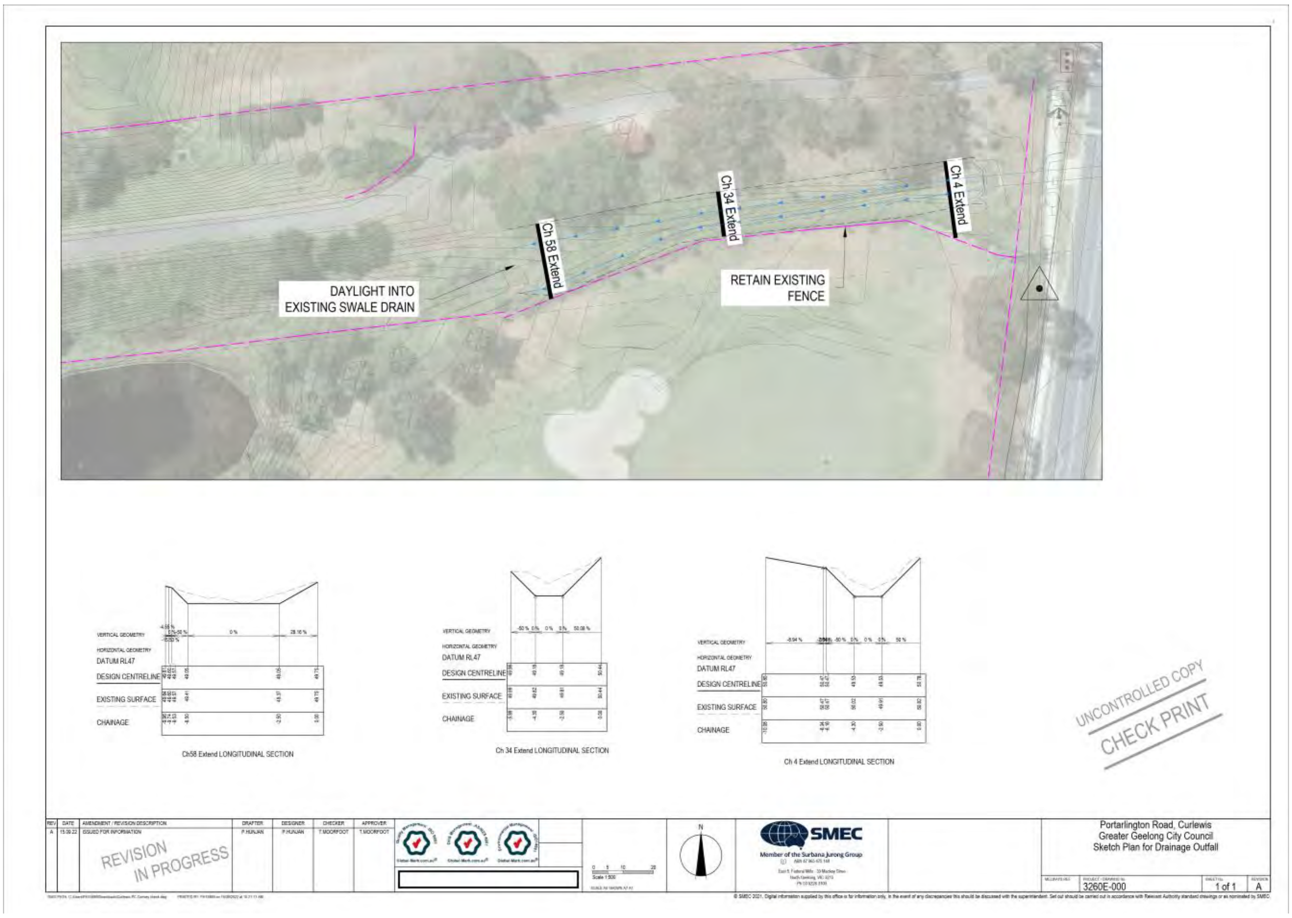

FIGURE 7-2 PROPOSED GOLF COURSE OUTFALL (SOURCE: SMEC, OCT-2022)

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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 discussions with 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.

Four retarding basin assets are proposed along the waterway corridor 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.

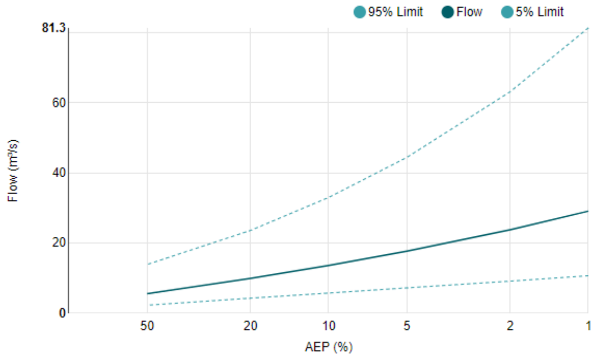
The existing waterway immediately downstream of the proposed constructed waterway has sufficient capacity to convey post-development flows with the recommendation for minor earthworks within the rail reserve adjacent to the golf course to maintain the flow within the reserve.

The proposed WSUD strategy to meet the BPEM target involves linear wetland assets along the waterway corridor. The functionality of this asset strategy has been developed through a series of discussions with the City of Greater Geelong, ultimately culminating in the current SMEC conceptual design arrangement which addressed the relevant guidelines and authority design constraints.

APPENDIX A
RORB MODEL PRE-VALIDATION

A-1 Regional Flood Frequency Estimate

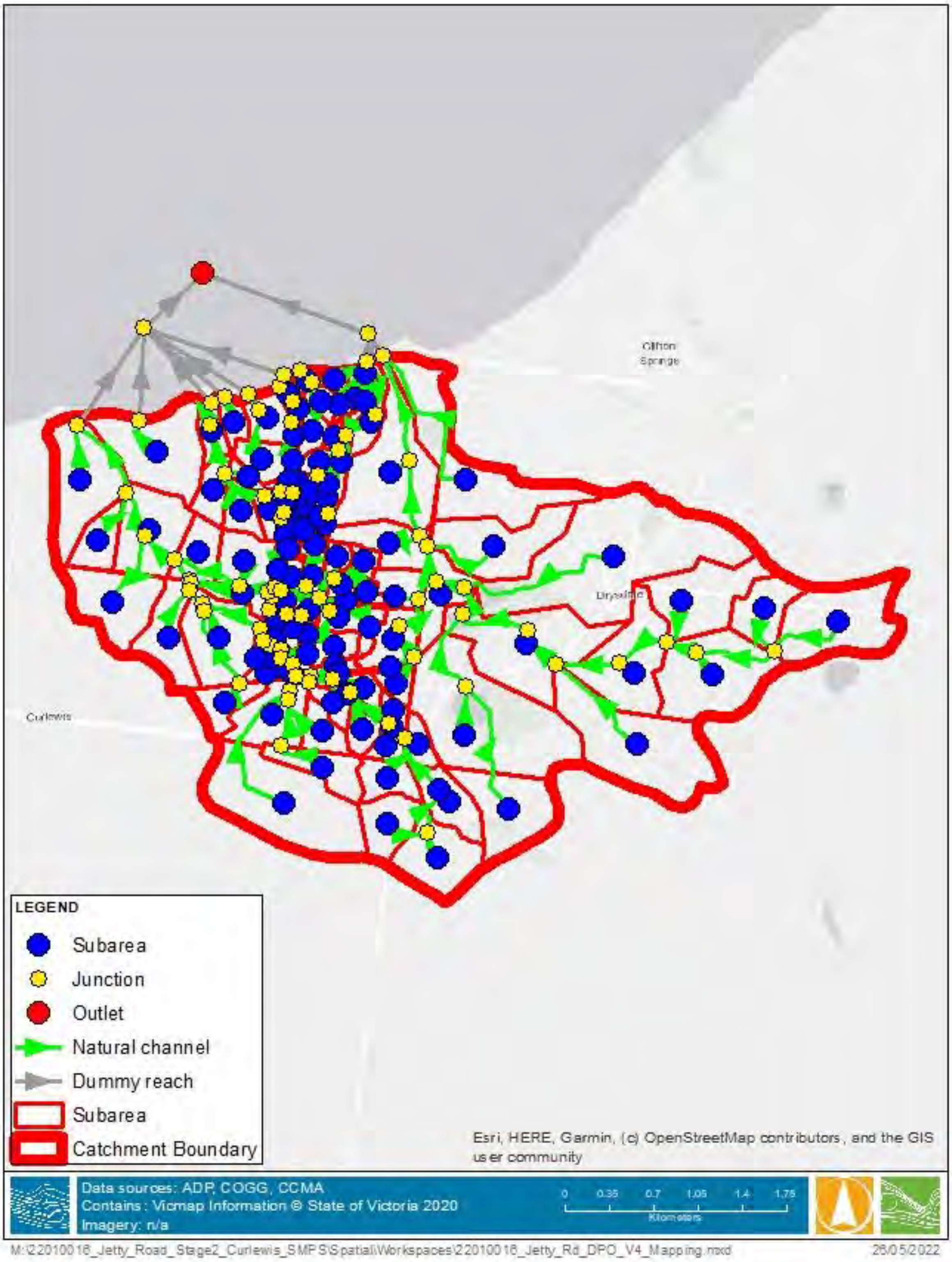
Results | Regional Flood Frequency Estimation Model



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

A-3 RORB Model Sensitivity Check

Design parameters of the two Drysdale Bypass basins (Jetty Road Basin and Grubbs Road Basin) were not available at the time of original RORB modelling was undertaken. Upon Council review of the draft SWMP, issued for constructed drawings and the detailed design report was made available. A sensitivity check was undertaken by updating the basin details in the existing scenario RORB model. This section provides a summary of model changes. Overall, changes resulted in only minor differences the 1% AEP peak flow rate into the site which are not likely to impact on the sizing of assets at this design stage.

TABLE A-1 BASIN B1 (GRUBBS ROAD BASIN)

Model Parameter	Original SWMP Model	Sensitivity Check
Max Height (m)	5	1.3
Max Volume (m^3)	59,125	15,987
Outlet pipe • Length (m) • Slope (%) • No. of pipes • Pipe diameter (m)	50 1 1 0.450	127 0.4 1 0.450
Spillway • Crest (m) • Width (m)	n/a	1.3 5

TABLE A-2 BASIN B2 (JETTY ROAD BASIN)

Model Parameter	Original SWMP Model	Sensitivity Check
Max Height (m)	6	3.4
Max Volume (m^3)	18,000	8,000
Outlet pipe • Length (m) • Slope (%) • No. of pipes • Pipe diameter (m)	25 1 1 0.525	16 1.71 1 .525
Spillway • Crest (m) • Width (m)	n/a	3.4 5

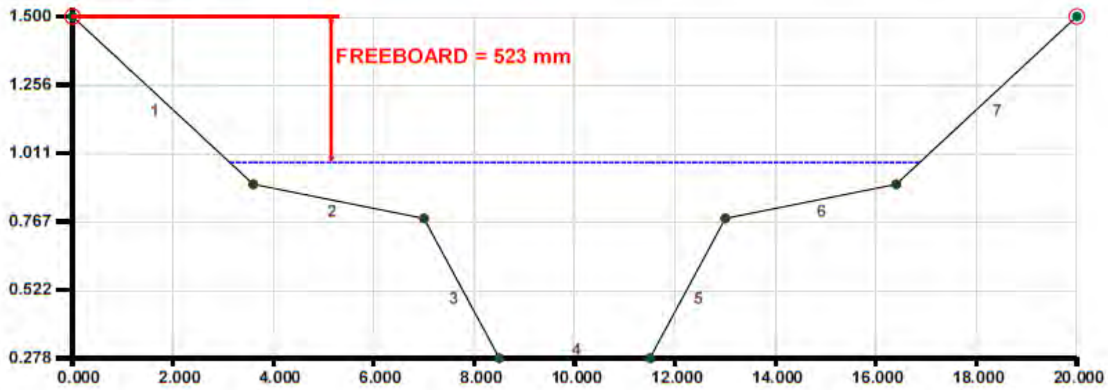
TABLE A-3 MODEL RESUTLS COMPARISON - 1% AEP EVENT EXISTING CONDITIONS

Model Parameter	Original SWMP Model	Sensitivity Check
Basin B1 • Peak flood storage (m^3) • Peak outflow rate (m^3/s)	 21,100 0.76	 23,700 0.43
Basin B2 • Peak flood storage (m^3) • Peak outflow rate (m^3/s)	 5,500 0.81	 3,960 0.82
Peak flow rate at the Jetty Road (m^3/s)	0.81	0.82
Peak flow rate at the Outfall 1 (m^3/s)	1.83	1.80

APPENDIX B
PC CONVEY ANALYSIS - WATERWAY

PROJECT: Jetty Rd Waterway
Comment
Print-out date: 29/08/2023 - Time: 3:31
Data File: 22010016_Waterway_1AEP_HFC.dat

1. CROSS-SECTION



2. DISCHARGE INFORMATION

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

Required overland / channel / watercourse discharge = 2.8 cumecs

3. RESULTS Water surface elevation = 0.978 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 (cumecs):	0.000	2.921	0.000	2.921
D(Max) = Max. Depth (m):	0.000	0.700	0.000	0.700
D(Ave) = Ave. Depth (m):	0.000	0.323	0.000	0.323
V = Ave. Velocity (m/s):	0.000	0.659	0.000	0.659
D(Max) x V (cumecs/m):	0.000	0.462	0.000	0.462
D(Ave) x V (cumecs/m):	0.000	0.213	0.000	0.213
Froude Number:	0.000	0.371	0.000	0.371
Area (m^2):	0.000	4.430	0.000	4.430
Wetted Perimeter (m):	0.000	13.910	0.000	13.910
Flow Width (m):	0.000	13.730	0.000	13.730
Hydraulic Radius (m):	0.000	0.318	0.000	0.318
Composite Manning's n:	0.000	0.050	0.000	0.050
Split Flow?	-	-	-	No

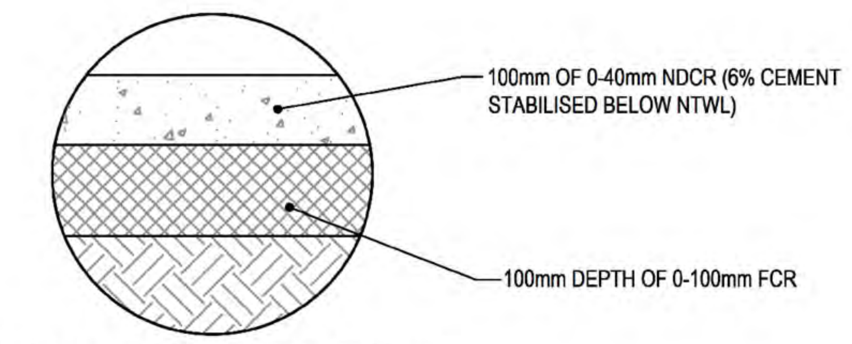
4. CROSS-SECTION DATA

	LEFT HAND POINT		RIGHT HAND POINT		
SEGMENT NO.	CHAINAGE (m)	R.L. (m)	CHAINAGE (m)	R.L. (m)	MANNING'S N
1	0.000	1.500	3.600	0.900	0.050
2	3.600	0.900	7.000	0.778	0.050
3	7.000	0.778	8.500	0.278	0.050
4	8.500	0.278	11.500	0.278	0.050
5	11.500	0.278	13.000	0.778	0.050
6	13.000	0.778	16.400	0.900	0.050
7	16.400	0.900	20.000	1.500	0.050

APPENDIX C
CONCEPTUAL DESIGN

LEGEND

- SEDIMENT POND
- ACCESS TRACK & DRYING BENCH
- D_{50} 300 ROCK BEACHING
- 1% AEP STORAGE LEVEL (Q100)
- NTWL (VARIES)
- FLOW DIRECTION
- SAFETY BENCH
- 53.0 DESIGN CONTOUR
- DRAINAGE PIPES
- 2.5m SHARED PATH
- EXISTING TREE TO BE RETAINED (TPZ)



TYPICAL ACCESS TRACK & DRYING BENCH DETAIL
200mm DEPTH

NOTE
3D DESIGN SURFACE FILE WILL BE AVAILABLE TO CONTRACTOR TO ASSIST IN SETOUT AND ACHIEVING ACCEPTABLE CONSTRUCTION TOLERANCES

RESERVE FENCING
Where shown on drawings, white cypress post and rail fencing, harvested from sustainable forest (1.2m high), is to be constructed across the Council Reserve boundaries. Refer to Council Standard Drawing CGG709. A demountable section for vehicle access must be installed within post and rial fencing to Council Standards. Refer to Council Standard Drawings CGG702 & CGG703

WARNING
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C	25/01/23	WETLANDS AMENDED	B.BARBER	B.BARBER	R.FORBES	T.MOORFOOT
D	10/02/23	WETLANDS AMENDED	M.SURRAO	B.BARBER	R.FORBES	T.MOORFOOT
E	28/02/23	WETLANDS & SED BASIN AMENDED	M.SURRAO	R.FORBES	R.FORBES	T.MOORFOOT
F	22/06/23	WETLANDS & SED BASINS AMENDED	M.SURRAO	R.FORBES	R.FORBES	T.MOORFOOT
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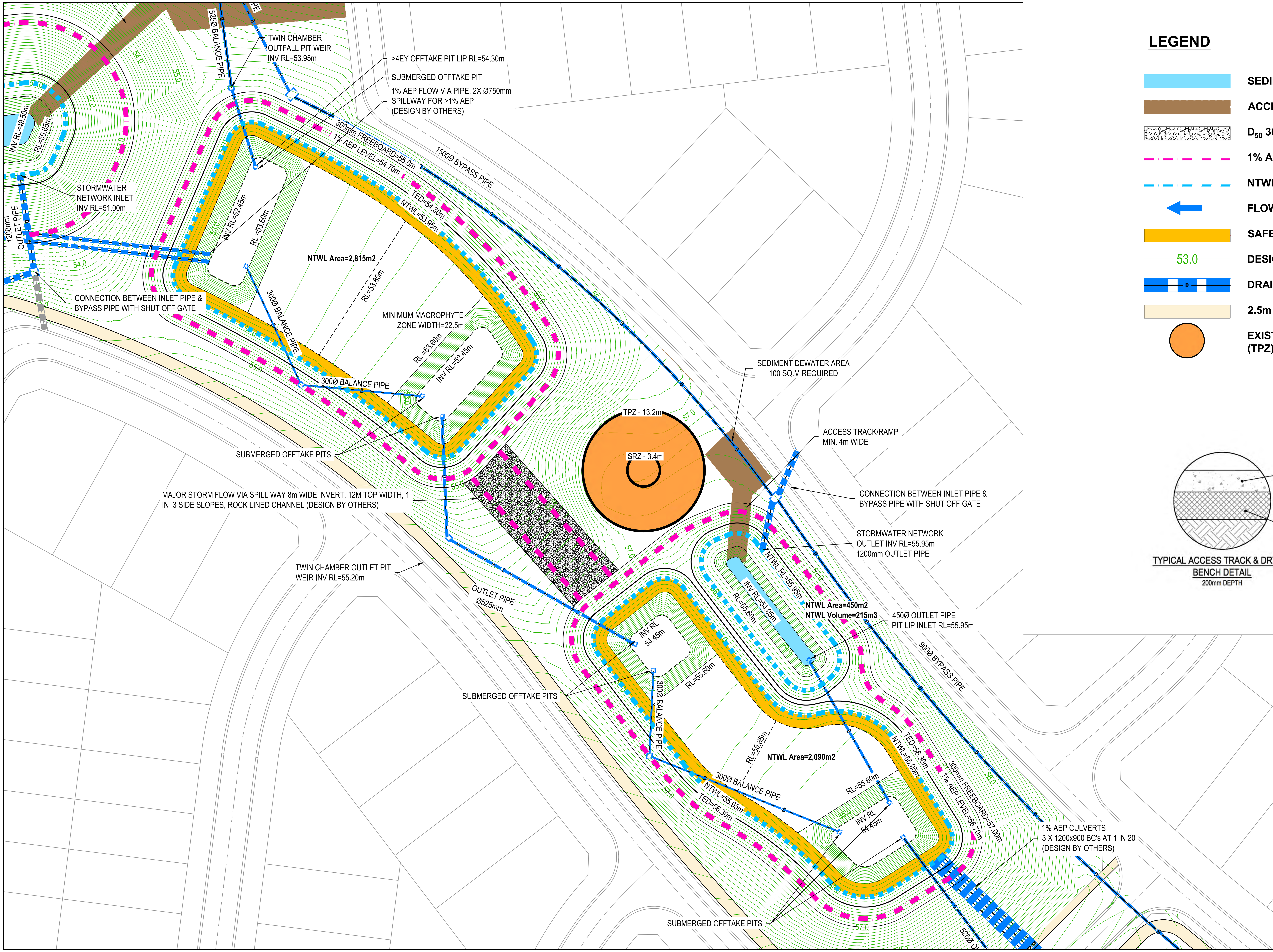

LEGEND

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- 1% AEP STORAGE LEVEL (Q100)
- NTWL (VARIES)
- FLOW DIRECTION
- SAFETY BENCH
- 53.0 DESIGN CONTOUR
- DRAINAGE PIPES
- 2.5m SHARED PATH
- EXISTING TREE TO BE RETAINED (TPZ)

100mm OF 0-40mm NDCR (6% CEMENT STABILISED BELOW NTWL)
100mm DEPTH OF 0-100mm FCR

TYPICAL ACCESS TRACK & DRYING BENCH DETAIL
200mm DEPTH

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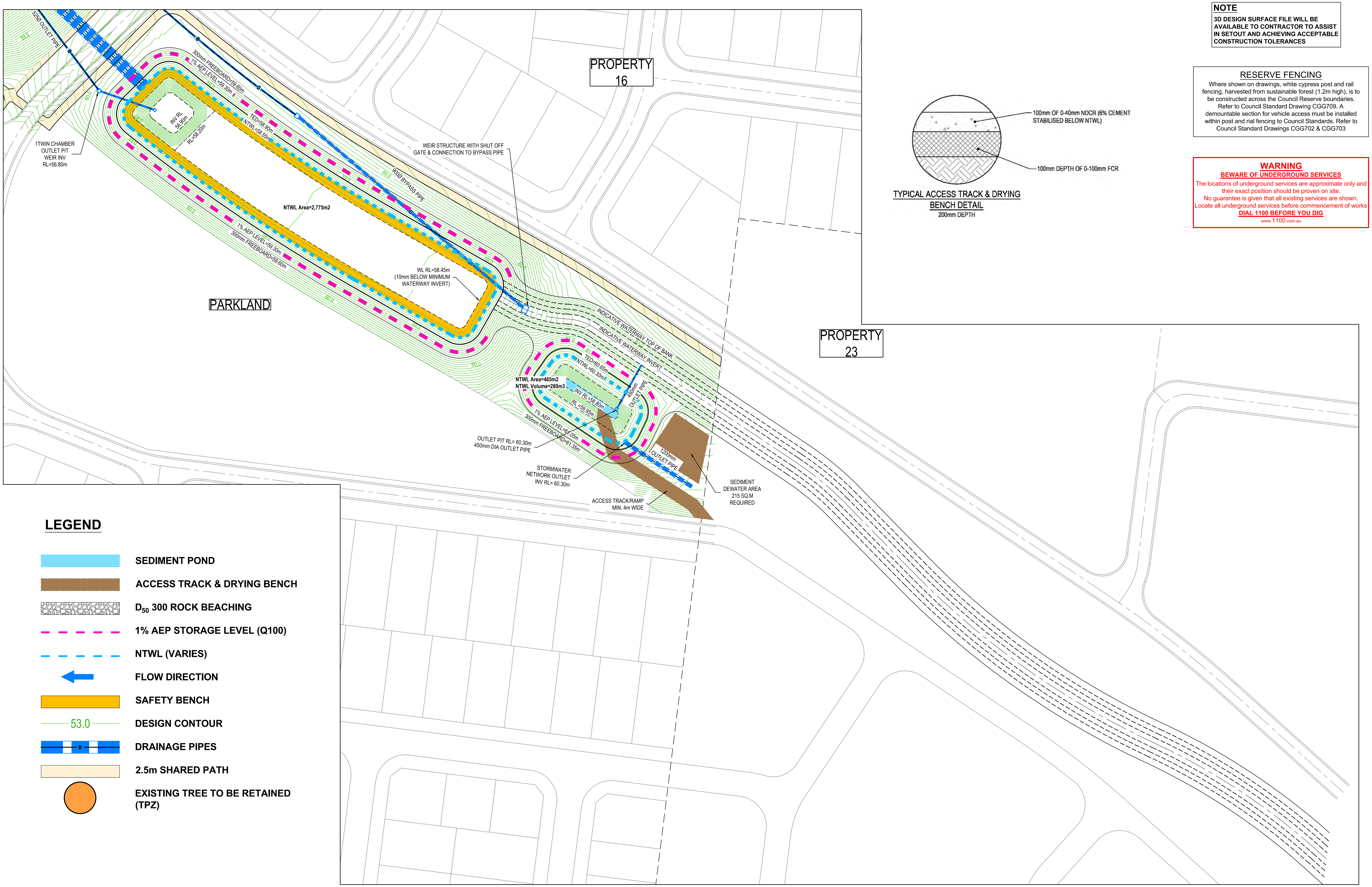

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100mm DEPTH OF 0-100mm FCR

TYPICAL ACCESS TRACK & DRYING BENCH DETAIL
200mm DEPTH

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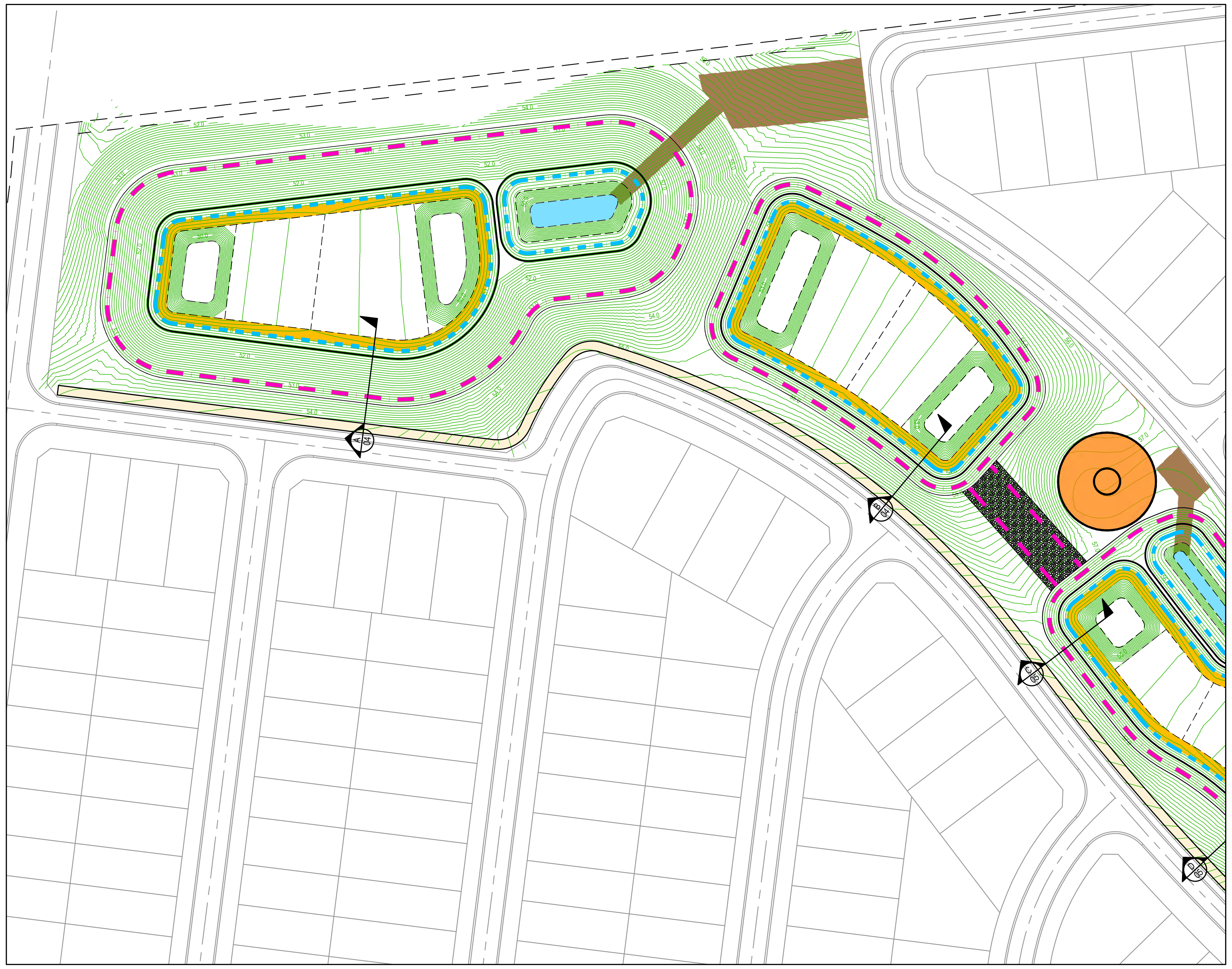
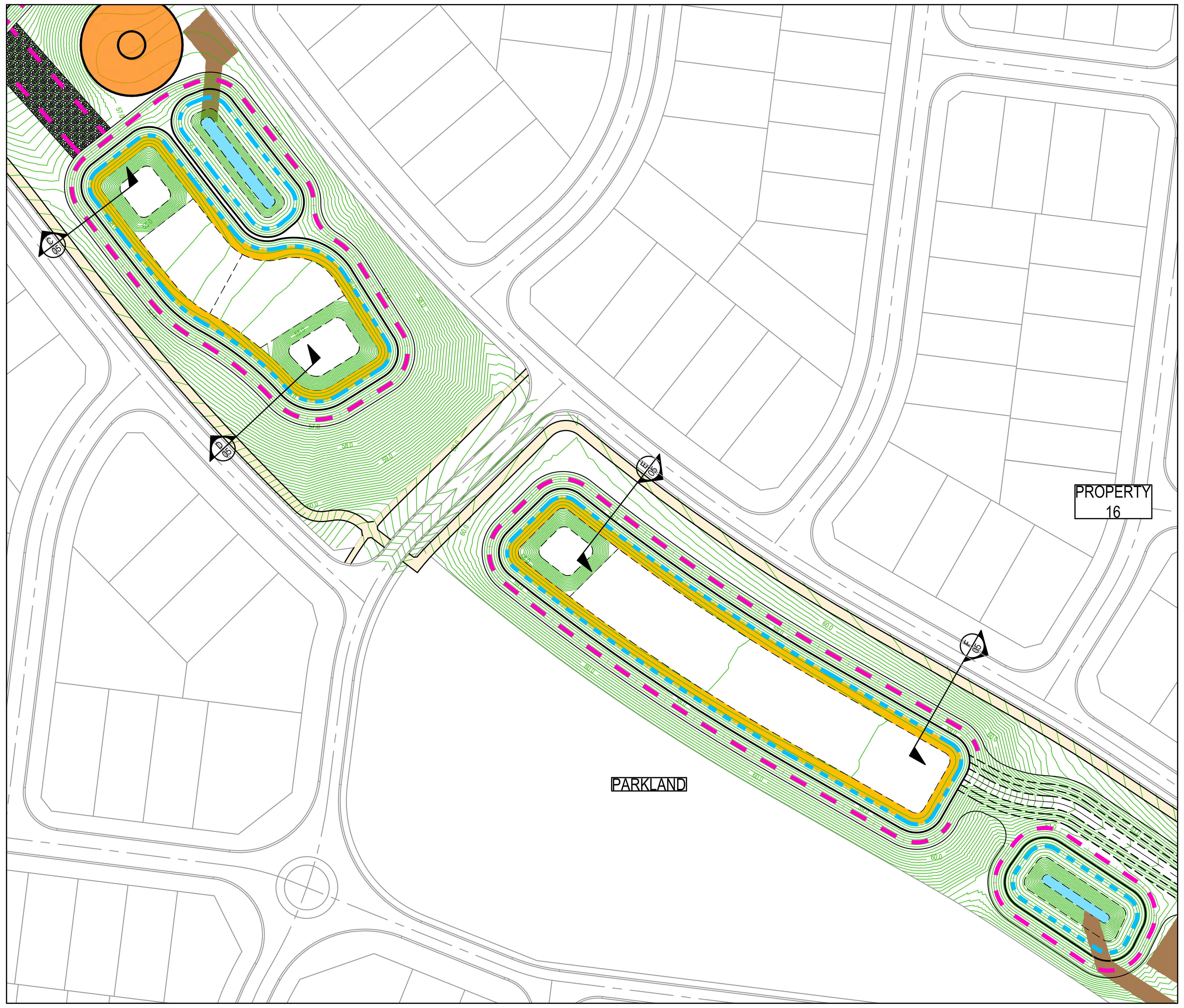
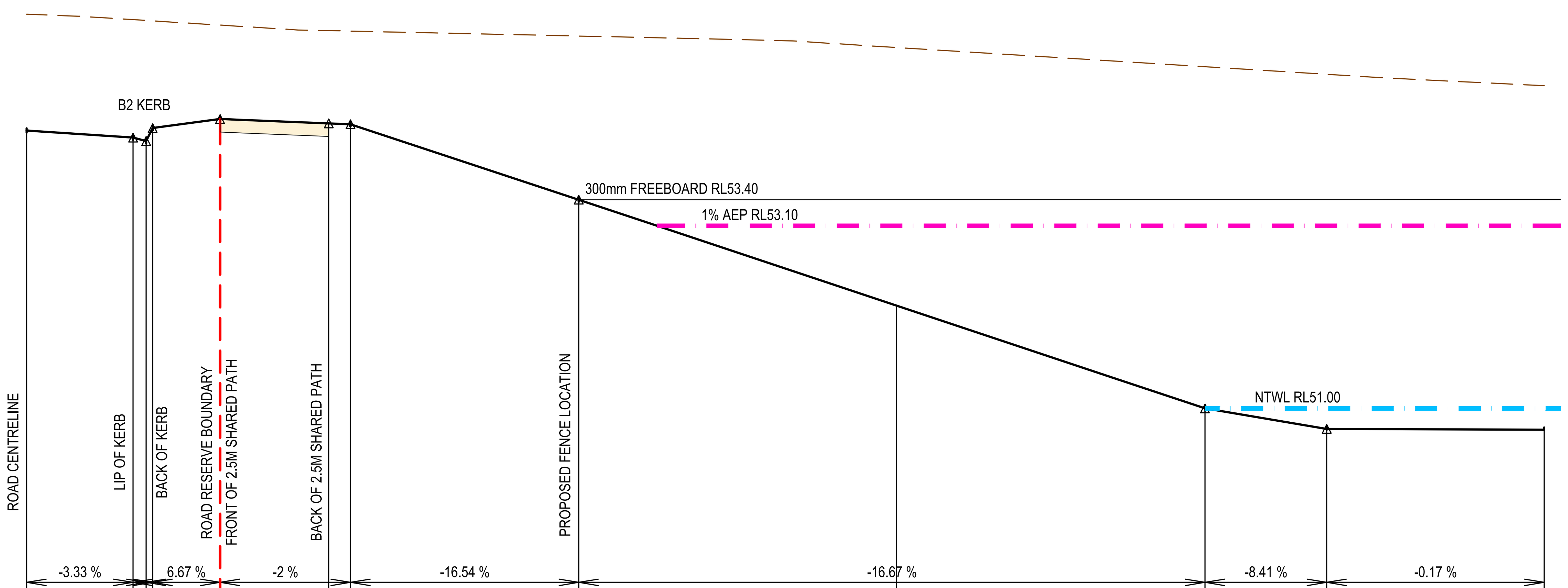






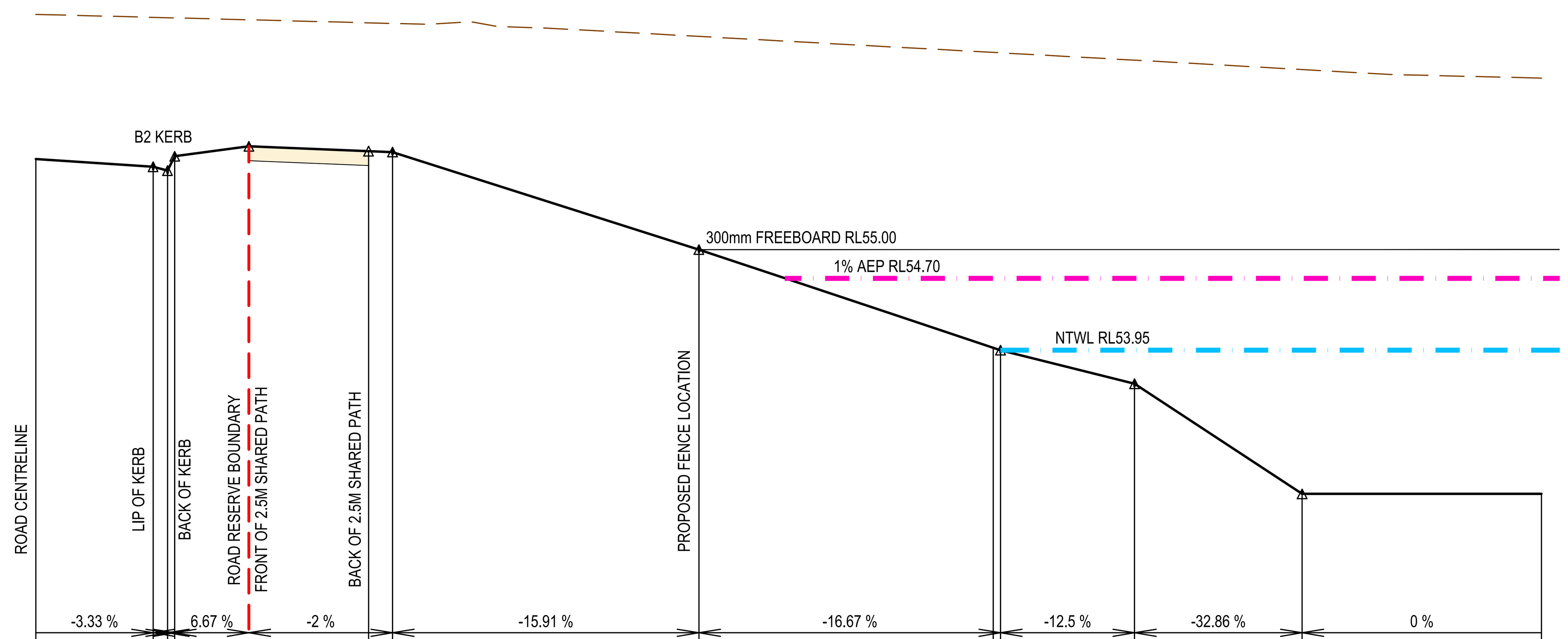
VERTICAL GEOMETRY: -3.33 % | 6.67 % | -2 % | -16.54 % | -16.67 % | -8.41 % | -0.17 %

HORIZONTAL GEOMETRY

DATUM RL48

DESIGN CENTRELINE	54.20	54.11	54.07	54.22	54.33		54.27	53.40	52.18	51.00	50.76	50.76
EXISTING SURFACE	55.53	55.47	55.46	55.46	55.41		55.34	55.28	55.15	54.93	54.84	54.71
CHAINAGE	0.00	2.45	2.75	2.90	4.45	6.95	7.45	12.70	20.00	27.10	29.90	34.90

A LONGITUDINAL SECTION



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HORIZONTAL GEOMETRY

DATUM RL50

DESIGN CENTRELINE	55.95	55.86	55.82	55.97	56.08		56.02	55.00	53.97	53.95	53.60	52.45	52.45
EXISTING SURFACE		57.42	57.42	57.42	57.40		57.36	57.23	57.06	57.05	56.98	56.88	56.79
CHAINAGE	0.00	2.45	2.75	2.90	4.45	6.95	7.45	13.85	20.00	20.15	22.95	26.45	31.45

B LONGITUDINAL SECTION

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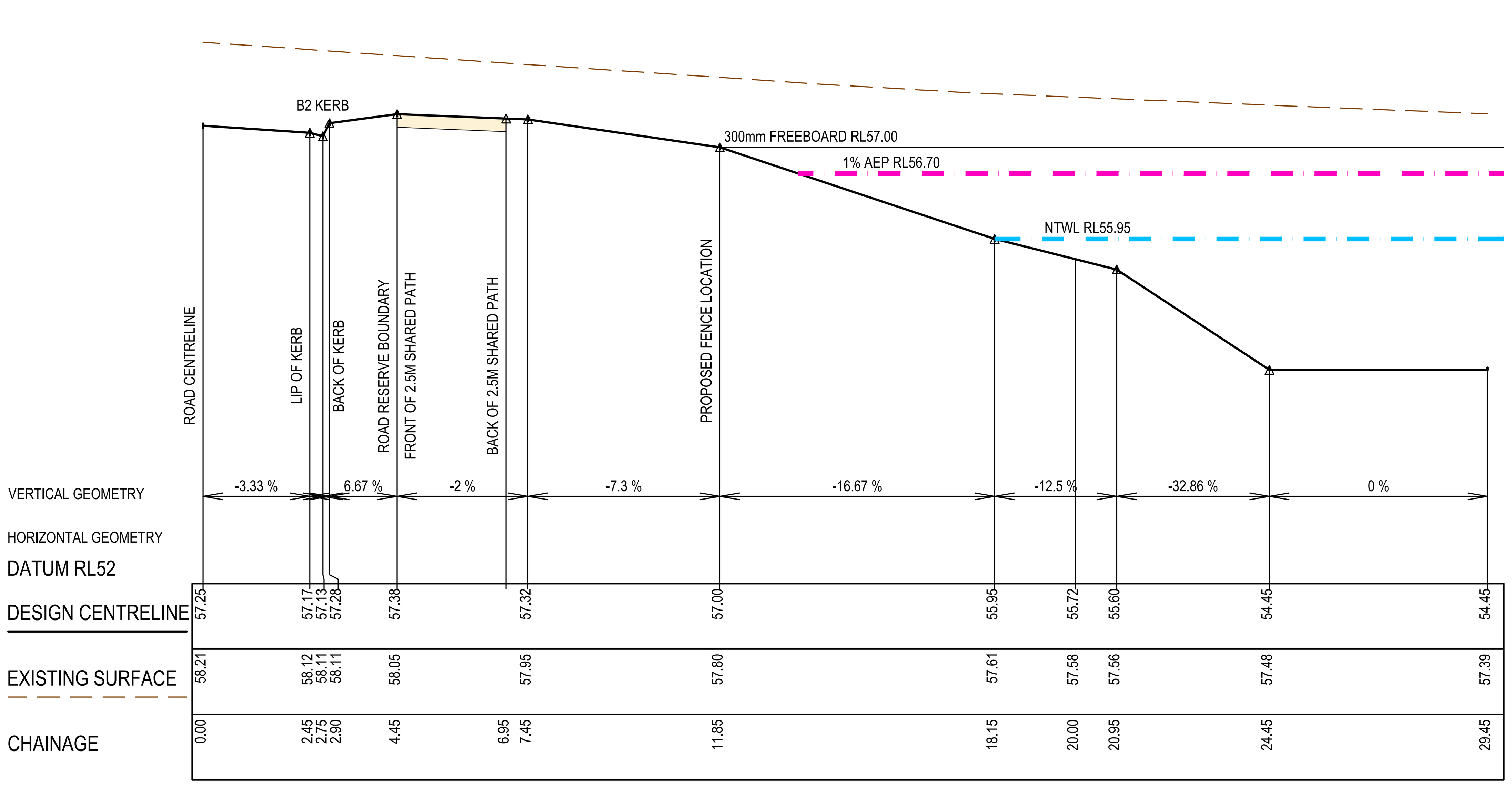

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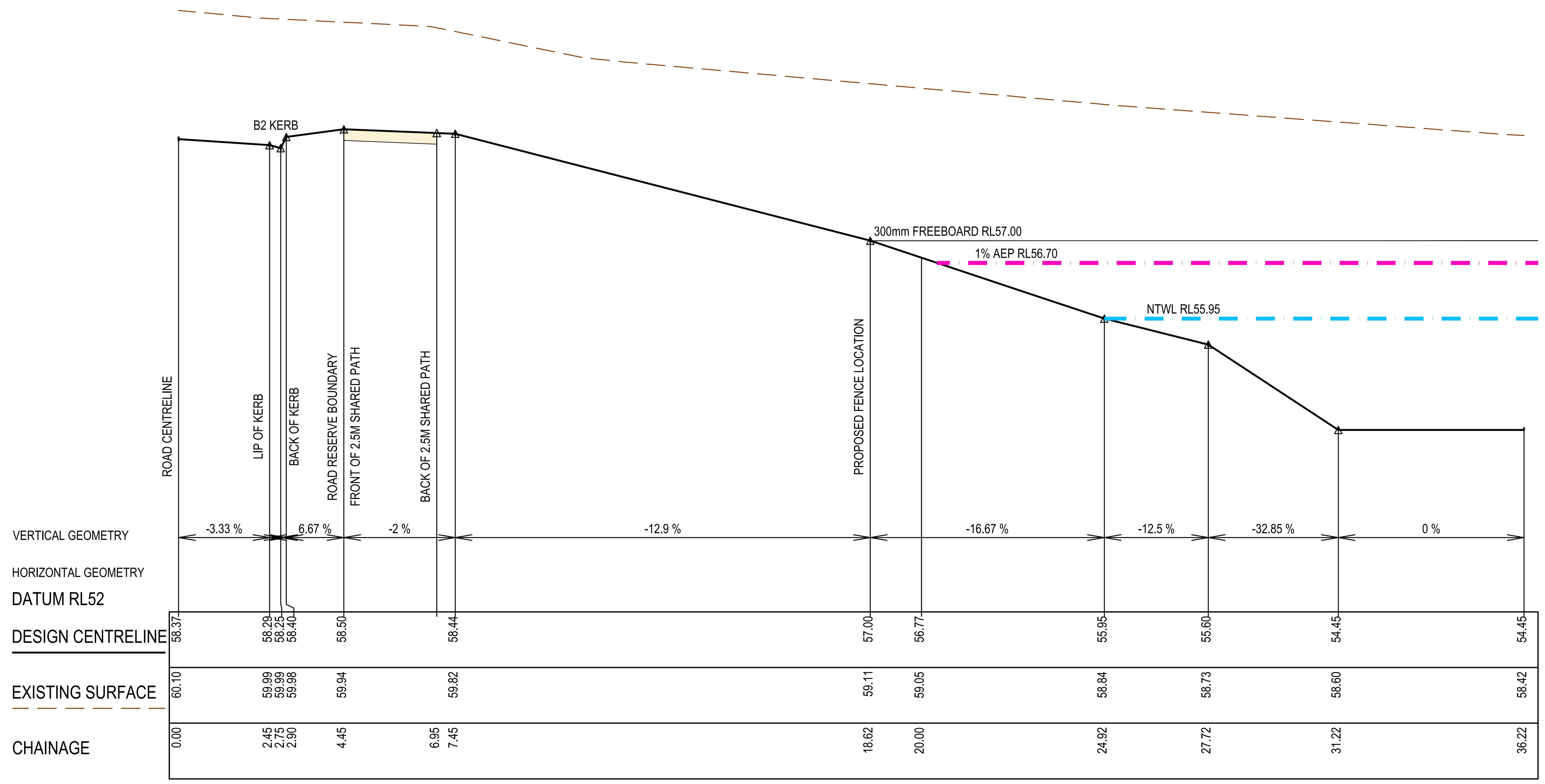

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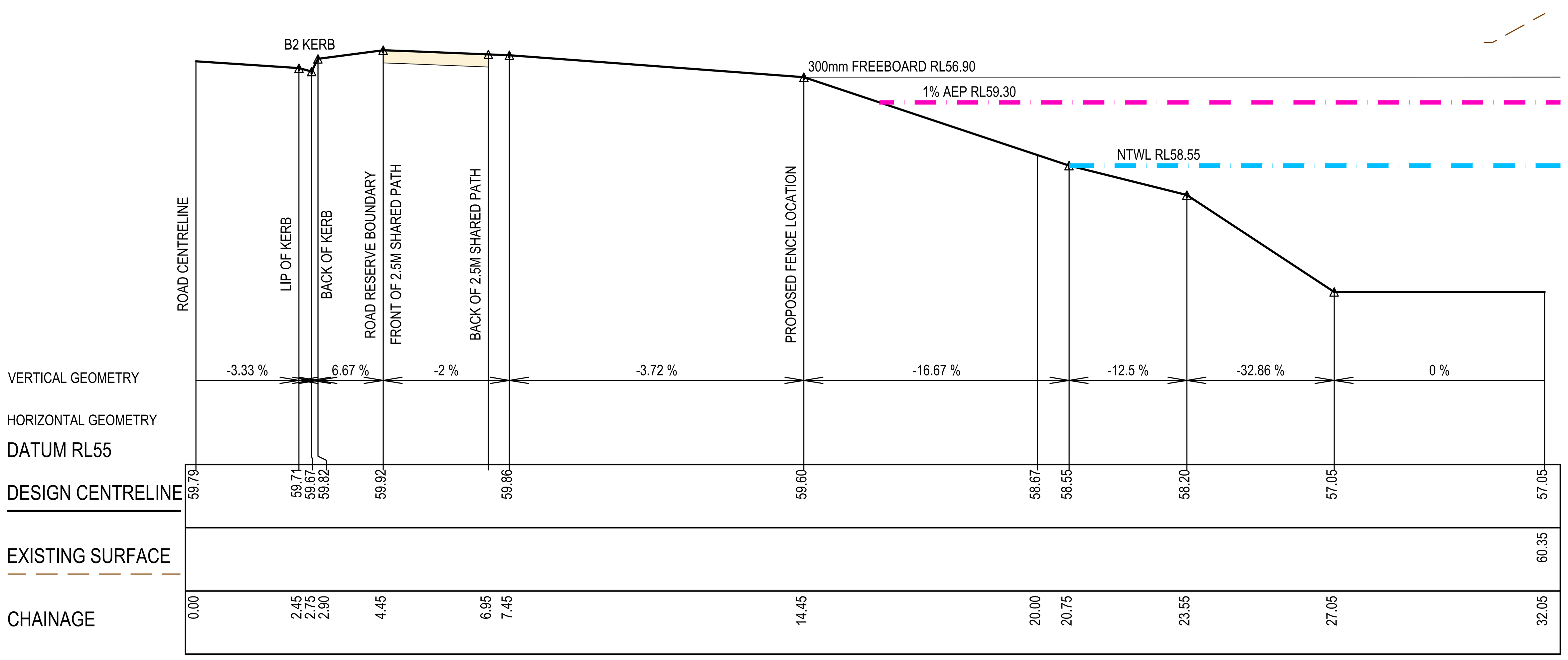

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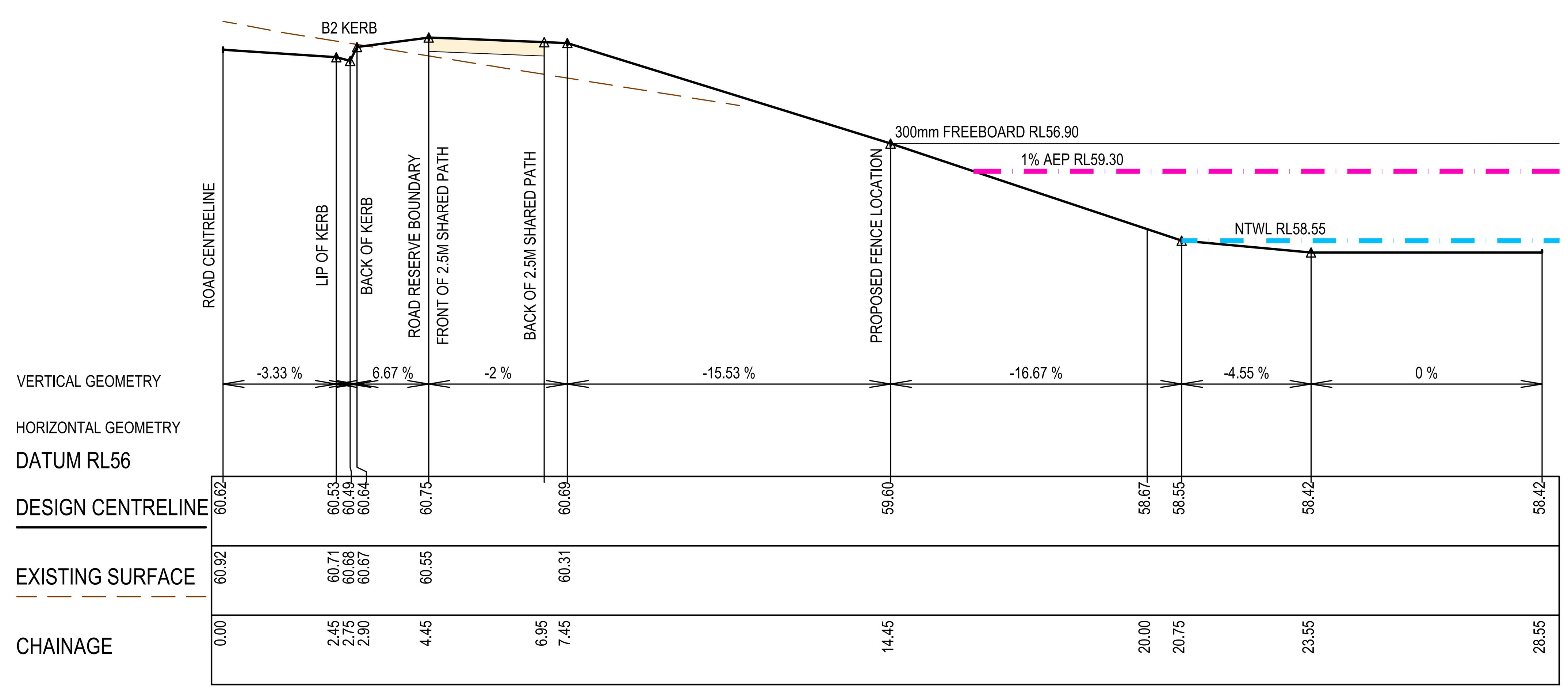

F LONGITUDINAL SECTION

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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

APPENDIX E
WATER TECHNOLOGY WETLAND ASSESEMENT MEMORANDUM

The previous Water Technology Memorandum[19] documented the additional analysis requested by the City of Greater Geelong. This memorandum included the following elements:

- Inundation frequency analysis
- Residency time analysis
- Controlling outlet structures
- H-S-Q relationships
- Sedimentation basin sizing
- Velocity computations
- MUSIC Auditor analysis
- Wetland Analysis Results

[19] Water Technology - 22010206_M01v04 – 19 July 2023

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

MEMORANDUM

To	Elizabeth Guyler (APD Projects)
From	Cameron Walsh
Date	19 July 2023
Subject	Jetty Road South of Rail Trail SWMS – Detention and Treatment Modelling Update
Our ref	22010206_M01v04

1 INTRODUCTION

Water Technology previously prepared a Stormwater Management Strategy (SWMS)[1] for the proposed development in land encompassing a number of parcels south of Bellarine Rail Trail as part of Jetty Road Stage 2. The SWMS documented a concept design to manage stormwater runoff from the proposed development.

Following issue of this report and associated conceptual asset design, SMEC has further developed multiple iterations of the conceptual design plans[2] [3] in line with discussions with the City of Greater Geelong.

The amended SMEC conceptual design plans (Rev F - dated 22/6/2023) have altered the detention and treatment asset arrangement, therefore the modelling which underpins the conceptual asset design for the retarding basin and treatment assets required updating. This Memorandum summaries the updated RORB and MUSIC modelling underpinning the current asset arrangement.

This memorandum documents:

- Amended RORB analysis for the stormwater assets:
  - Updated detention volumes and controlling structures to represent the current conceptual design revision. (Refer to Appendix for conceptual plans)
  - Removal of the proposed piped bypass flows around the linear wetland cells.
- Amended MUSIC analysis for the stormwater assets:
  - Custom H-S-Q relationships adopted for the wetland nodes

[1] Jetty Road South of Rail Trail SWMS – 22010206_R01v05 – December 2022
[2] 1421 Portarlington Road City of Greater Geelong Drainage Reserve Concept – 2360E_CL-001, 2360E_CL-002, 2360E_CL-003 – Rev E – 8/2/2023
[3] 1421 Portarlington Road City of Greater Geelong Drainage Reserve Concept – 3260E_CL-001 to 2360E_CL-005,– Rev F – 22/6/2023

2 AMENDED CATCHMENT STRATERGY

The amended asset strategy was developed based on the SMEC catchment strategy as per Figure 1.

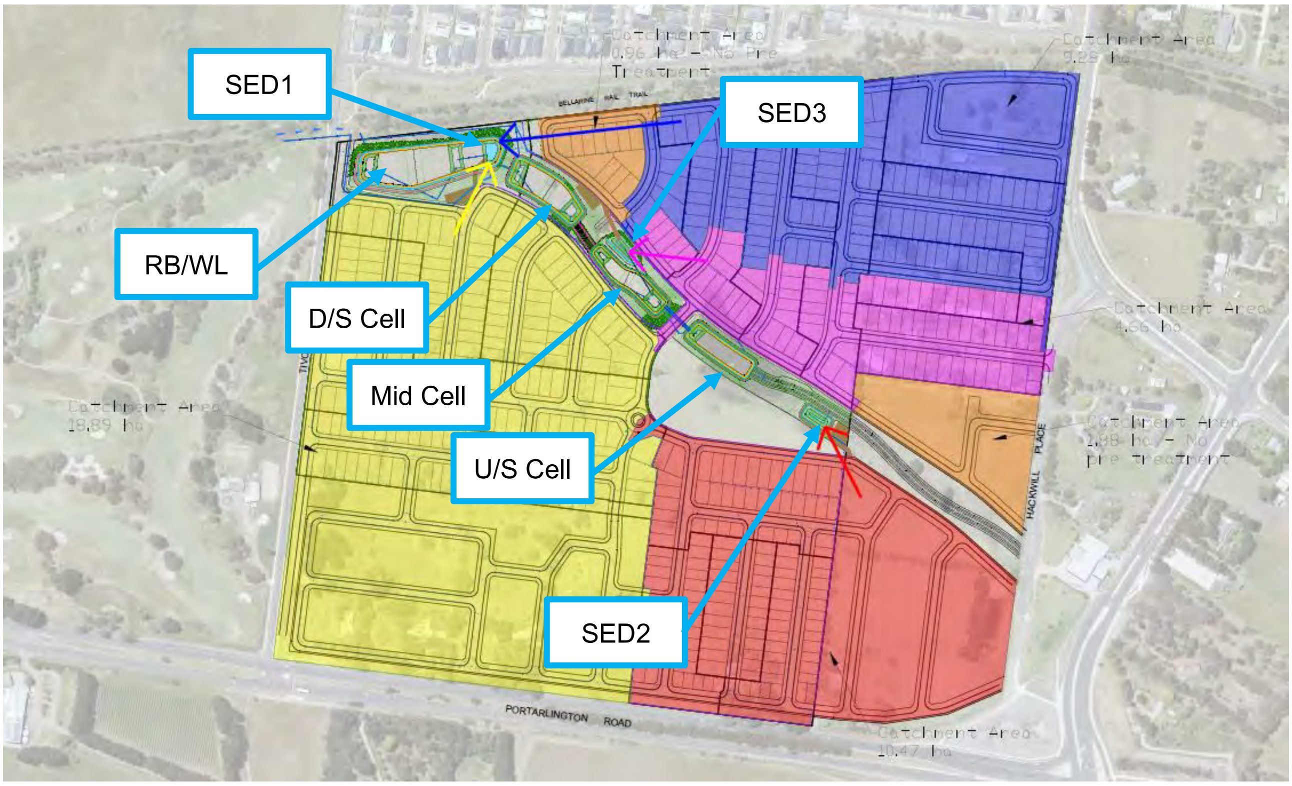


Figure 1 SMEC Overall Drainage Catchment Extract

2.1 Asset Arrangement

The proposed stormwater management strategy for the site consists of:

- A retarding basin and wetland located at the northwest corner of the site:
  - Inclusive of a sedimentation basin
- A three-cell linear wetland located along the waterway alignment:
  - Inclusive of detention volume provided above NWL within the wetland cells.
  - Inclusive of sedimentation basin associated with the Mid Cell
- A sedimentation basin for the southeast development catchment

Figure 2**Error! Reference source not found.** presents a layout of the proposed amended water quality treatment strategy.

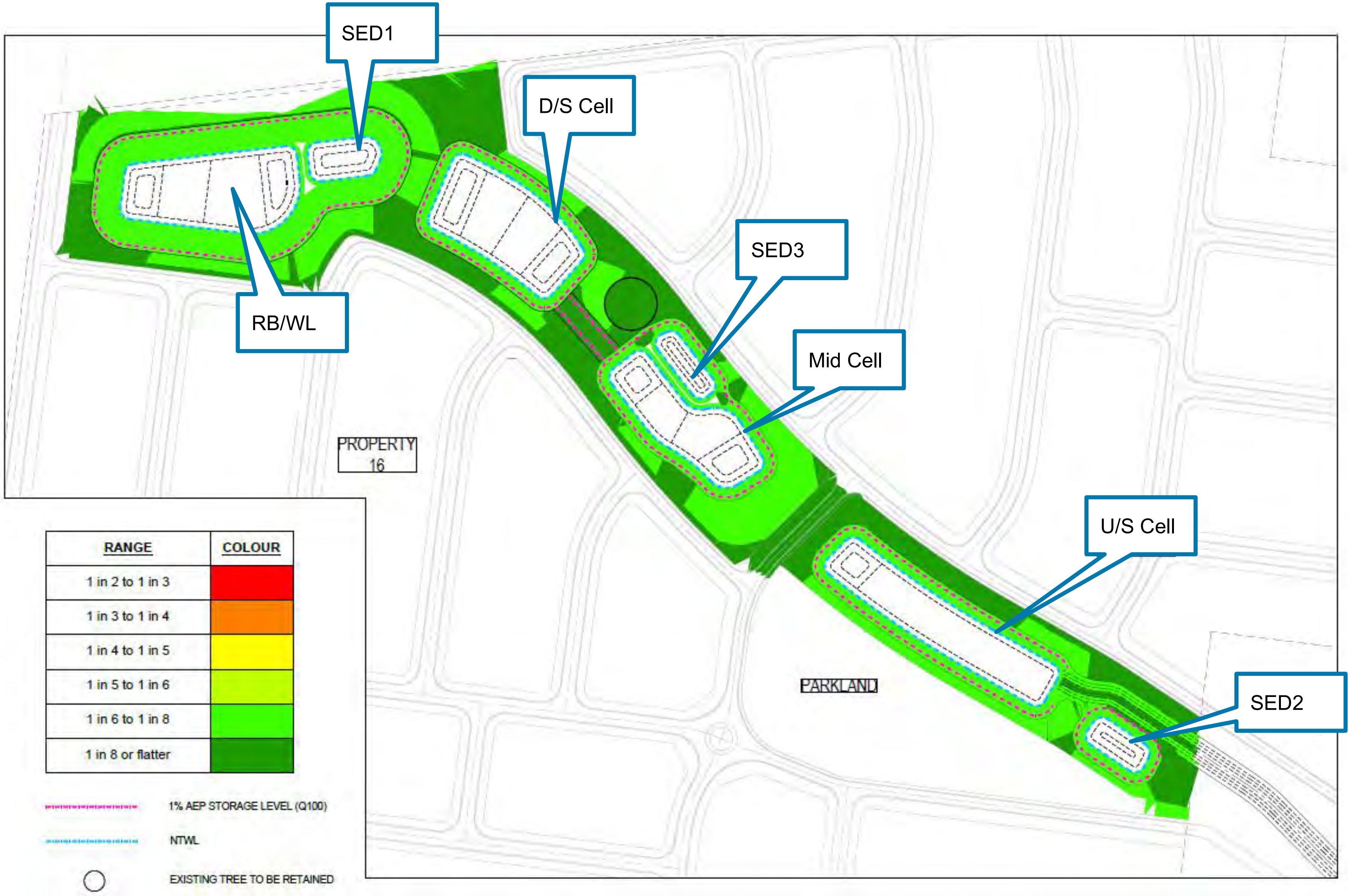


Figure 2 Updated Stormwater Management Concept Arrangement Schematic (Extract of SMEC Batter Slope Plan)

2.2 Controlling Outlet Pits

The conceptual design of the wetland cell outlets has been undertaken generally in accordance with the SMEC conceptual design. Refer to Appendix for arrangement computations.

3 UPDATED HYDROLOGICAL MODELLING

To support the amended asset arrangement, the hydrology assessment underpinning the SWMS has been updated based on the proposed catchment strategy and the inclusion of detention function into each of the three cells within the linear waterway. The 1%, and 10% AEP storm events were adopted to design the multi-storm outlet of each cell, and the ultimate site discharge location.

3.1 Developed Conditions

The RORB model underpinning the SWMS has been amended in accordance with the updated development catchment delineation, during this process the previous RORB model parameters have been updated:

- RORB, reach arrangement, and detention assets were updated accordingly within the model to represent the current overall drainage strategy.
- Inline with all previous asset arrangements, a minor diversion was included in the RORB model where the minor treatment flow of the linear wetland asset bypassed the final RB/WL asset.
- RORB parameters, FI, m, IL, and CL were adopted in line with the previous RORB modelling.

3.1.1 Retarding Basins

Four retarding basins are proposed within the development to retard development peak flows leaving the site back to existing conditions. Consisting of one RB/WL located at the northwest corner of the site, and three more detention volumes provided within each of the linear wetland cells.

The configurations of the proposed retarding basins to limit flows to target existing pre-development flow rates at the development site outlet are shown in Table 3-1 below.

Table 3-1 Retarding Basin Details

Retarding Basin Details	RB/WL	D/S Cell	Mid Cell	U/S Cell
Storage Volume Available (m^3) (NWL to Q100)	11,966	2,489	1,873	2,581

The proposed retardation asset(s) function is summaries below:

- Minor and major flows will enter the linear wetland retarding basin cells as piped flows and overland flows respectively from the contributing development catchment and external catchments upstream of the retarding basins.
- Minor and major flows will enter the final RB/WL asset as piped flows and overland flows respectively from the contributing development catchment and linear wetland system upstream of the retarding basins.
  - The minor treated flow from the U/S linear wetland system will be bypassed around the final RB/WL system.

- The retarding basin has been designed to have ensure it can cater for up to and including the 1% AEP flows.

Refer to Appendices for H-S-Q relationship, Controlling outlet pit computations, and SMEC conceptual design plans.

The overall detention performance is summarised in Table 3-2.

Table 3-2 Overall Retention performance (flow location adopted at final RB/WL in the overall series of detention assets).

Item	Details
Upstream Area (ha)	134*
Storage Volume (m^3)	Refer Table 3-1
Pre-development 1% AEP rate at RB/WL (m^3/s)	1.88*
Peak 1% AEP RB/WL Inflow (m^3/s)	4.14
Peak 1% AEP RB/WL Outflow (m^3/s)	1.771
Peak 1% AEP Site Outflow (m^3/s)	1.777
Peak 1% AEP Water Depth in RB/WL (m)	2.03
Pre-development 10% rate at RB (m^3/s)	0.83*
Peak 10% AEP RB/WL Inflow (m^3/s)	1.066
Peak 10% AEP RB/WL Outflow (m^3/s)	0.822
Peak 10% AEP Site Outflow (m^3/s)	0.828
Peak 10% AEP Water Depth in RB/WL (m)	1.26

*Adopted as per previous SWMS modelling

4 UPDATED WATER QUALITY MODELLING

In line with the updated development catchment delineation, the MUSIC model underpinning the SWMS has been amended. During this process the previous MUSIC model parameters have been updated:

- H-S-Q relationships adopted for the wetland nodes

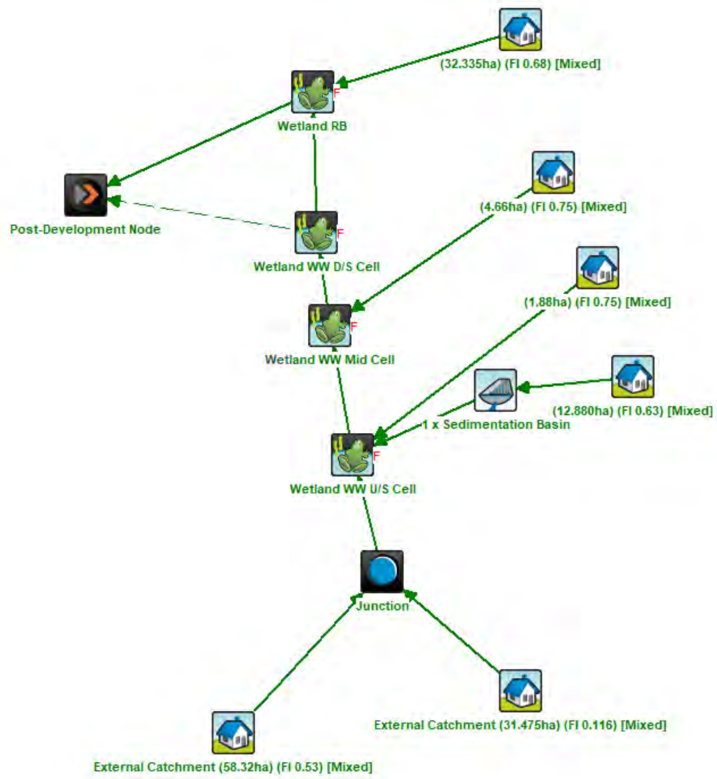


Figure 3 MUSIC Model Schematic

4.1 Design Criteria

Sediment basins were sized to allow:

- 95% removal of coarse particles ≥ 125 µm diameter; and
- 5-year clean-out frequency (minimum).

The constructed wetland was sized to allow:

- An extended Detention Depth of 350 mm;
- Approximately 48 -72 hr detention time;
  - For both the final RB/WL asset and the combined linear wetland system
  - Residence time(s) are discussed in subsequent sections of this Memorandum.

4.2 Asset Sizing

The key modelling parameters for the end-of-line treatment assets are shown in Table 4-1.

Table 4-1 Key Modelling Parameters Related to Sedimentation Ponds and Wetland

Catchment	System	Surface Area at the NWL (m^2)	Extended Detention Depth (m)	Low Flow Bypass (m^3/s)	High Flow Bypass (m^3/s)	Permanent Pool Volume (PPV) (m^3)	Notional Detention Time (hrs)
1	RL/WL	2,915	0.35	0.0*	n/a	1,040	Na**
2a	D/S Cell	2,815	0.35	0.0	n/a	1,085	Na**
2a	Mid Cell	2,090	0.35	0.0	n/a	1,005	Na**
2b	U/S Cell	2,775	0.35	0.0	n/a	680	Na**
2b	Sed Basin (Sed2)	420	0.35	0.0	0.13	205	~12hrs

*The Linear Wetland Cell bypasses the treated ‘pipe’ flows around the final RB/WL node

**Refer to residence time discussed subsequently in this Memorandum

4.3 Water Quality Benefits

Pollutant load reduction performance are summarised in Table 4-2 and Table 4-3. Table 4 presents the results of a scenario where it is assumed there will be no WSUD treatment upstream of the site to cater for any proposed development within the external catchment. However, it is likely any future development will adopt its own strategy to treat stormwater to meet the Best Practice Environment Management (BPEM) targets at minimum. Therefore, Table 4-3 represent the pollutant load reduction performance assuming the external catchment zoned for development is treated to BPEM.

In both scenarios, it is evident that the proposed WSUD strategy exceeds the BPEM target for the overall development. It is worth noting that wetland hydraulic performance also important for overall wetland function. Therefore, achieving pollutant load reduction targets as well as hydraulic performance targets (water levels and hydraulic residence time) are important. Refer to Section 4.4 for further detail

Table 4-2 Pollutant Load Reduction Performance for the Proposed Site (Excluding External Catchment Treatment)

Component	Total Inflow Loads (kg/yr)	Inflow Loads Produced by the Development (kg/yr)	Load Removed (kg/yr)	Percentage Removal of Development Load
Total Suspended Solids	62,300	31,500	49,200	156%
Total Phosphorus	130	64.1	87	135%
Total Nitrogen	937	453	372	82%

Table 4-3 Music Modelling Results for the Proposed Site (Assuming BPEM External Catchment Treatment)

Component	Total Inflow Loads (kg/yr)	Inflow Loads Produced by the Development (kg/yr)	Total Load Removed (kg/yr)	External BPEM Asset Load Removed (kg/yr)	Load Removed attributed to development (kg/yr)	Percentage Removal of Development Load
Total Suspended Solids	62,800	31,500	53,230	21,630	31,600	100%
Total Phosphorus	130	64.1	95	38.5	56	87%
Total Nitrogen	938	453	453	197	256	57%

4.4 Inundation Frequency and Residence Time

The City of Greater Geelong specify the following requirements to reduce the risk of plants drowning due to excessive depth, frequency and duration of inundation:

- The effective water depth (permanent pool depth plus depth above NWL) must not exceed half the average plant height for more than 20% of the time, see Figure 5.
- The average water level (exceeded 50 percent of the time) should not be more than 50 millimetres above normal water level.
- No more than one spell of 10 days or more occurs where the water level is greater than or equal to 300 millimetres above NWL (in a reference period of 10 years).

Additionally, the macrophyte zone must provide a 90th percentile residence time of at least 72 hours (no more than 80 hours) to ensure there's sufficient time to treat stormwater as it passes through the wetland.

Typically achieving pollutant load reduction targets, inundation criteria and residence time criteria is feasible if the wetland is properly sized relative to its contributing catchment. In situation, where an additional external catchment is passing through the wetland system, it is difficult to achieve all three design criteria simultaneously. The vegetation health is critical to ensure wetland pollutant removal performance, therefore, in the proposed arrangement the order of priority was set for achieving BPEM targets first and then the criteria for water level and residence time respectively.

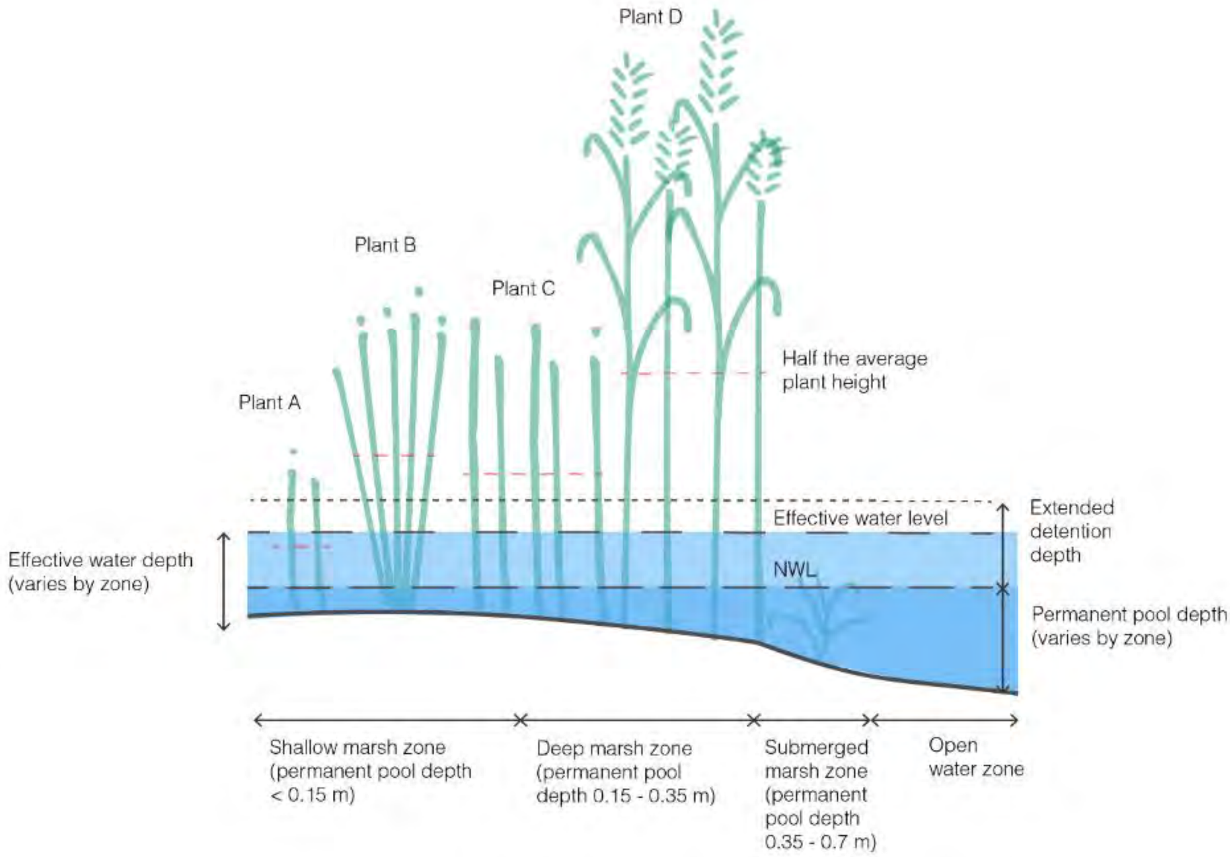


Figure 4 Comparing plant heights and effective water depth (Source: CoGG MUSIC modelling guidelines)

The wetland inundation and residence time analysis has been undertaken generally in accordance with the conceptual design undertaken by SMEC.

Table 4-4, Figure 5 to Figure 8 outlines the residence time analysis for the wetland asset cells. When reviewing the results on residence times presented in Table 6, Council should consider the results for “RB/WL” and “Waterway cells combined” (for linear wetland system). While the recommended 90th percentile retention time of 72 hours has not been achieved for the wetland system, it achieved the criteria for wetland water level.

For both the final RB/WL and combined linear wetland, the 72 and 48 hours were achieved for approximately the 80% and 90% percentile respectively. Considering the nature of the catchment and asset arrangement the achieved residence time is considered satisfactory.

Refer to the appendix for a summary of the Wetland Analysis Tool output(s).

Table 4-4 Summary of Residence Time

Criteria	RB/WL	DS Waterway Cell	Mid Waterway Cell	US Waterway Cell	Waterway Cells (Combined)***
72 hr residence (percentage exceedance)	78% **	50%	49%	55%	79% **
48 hr residence (percentage exceedance)	87% **	64%	61%	67%	91% **
24 hr residence (percentage exceedance)	96%	85% *	81% *	90% *	98%
20% level	0.152	0.167	0.166	0.171	na
50% level	0.047	0.049	0.045	0.043	na
Spells Analysis >= 0.3m (Number of times exceeded 10 day duration period)	0	0	0	0	na

* Targeting 24 Hours (at 90%)

** Targeting 48-72 hours (at 90%)

***The Linear wetland cells was also processed as a combined asset node consisting of the total treatment areas and volume of all cells,

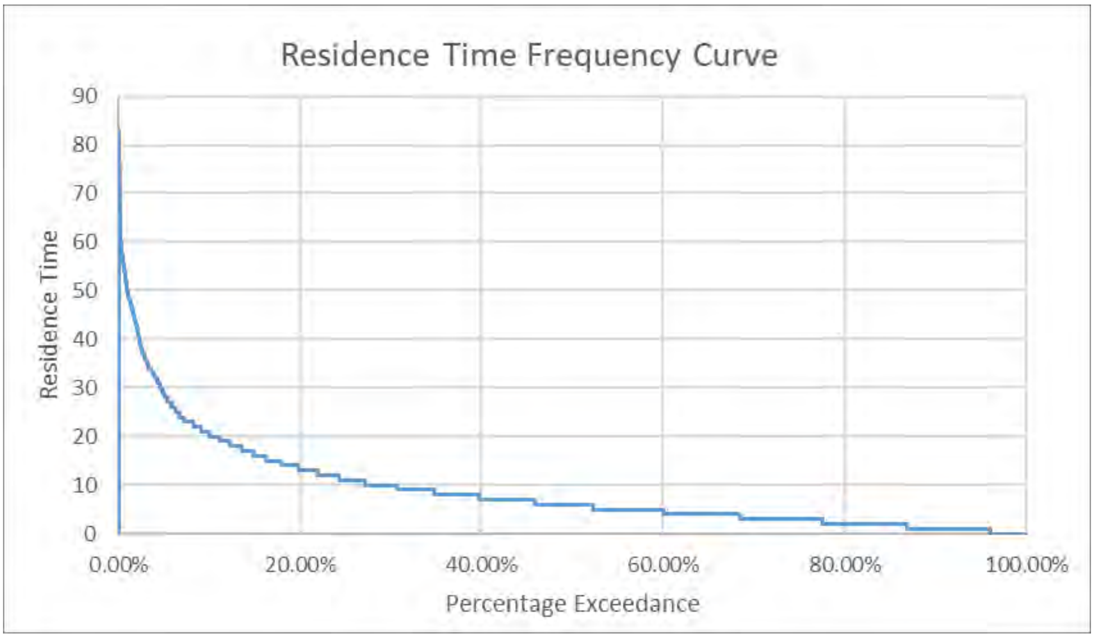


Figure 5 Residence Time Frequency Curve RB/WL

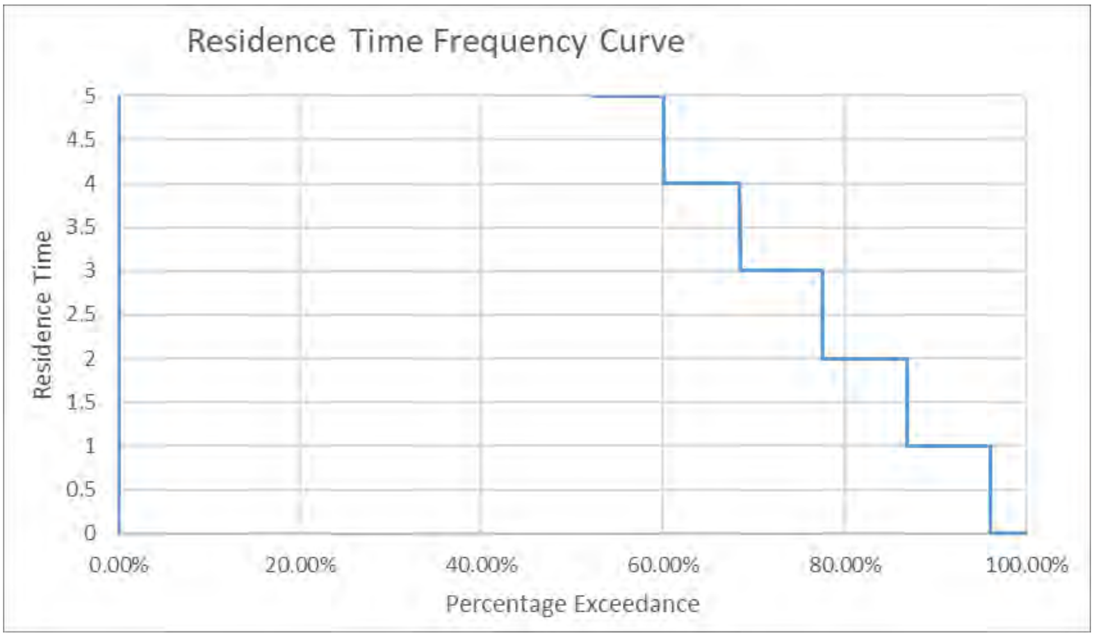


Figure 6 Residence Time Frequency Curve RB/WL – Reduced Y axis

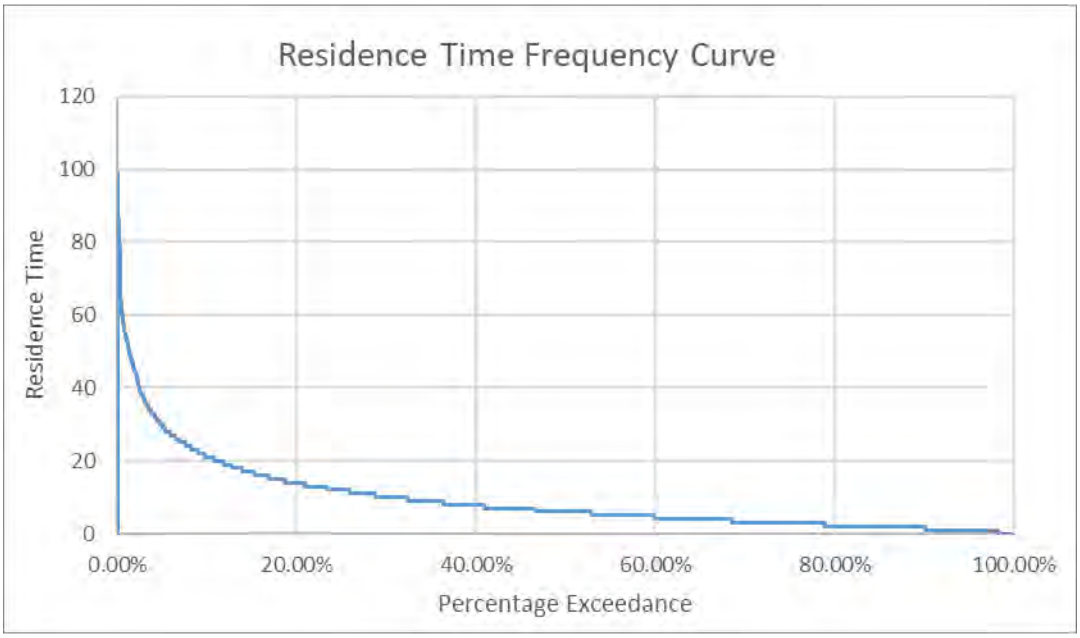


Figure 7 Residence Time Frequency Curve Combined Linear Cells

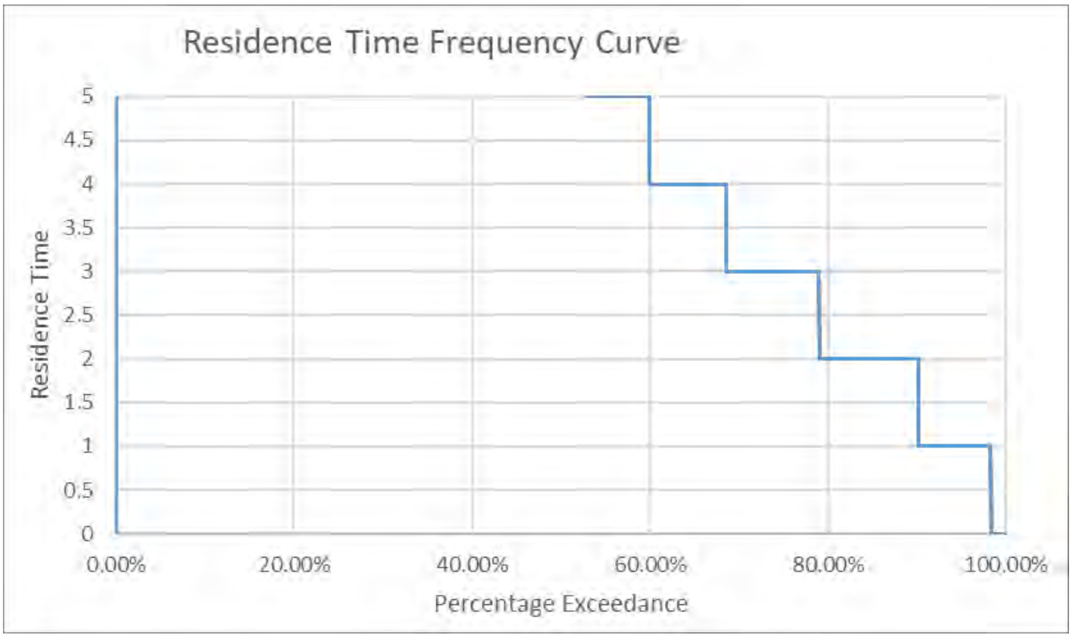


Figure 8 Residence Time Frequency Curve Combined Linear Cells – Reduced Y axis

SUMMARY

This memorandum outlines RORB and MUSIC modelling underpinning associated with the amended SMEC (Revision F) asset design in Appendix A.

The status and content of this memorandum has evolved over time as follows:

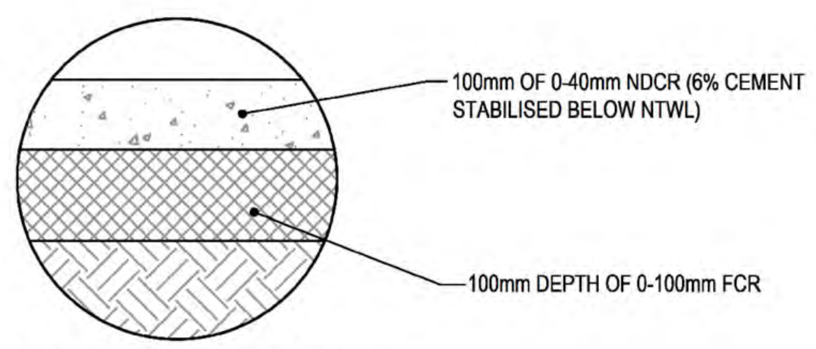
1. Memo was submitted to Council & discussed in meeting at Council Offices on 26/04/2023. Council agreed that bypassing external flows is the preferred option to improve wetland residence times.
2. Memo resubmitted to Council on11/05/2023 including remodelling results which excluded the external flows (by bypassing them through the bypass pipes). Residence time criteria was achieved in the critical wetland cells (72hrs for 90% of time in the main RB).
3. Meeting with Council Engineers & Planners on 8/06/2023 to confirm that APD would increase land take by 1ha (15 m along waterway) to achieve 1V:6H or shallower batters. It was discussed the CoGG and CCMA preference would be to use bypass pipe only for maintenance purposes.
4. Meeting with Council planning on 23/06/2023 (engineers were invited but did not attend). Revised SMEC design which included revised batter profiles & additional 1 ha land take was presented to Council planning. Full set of plans submitted including DCP plan, X-Sections, Heat Maps, overall waterway context plans.
5. Response from Council planning on 28/06/2023 stating that a MUSIC model should be provided which demonstrates if there is a need for a low flow diversion around the wetland cells. Noting that the current bypass pipe in the design should only be used for maintenance purposes (per CMA advice). Council have also requested Longitudinal sections of the wetland.

In the current memorandum, it has been demonstrated that the SMEC Revision F asset design meets the requirements for effective water level exceedance criteria (50% level is ≤50 mm). Therefore, water levels for plant health establishment and longevity are acceptable.

However, it is noted that residence times (90% achieved for 72hrs) are not met. The 72-hour residence time is exceeded 78% of the time in the RB/WL and 79% in the Combined linear waterway system. Wetland hydraulic performance is expected to be optimised for the RB/WL through asset functional/detailed design phases.

Overall, the proposed asset arrangement meets the requirement for peak flow attenuation, exceeds pollutant load reduction performance, and the project team are satisfied that this asset (SMEC REV F design) meets the required objectives for this site. Council must consider the competing interests of residence times, effective water levels & the CCMA's input regarding use of bypass pipe (for maintenance only). The project team are open to further discussion with CoGG and CCMA during functional design to optimize the design further. This should include preparation of an operation and maintenance plan for the bypass pipe, which will assist in understanding its function and use.

APPENDIX A
SMEC DESIGN PLAN



TYPICAL ACCESS TRACK & DRYING BENCH DETAIL

200mm DEPTH

NOTE

3D DESIGN SURFACE FILE WILL BE AVAILABLE TO CONTRACTOR TO ASSIST IN SETOUT AND ACHIEVING ACCEPTABLE CONSTRUCTION TOLERANCES

RESERVE FENCING

Where shown on drawings, white cypress post and rail fencing, harvested from sustainable forest (1.2m high), is to be constructed across the Council Reserve boundaries. Refer to Council Standard Drawing CGG709. A demountable section for vehicle access must be installed within post and rial fencing to Council Standards. Refer to Council Standard Drawings CGG702 & CGG703

WARNING

BEWARE OF UNDERGROUND SERVICES

The locations of underground services are approximate only and their exact position should be proven on site. No guarantee is given that all existing services are shown. Locate all underground services before commencement of works

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C	25/01/23	WETLANDS AMENDED	B.BARBER	B.BARBER	R.FORBES	T.MOORFOOT
D	10/02/23	WETLANDS AMENDED	M.SURRAO	B.BARBER	R.FORBES	T.MOORFOOT
E	28/02/23	WETLANDS & SED BASIN AMENDED	M.SURRAO	R.FORBES	R.FORBES	T.MOORFOOT
F	22/06/23	WETLANDS & SED BASINS AMENDED	M.SURRAO	R.FORBES	R.FORBES	T.MOORFOOT
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ABN 47 065 475 149

East 5, Federal Mills - 33 Mackey Street
North Geelong, VIC 3215
Ph 03 5228 3100

Curlewis - APD
1421 Portarlington Road
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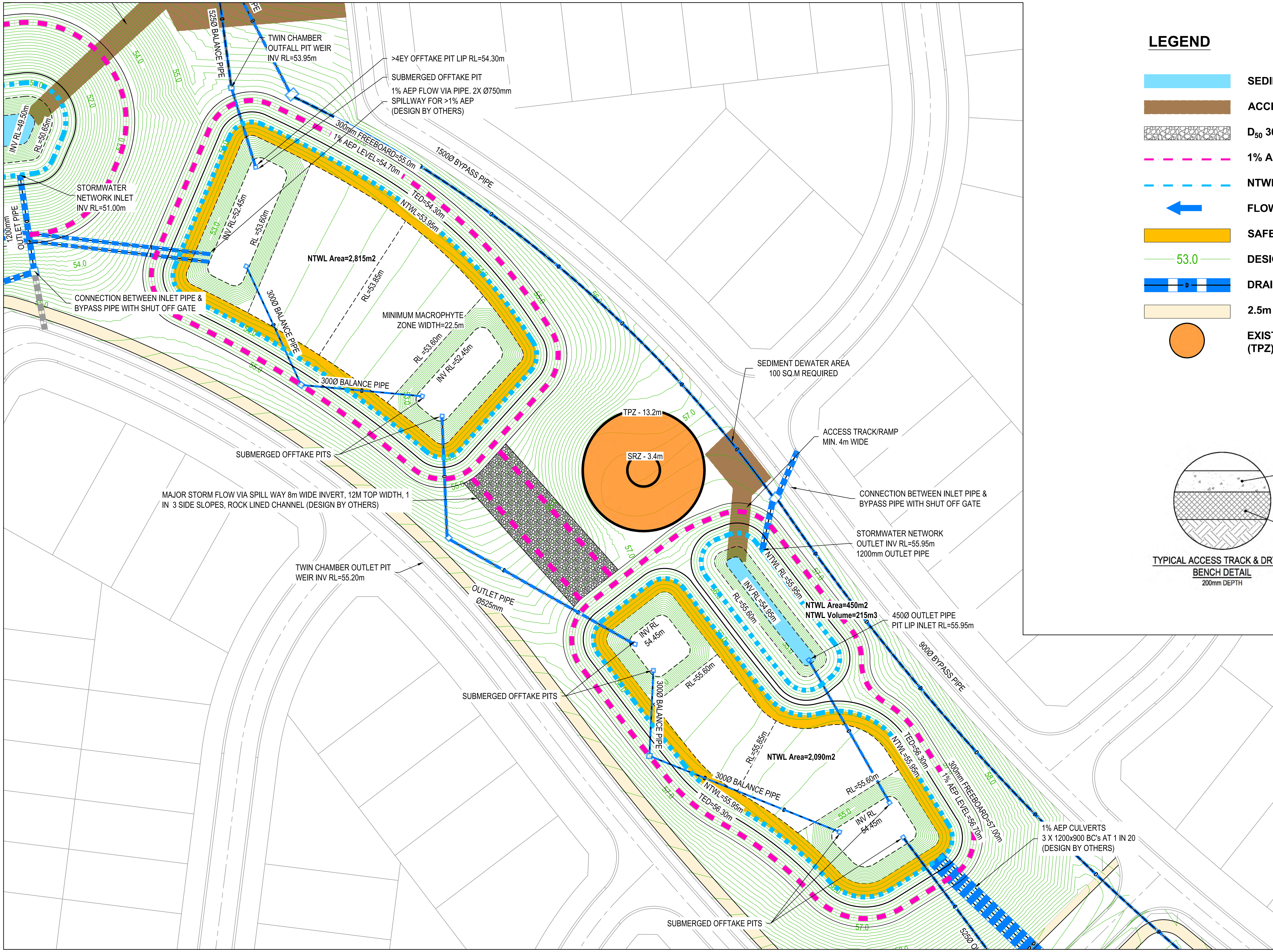

LEGEND

- SEDIMENT POND
- ACCESS TRACK & DRYING BENCH
- D_{50} 300 ROCK BEACHING
- 1% AEP STORAGE LEVEL (Q100)
- NTWL (VARIES)
- FLOW DIRECTION
- SAFETY BENCH
- 53.0 DESIGN CONTOUR
- DRAINAGE PIPES
- 2.5m SHARED PATH
- EXISTING TREE TO BE RETAINED (TPZ)

100mm OF 0-40mm NDCR (6% CEMENT STABILISED BELOW NTWL)
100mm DEPTH OF 0-100mm FCR

TYPICAL ACCESS TRACK & DRYING BENCH DETAIL
200mm DEPTH

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Member of the Surbana Jurong Group
© ABN 47 065 475 149
East 5, Federal Mills - 33 Mackey Street
North Geelong, VIC 3215
Ph 03 5228 3100

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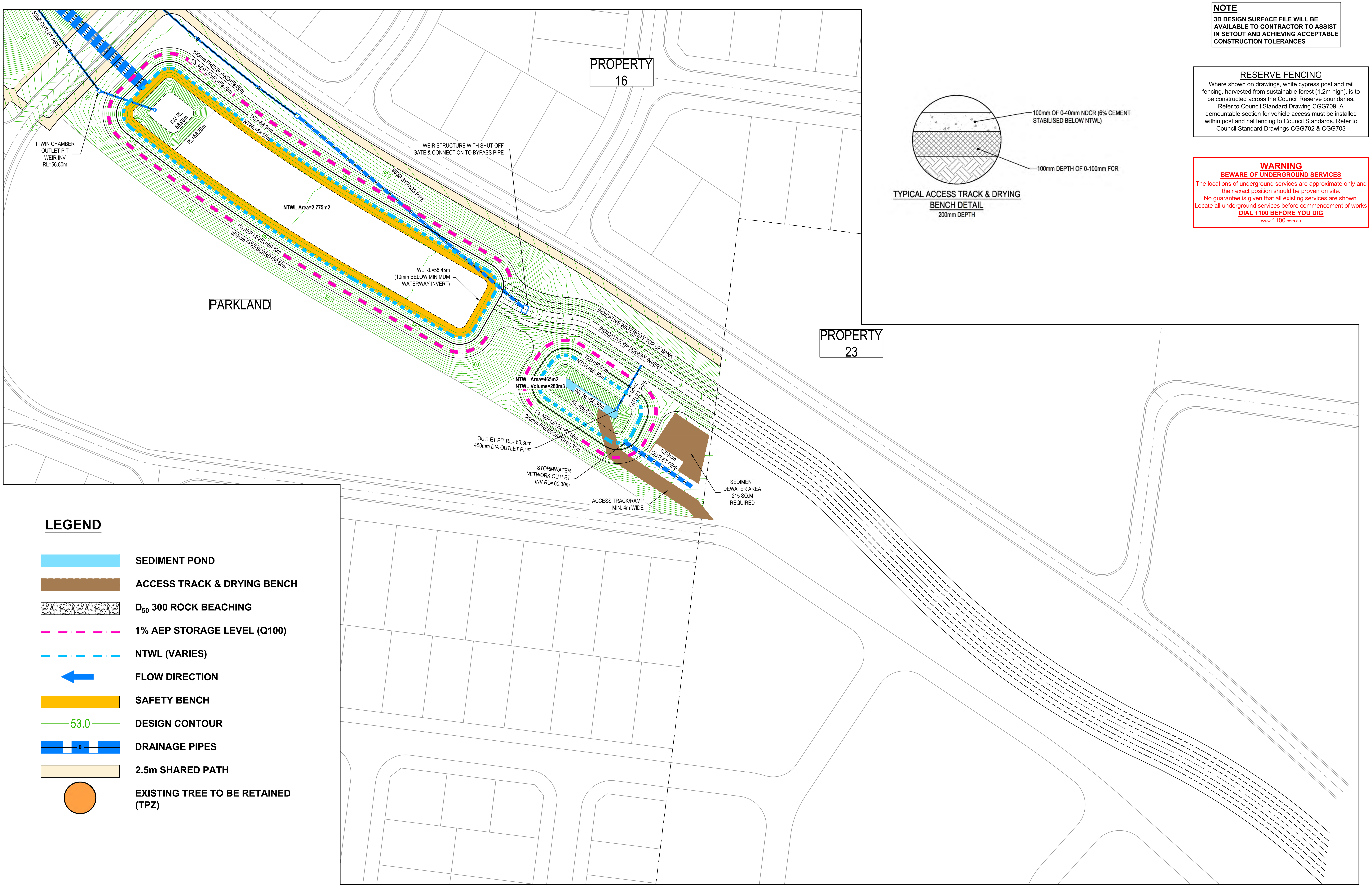

LEGEND

- SEDIMENT POND
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- NTWL (VARIES)
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100mm OF 0-40mm NDCR (6% CEMENT STABILISED BELOW NTWL)
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F	22/06/23	WETLANDS & SED BASINS AMENDED	M.SURRAO	R.FORBES	R.FORBES	T.MOORFOOT
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Member of the Surbana Jurong Group
ABN 47 065 475 149
East 5, Federal Mills - 33 Mackey Street
North Geelong, VIC 3215
Ph 03 5228 3100

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1421 Portarlington Road
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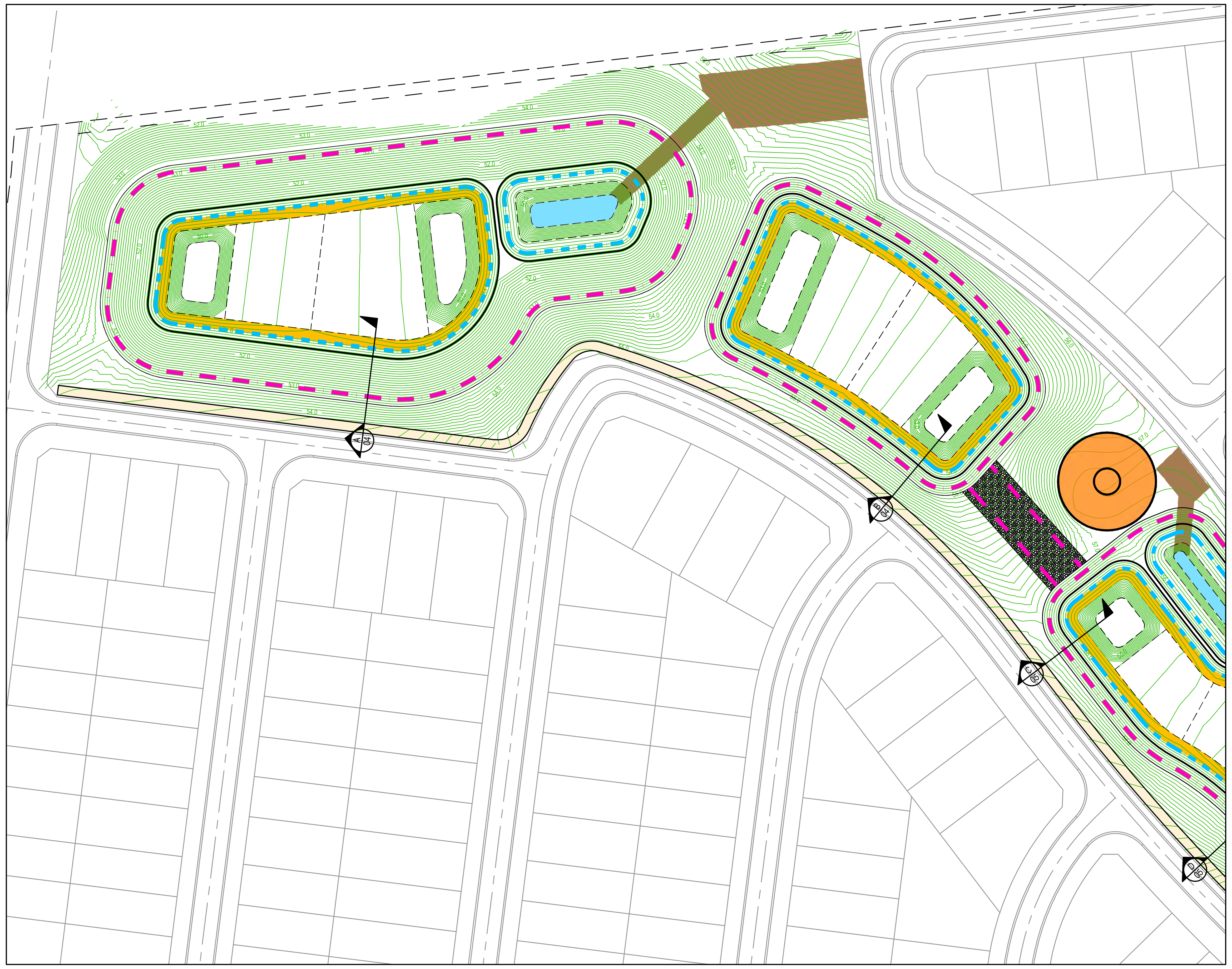
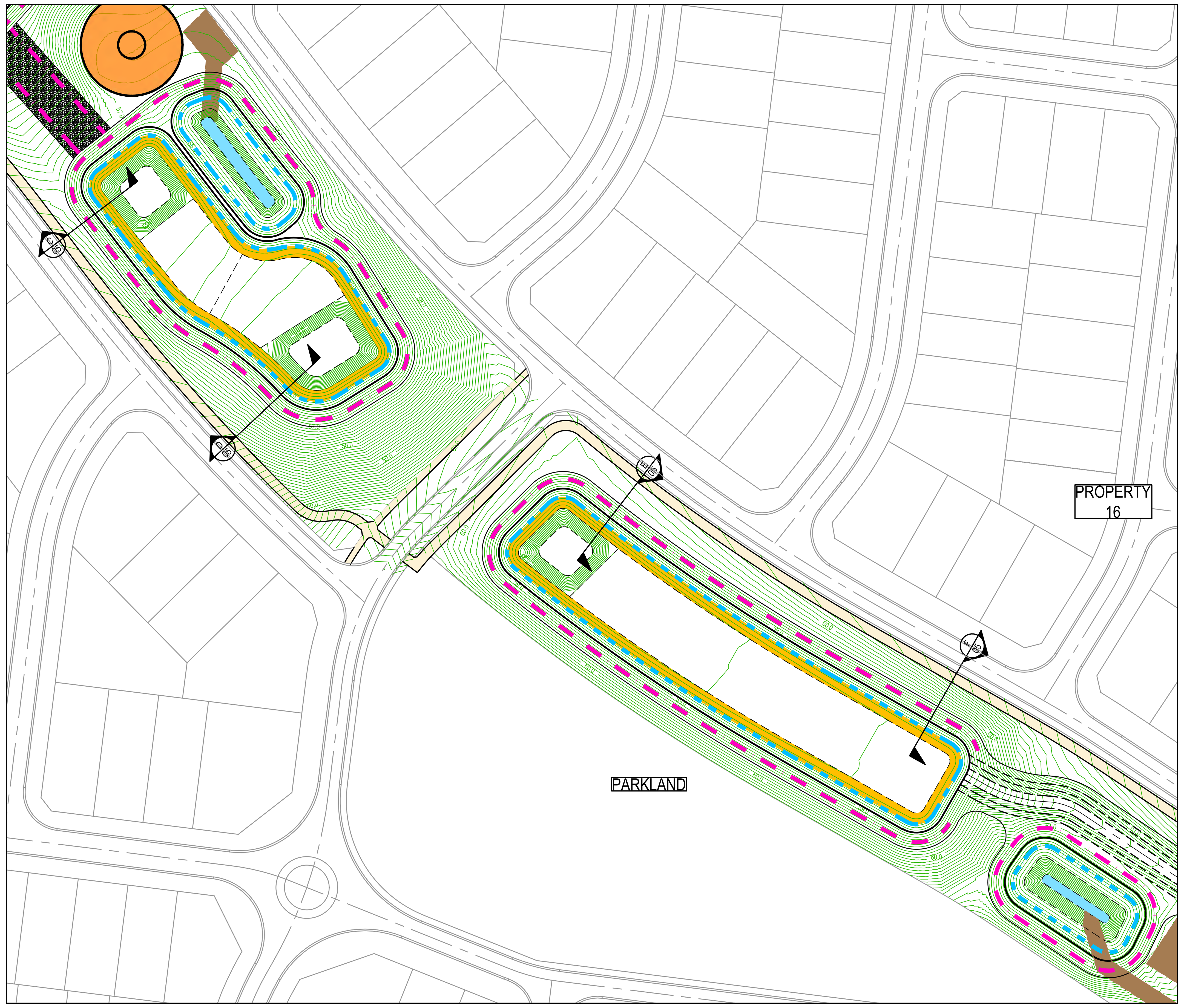

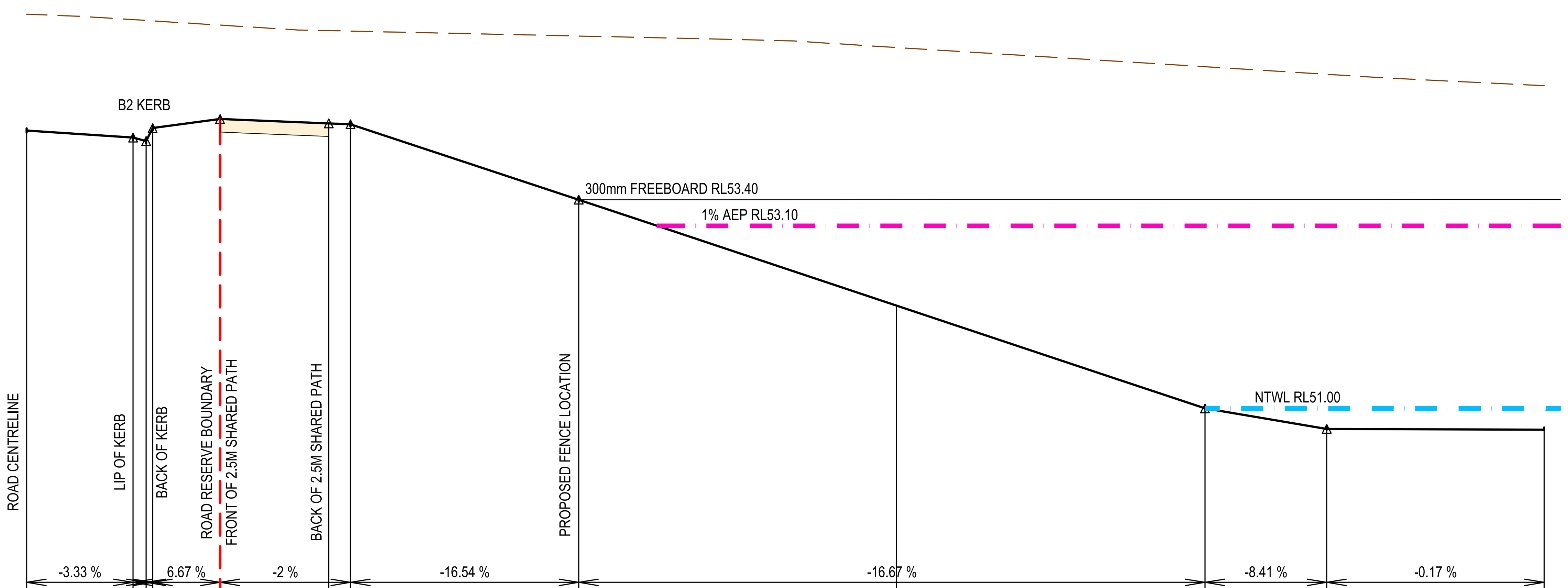

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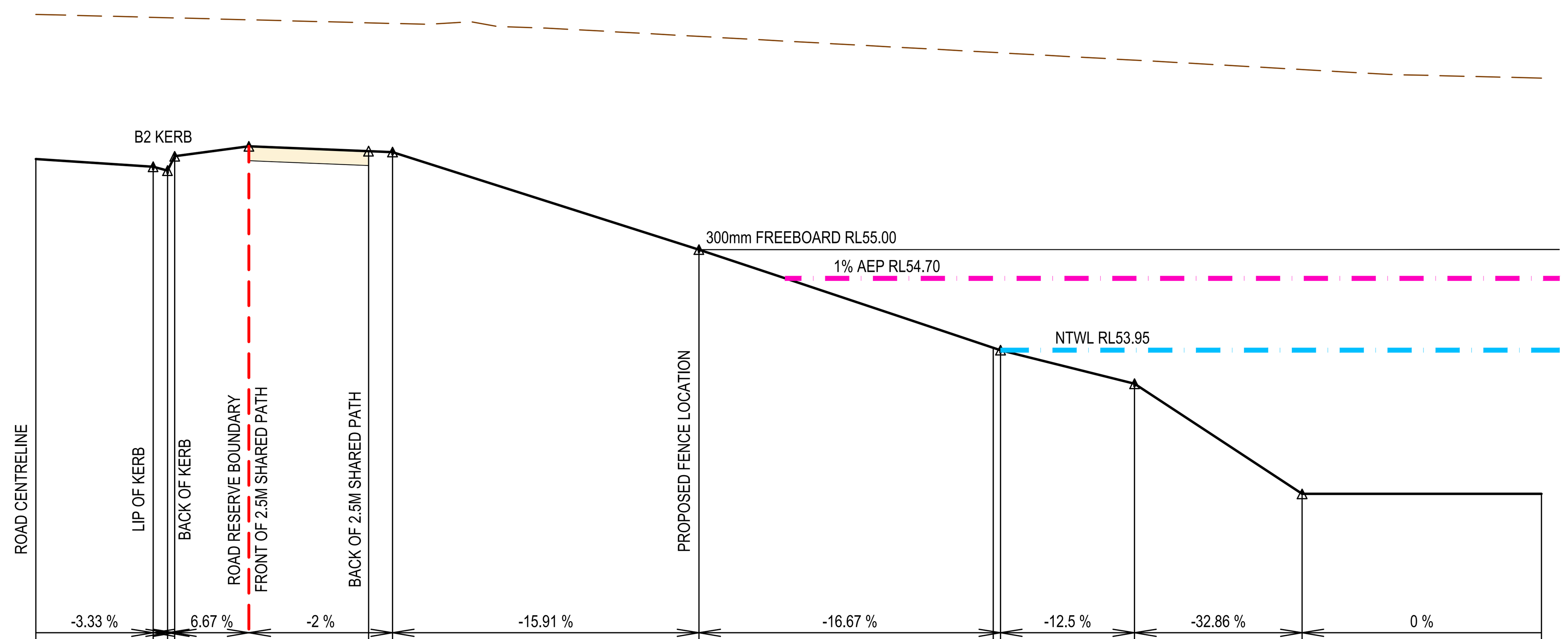
EXISTING SURFACE	55.53	55.47	55.46	55.46	55.41		55.34	55.28	55.15	54.93	54.84	54.71
CHAINAGE	0.00	2.45	2.75	2.90	4.45	6.95	7.45	12.70	20.00	27.10	29.90	34.90

A LONGITUDINAL SECTION



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EXISTING SURFACE		57.42	57.42	57.42	57.40		57.36	57.23	57.06	57.05	56.98	56.88	56.79
CHAINAGE	0.00	2.45	2.75	2.90	4.45	6.95	7.45	13.85	20.00	20.15	22.95	26.45	31.45

B LONGITUDINAL SECTION

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Ph 03 5228 3100

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.	3260E-CL-004	4 of 5	B

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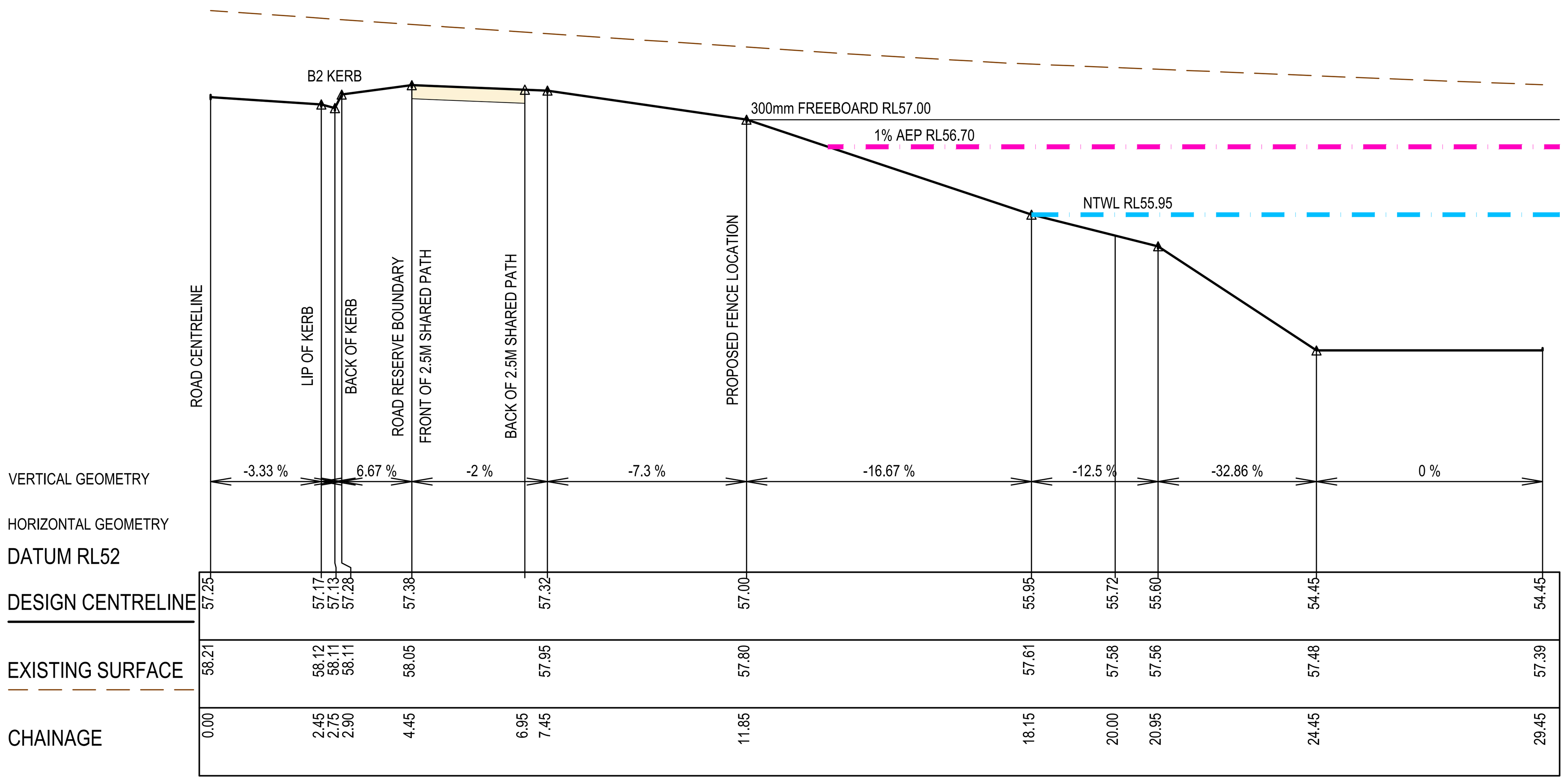
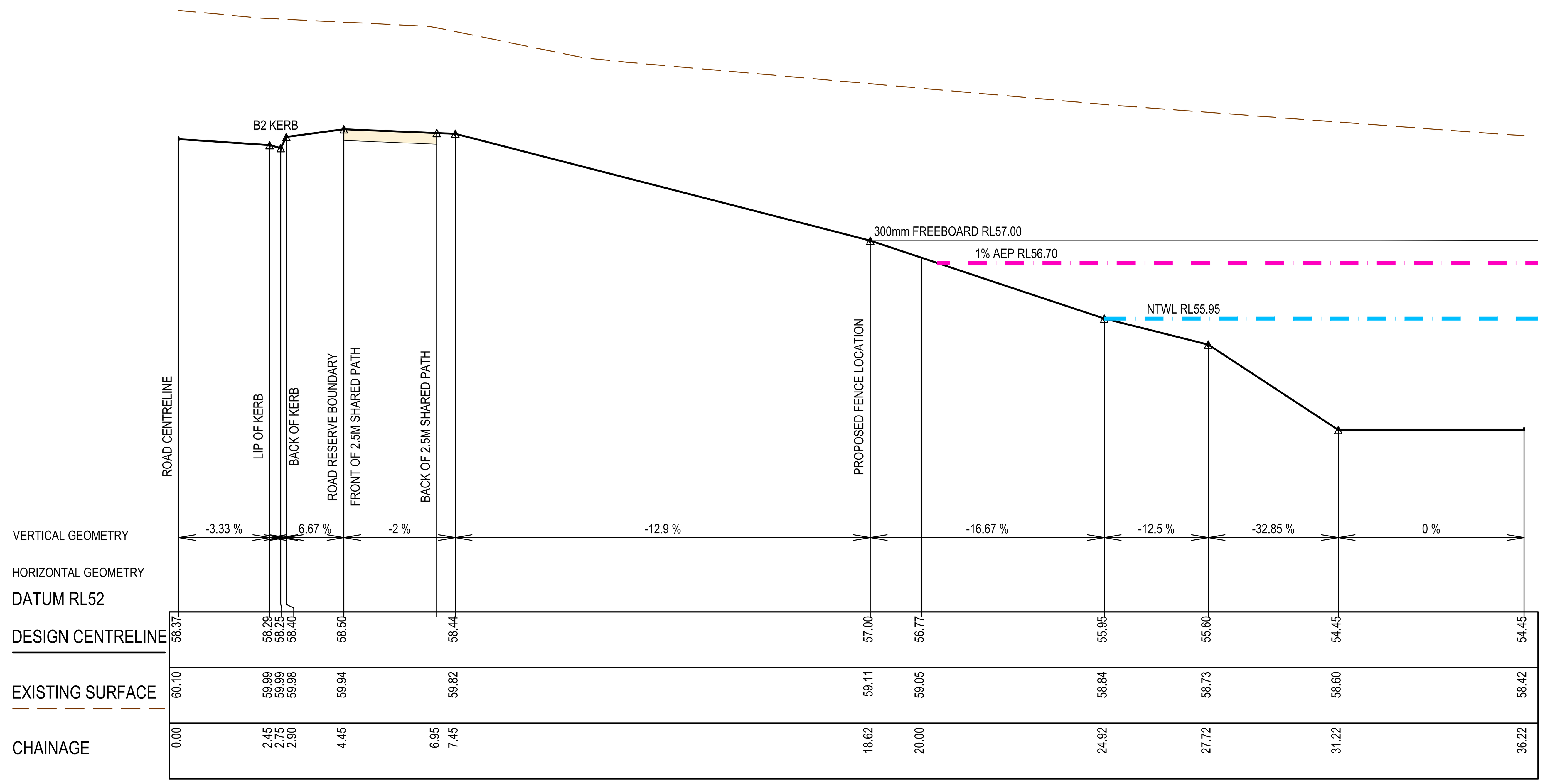
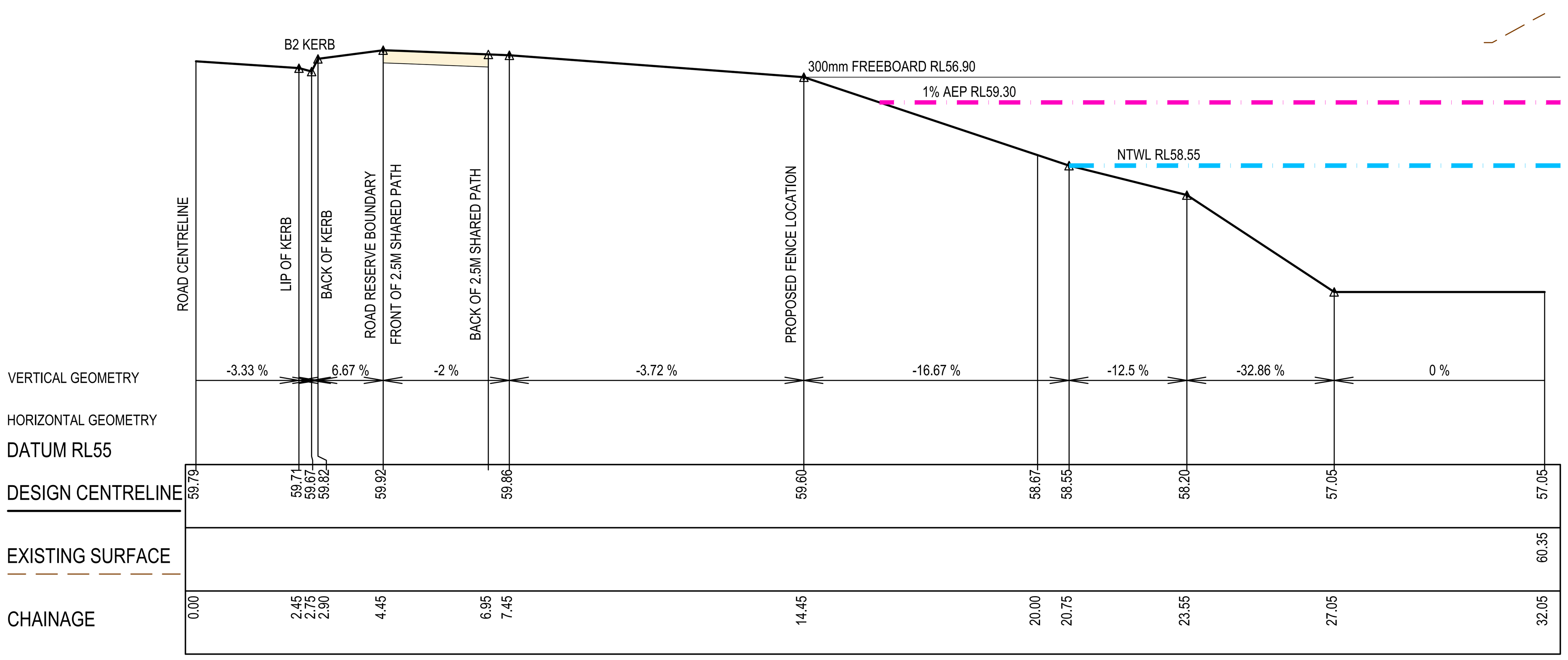


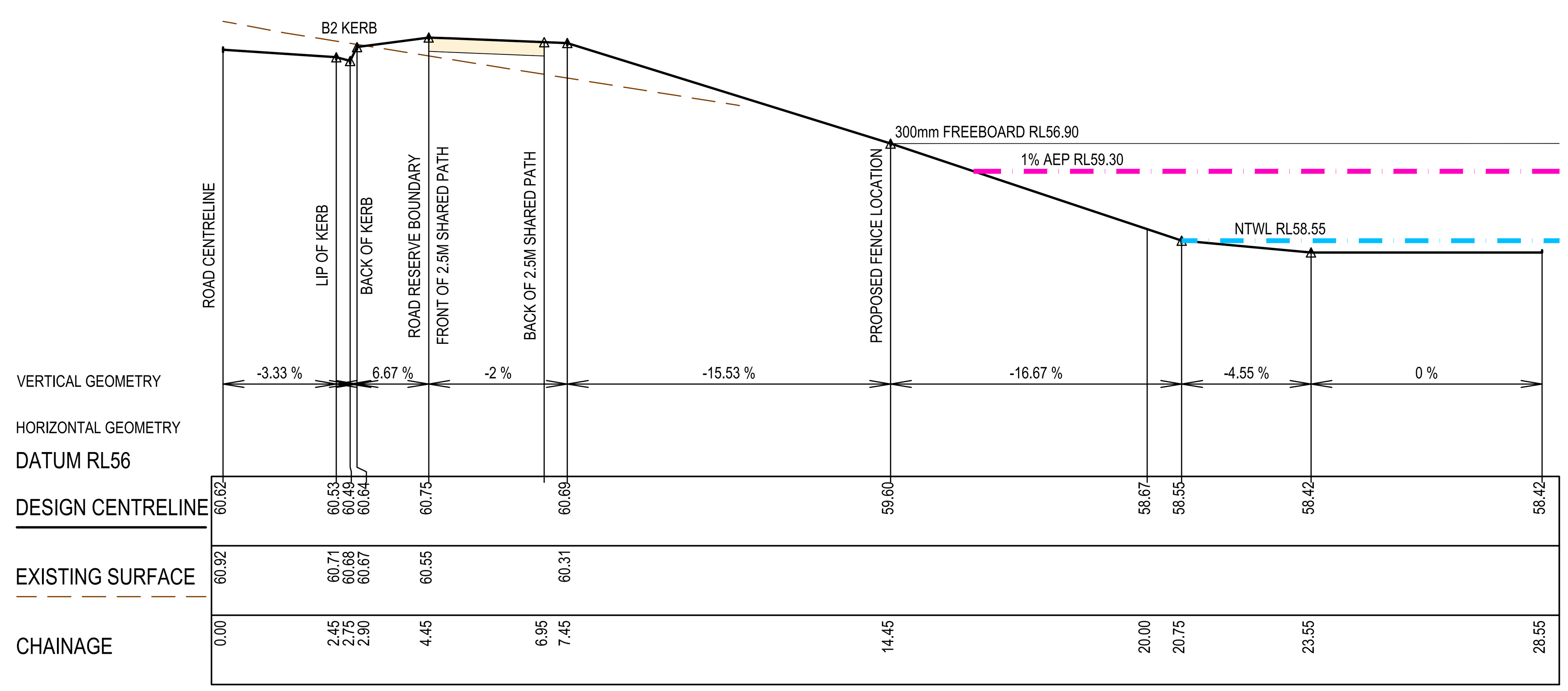
C LONGITUDINAL SECTION



D LONGITUDINAL SECTION



E LONGITUDINAL SECTION



F LONGITUDINAL SECTION

REV	DATE	AMENDMENT / REVISION DESCRIPTION	DRAFTER	DESIGNER	CHECKER	RP ENG
A	29.03.23	ISSUED FOR INFORMATION	T.GOUGH	B.BARBER	R.FORBES	T.MOORFOOT
B	22.06.23	WETLAND CROSS SECTIONS UPDATED	T.GOUGH	R.FORBES	R.FORBES	T.MOORFOOT

Quality Management ISO 9001 · OHS Management AS/NZS 4801 · Environmental Management ISO14001 · Global-Mark.com.au®

PRELIMINARY

RP ENG .
RP ENG NO. .
DATE .

0 5 10 20
Scale 1:500
SCALE AS SHOWN AT A1

N

SMEC
Member of the Surbana Jurong Group
ABN 47 065 475 149
East 5, Federal Mills - 33 Mackey Street
North Geelong, VIC 3215
Ph 03 5228 3100

Curlewis - APD
1421 Portarlington Road
City of Greater Geelong
Drainage Reserve Concept
Slope Cross Sections

MELWAYS REF: .
PROJECT / DRAWING No.: 3260E-CL-005
SHEET No.: 5 of 5
REVISION: B

DWG PATH: C:\Users\MS16697\OneDrive - Surbana Jurong Private Limited\Desktop\Curlewis DCP\X-3260E-CL-005.dwg PRINTED BY: MS16697 on 22/06/2023 at 05:04:01 PM

APPENDIX B
MUSIC AUDITOR TOOL

The following items are noted in relation to the MUSIC AUDIT:

- PPV is based on the 3D modelling of the SMEC design plans and conceptual bathymetry sizing criteria. Minor adjustments to the PVV/bathymetry can be made through the functional/detail design states if desired.
- H-S-Q relationships adopted for the wetland asset nodes

Melbourne Water

Date Completed: 18-07-23
MUSIC Guideline Audited: 2016 Guidelines
MUSIC Version Audited: 6.2 or 6.3
Filename is 087133_GeelongNorth_1971-1990_6min_JettyRdStage2_V8h3b_MUSIC-Auditor.mrt
PDF Download

Source Nodes				
Parameter	User Input	Check	Guideline	Comments
Node (32.335ha) (FI 0.68) does not have any errors. (Node 1)				
Node (12.880ha) (FI 0.63) does not have any errors. (Node 5)				
Node (4.66ha) (FI 0.75) does not have any errors. (Node 6)				
Node (1.88ha) (FI 0.75) does not have any errors. (Node 10)				
Node External Catchment (31.475ha) (FI 0.116) does not have any errors. (Node 11)				
Node External Catchment (58.32ha) (FI 0.53) does not have any errors. (Node 12)				

Treatment Nodes				
Parameter	User Input	Check	Guideline	Comments
Wetland RB (Node 2) Music Help				
Overflow weir width (m)	0.5	<	10	Warning - check is large enough to ensure wetland can overflow freely, if not may result in system filling to unrealistic depths. FAQ
Notional Detention Time (hrs)	42	<	62	Detention time should be approximately 72 hours as recommended in the WSUD Engineering Procedures. The minimum recommended is 48 hours and this should only be used in exceptional circumstances, generally only in retrofit situations with evidence wetland is adequately sized for the catchment or is constrained and has an acceptable inundation frequency.
Wetland WW D/S Cell (Node 3) Music Help				
Overflow weir width (m)	3	<	10	Warning - check is large enough to ensure wetland can overflow freely, if not may result in system filling to unrealistic depths. FAQ

Treatment Nodes				
Notional Detention Time (hrs)	31.8	<	62	Detention time should be approximately 72 hours as recommended in the WSUD Engineering Procedures. The minimum recommended is 48 hours and this should only be used in exceptional circumstances, generally only in retrofit situations with evidence wetland is adequately sized for the catchment or is constrained and has an acceptable inundation frequency.
1 x Sedimentation Basin (Node 4) Music Help				
Permanent pool volume (m3)	205	<	1 * 420 = 420	Shallow permanent pool depth given area, wind action may result in sediment being re-entrained. Check sediment storage volume (50% of permanent pool, is sufficient). FAQ
Overflow weir width (m)	2	<	10	Warning - check is large enough to ensure sediment basin can overflow freely, if not may result in system filling to unrealistic depths. FAQ
Wetland WW Mid Cell (Node 8) Music Help				
Overflow weir width (m)	3	<	10	Warning - check is large enough to ensure wetland can overflow freely, if not may result in system filling to unrealistic depths. FAQ
Notional Detention Time (hrs)	23.6	<	62	Detention time should be approximately 72 hours as recommended in the WSUD Engineering Procedures. The minimum recommended is 48 hours and this should only be used in exceptional circumstances, generally only in retrofit situations with evidence wetland is adequately sized for the catchment or is constrained and has an acceptable inundation frequency.
Wetland WW U/S Cell (Node 9) Music Help				
Overflow weir width (m)	3	<	10	Warning - check is large enough to ensure wetland can overflow freely, if not may result in system filling to unrealistic depths. FAQ
Notional Detention Time (hrs)	31.4	<	62	Detention time should be approximately 72 hours as recommended in the WSUD Engineering Procedures. The minimum recommended is 48 hours and this should only be used in exceptional circumstances, generally only in retrofit situations with evidence wetland is adequately sized for the catchment or is constrained and has an acceptable inundation frequency.

End Date	31/12/1990 11:54:00 PM	not one of	31/12/2001 11:54:00 PM;31/12/1980 11:54:00 PM;31/12/1961 11:54:00 PM;31/12/1980 11:54:00 PM;31/12/1993 11:54:00 PM;31/12/2004 11:54:00 PM	Should be based one of the recommended Melbourne Water regional templates un alternative period is provided and justified.
Rainfall Station	087133 GEELONG	not one of	087033 LITTLE; 086282 MELBOURNE; 086071 MELBOURNE; 086314 KOO; 086085 NARRE; 086142 TOOLANGI	Should be one of the recommended Melbourne Water regional templates unless ar alternative period is provided and justified as being well suited for the site taking i consideration relevant climate conditions. This should generally be based on a wei average mean annual rainfall of multiple daily rainfall stations in proximity of the region of interest with an appropriate corresponding period of mostly complete 6 r data chosen. Standard templates are available on Melbourne Water's website.
Mean Annual Rainfall (mm)	531	not one of	472;575;708;769;932;1221	Should be based one of the recommended Melbourne Water regional templates un alternative period is provided and justified.
Mean Annual ET (mm)	1108	not one of	1067;1041;995;1008;985;962	Should be based one of the recommended Melbourne Water regional templates un alternative period is provided and justified. Note that the PET station nearest or m representative of the site may be chosen in preference to the template.

APPENDIX C
SEDIMENT BASIN COMPUTATIONS

Sedimentation Basin design flow rates were adopted based on Rational method and/or scaled RORB output. This is deemed suitable, as it is industry practice that the sedimentation basin assets size(s) are driver by the 5 year cleanout frequency.

Calculation of Sediment Pond Size 1

This spreadsheet has been updated by Cintia Dotto of WT (June 2017)

We do not accept any responsibility for any errors contained within the spreadsheet
or for any assumptions made by any future user.
The user accepts full responsibility in relation to the use and verification of the calculations detailed within the spreadsheet.

The spreadsheet cannot be passed onto any third party (eg Other consultants or contractors)
without prior written permission by WT.

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

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

$$\lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$$

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

Enter the values in the cells in green

Calculations

Sediment Target =	**Very fine sand**			*Very fine saand for standard residential devlopments*
Vs =	0.011	m/s	This value changes for different particle size target	
d_e =	0.35	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)	
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.37			
Q =	0.37	m^3/s	Ration Method	
A =	**690**	m^2	Area of the sediment basin at NWL	
L/W =	1.4		Length/Width Ratio (assuming rectangular shape)	
$\frac{V_s}{Q/A}$ =	20.51			
λ =	0.11		Pond shape assumption (see figure 10.5 above)	
n =	1.12			

Fraction of Initial Solids Removed

R = **97.43%**

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

Cleanout Frequency

Catchment area =	29.1	ha	Just urban catchment	
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	5 Year accumulation V 291.30 (m3)

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 = 294.35 m^3 — Basin volume at 500 mm below the NWL

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3/y)}{\text{ActualBasinVolume } (m^3)}$ 0.20 per year — Clean out every 5.05 years

Option 2 *Assumes clean out when basin 50% full (WSUD Manual, 2005)*

Actual basin depth = 0.75 m — 0.5 of the dp
Actual Basin Volume = 183.32 m^3 — Volume at half of dp

Therefore, cleanout frequency required = $\frac{(1.6+0.4)A_{catchment} =}{\text{ActualBasinVolume}}$ 0.32 per year — Clean out every 3.1 years

Try to minimise cleanouts - ideally, once every 5 years

Dewatering Area

Dewatering depth =	0.50	m	Max depostion height *## max 0.5 m for MW; 0.3 m good practice among some Councils*
Sediment volume collected every 5 years =	294.35	m^3	volume of sediment acumulated up to 0.5 m below the NWL
Required Dewatering area =	588.70	m^2	

Calculation of Sediment Pond Size 2

This spreadsheet has been updated by Cintia Dotto of WT (June 2017)

We do not accept any responsibility for any errors contained within the spreadsheet
or for any assumptions made by any future user.
The user accepts full responsibility in relation to the use and verification of the calculations detailed within the spreadsheet.

The spreadsheet cannot be passed onto any third party (eg Other consultants or contractors)
without prior written permission by WT.

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

Enter the values in the cells in green

Calculations

Sediment Target =	**Very fine sand**		*Very fine saand for standard residential devlopments*	
Vs =	0.011	m/s	This value changes for different particle size target	
d_e =	0.35	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)	
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.37			
Q =	0.13	m^3/s	Ration Method	
A =	**420**	m^2	Area of the sediment basin at NWL	
L/W =	2.2		Length/Width Ratio (assuming rectangular shape)	
$\frac{V_s}{Q/A}$ =	35.54			
λ =	0.11		Pond shape assumption (see figure 10.5 above)	
n =	1.12			

Fraction of Initial Solids Removed

R = **98.59%**

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

Cleanout Frequency

Catchment area =	10.5	ha	Just urban catchment	
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	5 Year accumulation V 104.70 (m3)

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 =	105.95	m^3	Basin volume at 500 mm below the NWL

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3/y)}{\text{ActualBasinVolume } (m^3)}$ 0.20 per year — Clean out every 5.06 years

Option 2 *Assumes clean out when basin 50% full (WSUD Manual, 2005)*

Actual basin depth =	0.75	m	0.5 of the dp
Actual Basin Volume =	57.15	m^3	Volume at half of dp

Therefore, cleanout frequency required = $\frac{(1.6+0.4)A_{catchment}}{\text{ActualBasinVolume}}$ = 0.37 per year — Clean out every 2.7 years

Try to minimise cleanouts - ideally, once every 5 years

Dewatering Area

Dewatering depth =	0.50	m	Max depostion height *## max 0.5 m for MW; 0.3 m good practice among some Councils*
Sediment volume collected every 5 years =	105.95	m^3	volume of sediment acumulated up to 0.5 m below the NWL
Required Dewatering area =	211.90	m^2	

Calculation of Sediment Pond Size - SED 3

This spreadsheet has been updated by Cintia Dotto of WT (June 2017)

We do not accept any responsibility for any errors contained within the spreadsheet
or for any assumptions made by any future user.
The user accepts full responsibility in relation to the use and verification of the calculations detailed within the spreadsheet.

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

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

$$\lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$$

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

Enter the values in the cells in green

Calculations

Sediment Target =	**Very fine sand**		*Very fine saand for standard residential devlopments*	
Vs =	0.011	m/s	This value changes for different particle size target	
d_e =	0.35	m	Extended Detention Depth	*max 0.35 for MW*
d_p =	1.3	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)	
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.22			
Q =	0.06	m^3/s	Ration Method	
A =	**320**	m^2	Area of the sediment basin at NWL	
L/W =	2.5		Length/Width Ratio (assuming rectangular shape)	
$\frac{V_s}{Q/A}$ =	58.67			
λ =	0.11		Pond shape assumption (see figure 10.5 above)	
n =	1.12			

Fraction of Initial Solids Removed

R = **99.08%**

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

Cleanout Frequency

Catchment area =	4.7	ha	Just urban catchment		
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	5 Year accumulation V	46.60 (m3)

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

Actual basin depth =	0.80	m	500 mm below the NWL (i.e. dp-0.5)
Actual Basin volume =	47.79	m^3	Basin volume at 500 mm below the NWL

Therefore, cleanout frequency required = $\frac{\text{Catchment Load } (m^3/y)}{\text{ActualBasinVolume } (m^3)}$ 0.20 per year — Clean out every 5.13 years

Option 2 *Assumes clean out when basin 50% full (WSUD Manual, 2005)*

Actual basin depth =	0.65	m	0.5 of the dp
Actual Basin Volume =	27.35	m^3	Volume at half of dp

Therefore, cleanout frequency required = $\frac{(1.6+0.4)A_{catchment} =}{\text{ActualBasinVolume}}$ 0.34 per year — Clean out every 2.9 years

Try to minimise cleanouts - ideally, once every 5 years

Dewatering Area

Dewatering depth =	0.50	m	Max depostion height *## max 0.5 m for MW; 0.3 m good practice among some Councils*
Sediment volume collected every 5 years =	47.79	m^3	volume of sediment acumulated up to 0.5 m below the NWL
Required Dewatering area =	95.58	m^2	

APPENDIX D
WETLAND ANALYSIS TOOL RESULTS

Wetland Analysis Tool

Welcome to the Wetland Analysis Tool for checking compliance with the Melbourne Water Constructed Wetland Manual. This tool assesses the wetland depths relative to plant heights and the wetland residence time and advises whether the Deemed to Comply requirements are satisfied.

Please enter the '<u>Shallow marsh zone planting depth</u>' and '<u>Deep marsh zone plating depth</u>'.

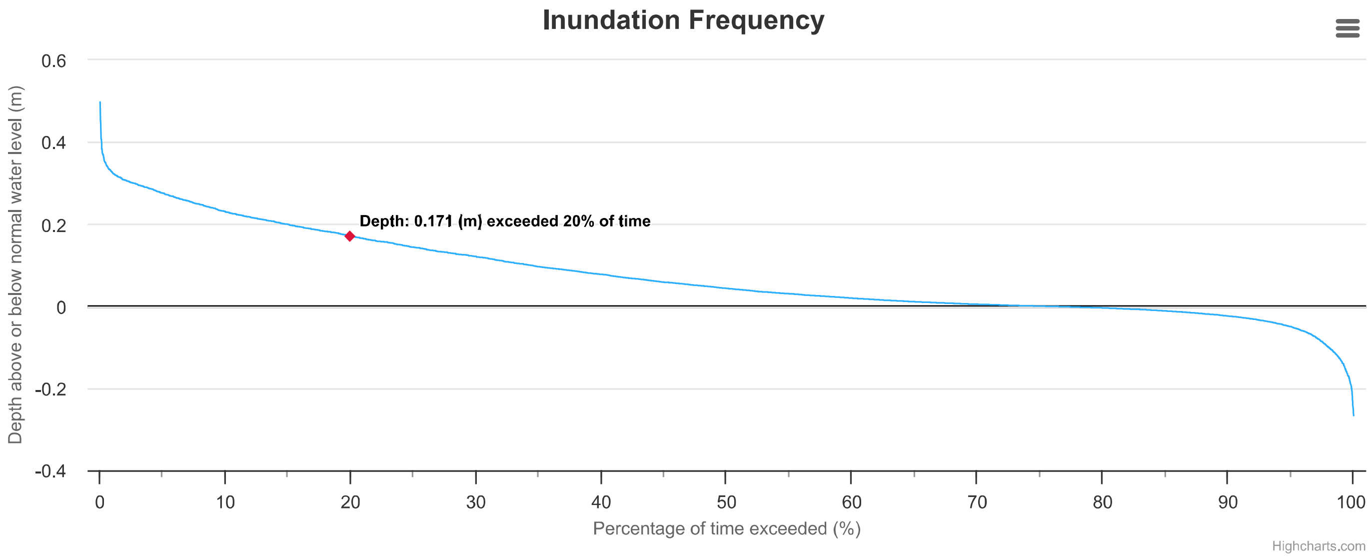
Shallow Planting Depth 0.15 m
Deep Planting Depth 0.35 m

Please enter the permanent pool volume.

Permanent Pool Volume 800 m�

Please select the <u>daily flux file</u> generated in MUSIC for a wetland.
The file must be generated with MUSIC Version 6 and be a �DAILY� flux file.

Choose File MUSIC FLU…- WL US.csv
FILE IS UPLOADED



Please select at least 3 plants for each of the shallow and deep marsh zones.

Clear Selection

Name	Average plant height (m)	Shallow marsh plants	Deep marsh plants	Suitability
Sea Club-rush *Bolboschoenus caldwellii*	1	☑		Shallow Only
Water Ribbons *Triglochin procerum*	1	☑	☐	

Jointed Club-rush *Baumea articulata*	1.8	☑	☐	
Tall Club-rush *Bolboschoenus fluviatilis*	1.8	☑	☐	
Marsh Club-rush *Bolboschoenus medianus*	1.5	☐	☑	Shallow and Deep
Leafy Twig-rush *Cladium procerum*	2	☐	☑	
River Club-rush *Schoenoplectus tabernaemontani*	1.8	☐	☑	
Tall Spike-rush *Eleocharis sphacelata*	1.5		☐	Deep Only
Common reed *Phragmites australis*	2.5		☐	
Common Spike-rush *Eleocharis acuta*	0.5	☐		Unsuitable

[+] Add user defined plant

Report

File: MUSIC FLUX v8h3b - WL US.csv
Shallow marsh zone meets deemed to comply criteria

Deep marsh zone meets deemed to comply criteria

Water level exceeded for 20% of time: 0.171 m

Water level exceeded for 50% of time: 0.0434 m
·Effective water level is within 50 mm of normal water level and is acceptable.

90th Percentile Residence Time: 1 days

☑ Spells Analysis?

Spells Analysis

Please see [water levels and spells for wetland plants](#) for preliminary guidance on suitable water level spells
Note: depth threshold of 0 corresponds to normal water level

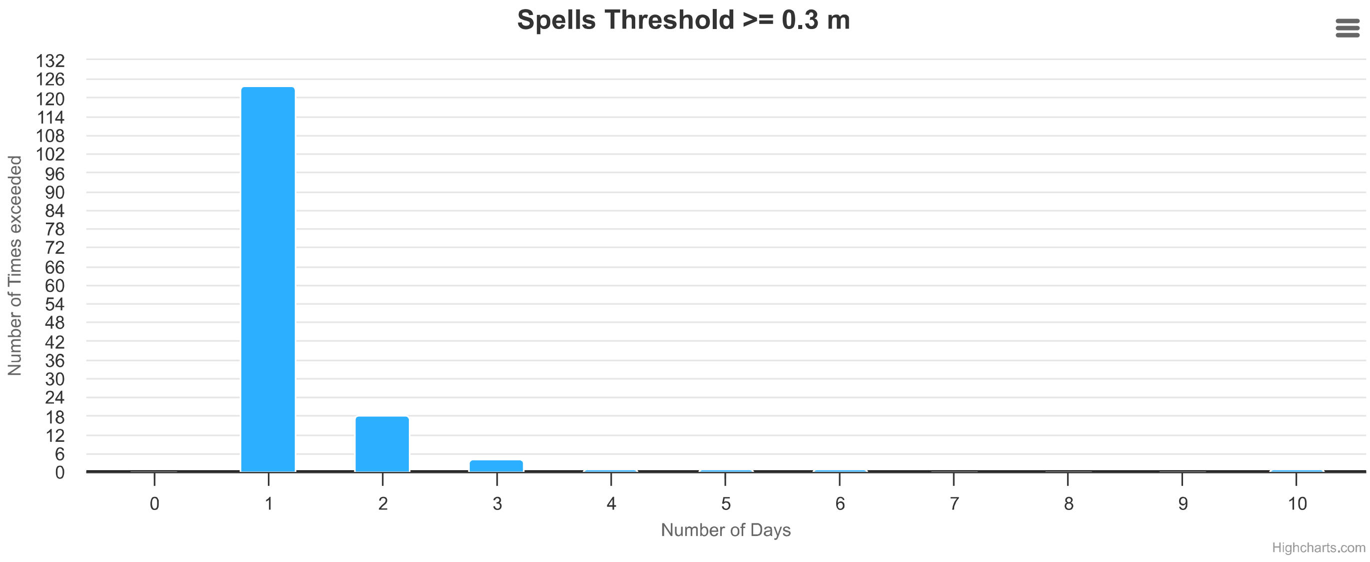
Find spells with water level that is:

Depth threshold(m)
>= 0.3

[+] Add depth threshold

[Clear Selection]

Spells Threshold Results



Spells Threshold >= 0.3 m
Number of Times exceeded
132
126
120
114
108
102
96
90
84
78
72
66
60
54
48
42
36
30
24
18
12
6
0
0
1
2
3
4
5
6
7
8
9
10
Number of Days
Highcharts.com

Wetland Analysis Tool

Welcome to the Wetland Analysis Tool for checking compliance with the Melbourne Water Constructed Wetland Manual. This tool assesses the wetland depths relative to plant heights and the wetland residence time and advises whether the Deemed to Comply requirements are satisfied.

Please enter the '[Shallow marsh zone planting depth]' and '[Deep marsh zone plating depth]'.

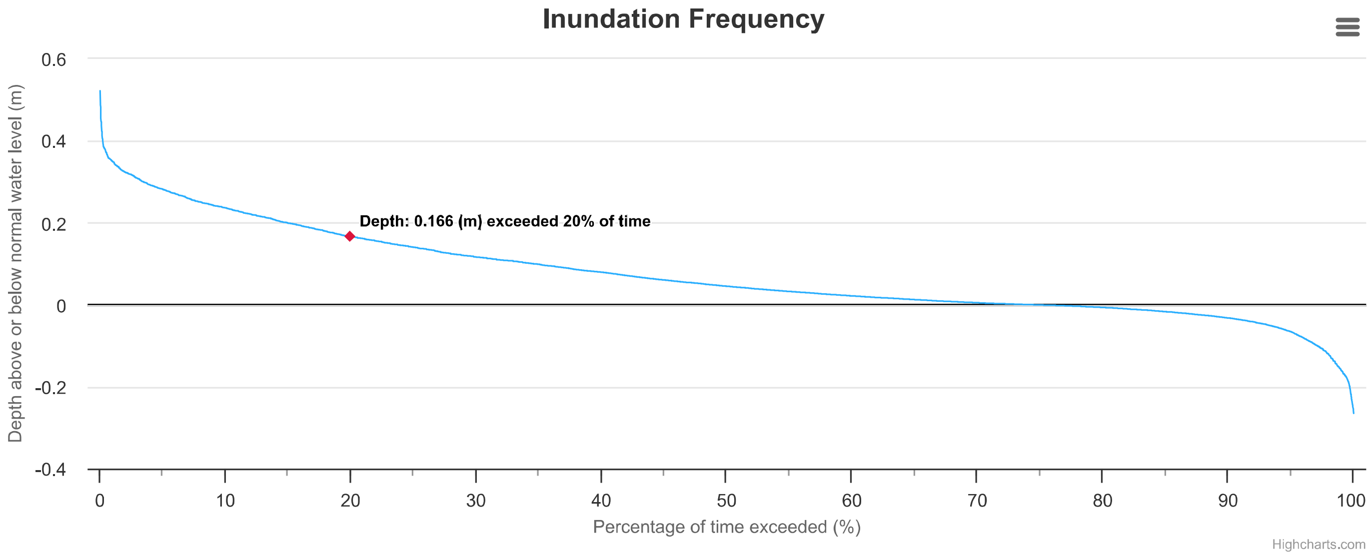
Shallow Planting Depth 0.15 m
Deep Planting Depth 0.35 m

Please enter the permanent pool volume.

Permanent Pool Volume 836 m�

Please select the daily flux file generated in MUSIC for a wetland.
The file must be generated with MUSIC Version 6 and be a �DAILY� flux file.

Choose File MUSIC FLU...- WL Mid.csv
FILE IS UPLOADED



Please select at least 3 plants for each of the shallow and deep marsh zones.

Clear Selection

Name	Average plant height (m)	Shallow marsh plants	Deep marsh plants	Suitability
Sea Club-rush *Bolboschoenus caldwellii*	1	☑		Shallow Only
Water Ribbons *Triglochin procerum*	1	☑	☐	

Jointed Club-rush *Baumea articulata*	1.8	☑	☐	
Tall Club-rush *Bolboschoenus fluviatilis*	1.8	☐	☑	
Marsh Club-rush *Bolboschoenus medianus*	1.5	☐	☑	Shallow and Deep
Leafy Twig-rush *Cladium procerum*	2	☐	☑	
River Club-rush *Schoenoplectus tabernaemontani*	1.8	☐	☐	
Tall Spike-rush *Eleocharis sphacelata*	1.5		☐	Deep Only
Common reed *Phragmites australis*	2.5		☐	
Common Spike-rush *Eleocharis acuta*	0.5	☐		Unsuitable

[+] Add user defined plant

Report

File: MUSIC FLUX v8h3b - WL Mid.csv
Shallow marsh zone meets deemed to comply criteria

Deep marsh zone meets deemed to comply criteria

Water level exceeded for 20% of time: 0.166 m

Water level exceeded for 50% of time: 0.04505 m
·Effective water level is within 50 mm of normal water level and is acceptable.

90th Percentile Residence Time: 0 days

☑ Spells Analysis?

Spells Analysis

Please see water levels and spells for wetland plants for preliminary guidance on suitable water level spells
Note: depth threshold of 0 corresponds to normal water level

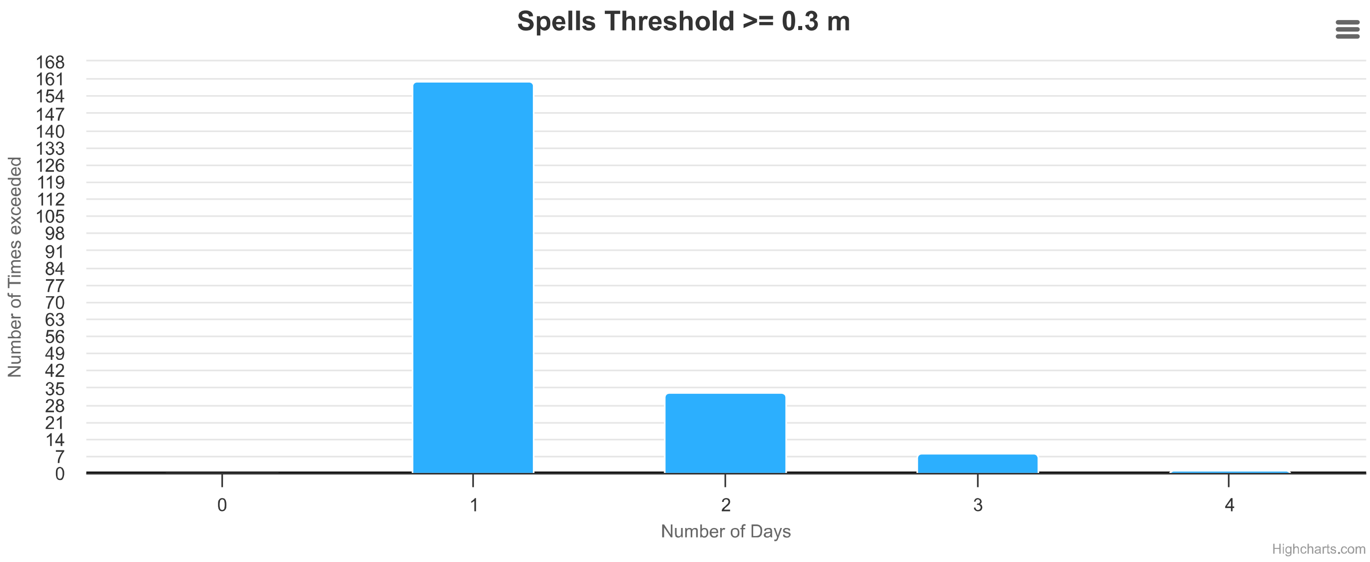
Find spells with water level that is:

Depth threshold(m)
>= 0.3

[+] Add depth threshold

[Clear Selection]

Spells Threshold Results


Spells Threshold >= 0.3 m
Number of Times exceeded
168
161
154
147
140
133
126
119
112
105
98
91
84
77
70
63
56
49
42
35
28
21
14
7
0
0
1
2
3
4
Number of Days
Highcharts.com

Wetland Analysis Tool

Welcome to the Wetland Analysis Tool for checking compliance with the Melbourne Water Constructed Wetland Manual. This tool assesses the wetland depths relative to plant heights and the wetland residence time and advises whether the Deemed to Comply requirements are satisfied.

Please enter the '[Shallow marsh zone planting depth]' and '[Deep marsh zone plating depth]'.

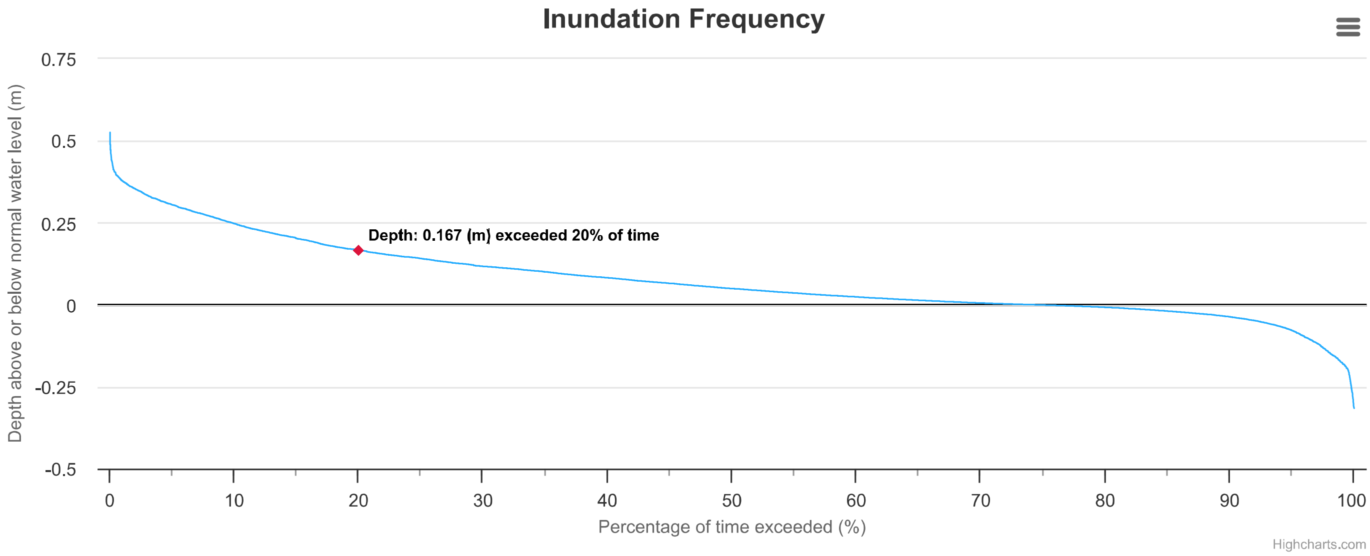
Shallow Planting Depth 0.15 m
Deep Planting Depth 0.35 m

Please enter the permanent pool volume.

Permanent Pool Volume 1126 m�

Please select the daily flux file generated in MUSIC for a wetland.
The file must be generated with MUSIC Version 6 and be a �DAILY� flux file.

Choose File MUSIC FLU...- WL DS.csv
FILE IS UPLOADED



Please select at least 3 plants for each of the shallow and deep marsh zones.

Clear Selection

Name	Average plant height (m)	Shallow marsh plants	Deep marsh plants	Suitability
Sea Club-rush *Bolboschoenus caldwellii*	1	☑		Shallow Only
Water Ribbons *Triglochin procerum*	1	☑	☐	

Jointed Club-rush *Baumea articulata*	1.8	[x]	[ ]	
Tall Club-rush *Bolboschoenus fluviatilis*	1.8	[ ]	[x]	
Marsh Club-rush *Bolboschoenus medianus*	1.5	[ ]	[x]	Shallow and Deep
Leafy Twig-rush *Cladium procerum*	2	[ ]	[x]	
River Club-rush *Schoenoplectus tabernaemontani*	1.8	[ ]	[ ]	
Tall Spike-rush *Eleocharis sphacelata*	1.5		[ ]	Deep Only
Common reed *Phragmites australis*	2.5		[ ]	
Common Spike-rush *Eleocharis acuta*	0.5	[ ]		Unsuitable

[+] Add user defined plant

Report

File: MUSIC FLUX v8h3b - WL DS.csv
Shallow marsh zone meets deemed to comply criteria

Deep marsh zone meets deemed to comply criteria

Water level exceeded for 20% of time: 0.167 m

Water level exceeded for 50% of time: 0.0492 m
·Effective water level is within 50 mm of normal water level and is acceptable.

90th Percentile Residence Time: 0 days

[x] Spells Analysis?

Spells Analysis

Please see water levels and spells for wetland plants for preliminary guidance on suitable water level spells
Note: depth threshold of 0 corresponds to normal water level

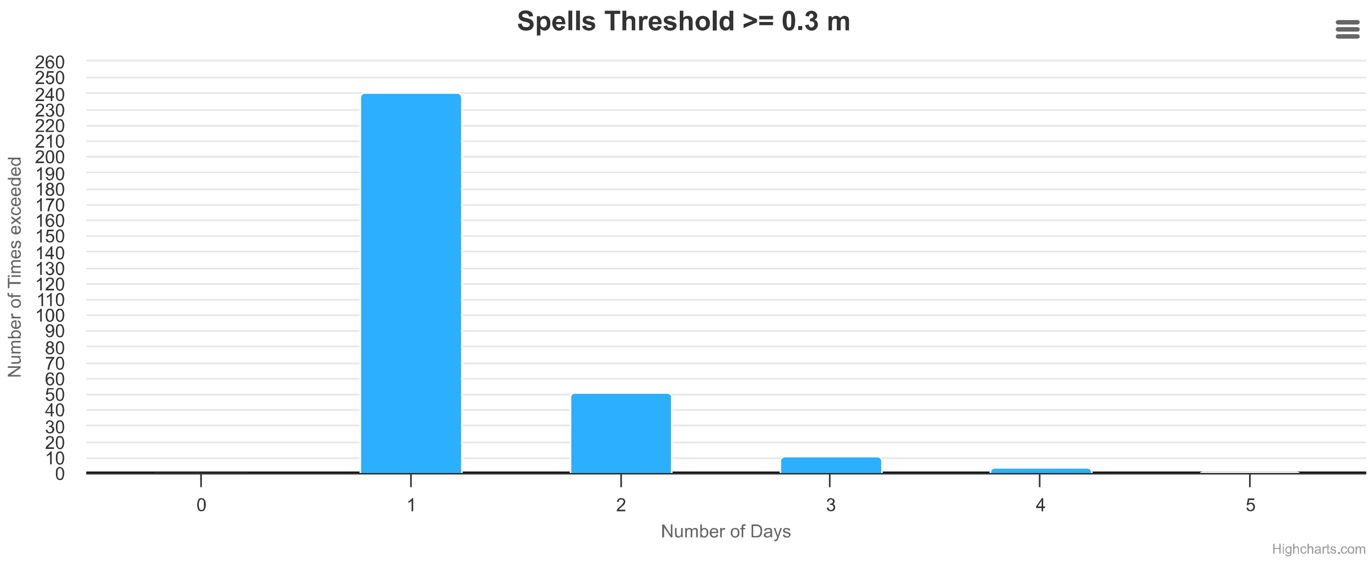
Find spells with water level that is:

Depth threshold(m)
>= 0.3

[+] Add depth threshold

[Clear Selection]

Spells Threshold Results



Spells Threshold >= 0.3 m
Number of Times exceeded
260
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
5
Number of Days
Highcharts.com

Wetland Analysis Tool

Welcome to the Wetland Analysis Tool for checking compliance with the Melbourne Water Constructed Wetland Manual. This tool assesses the wetland depths relative to plant heights and the wetland residence time and advises whether the Deemed to Comply requirements are satisfied.

Please enter the 'Shallow marsh zone planting depth' and 'Deep marsh zone plating depth'.

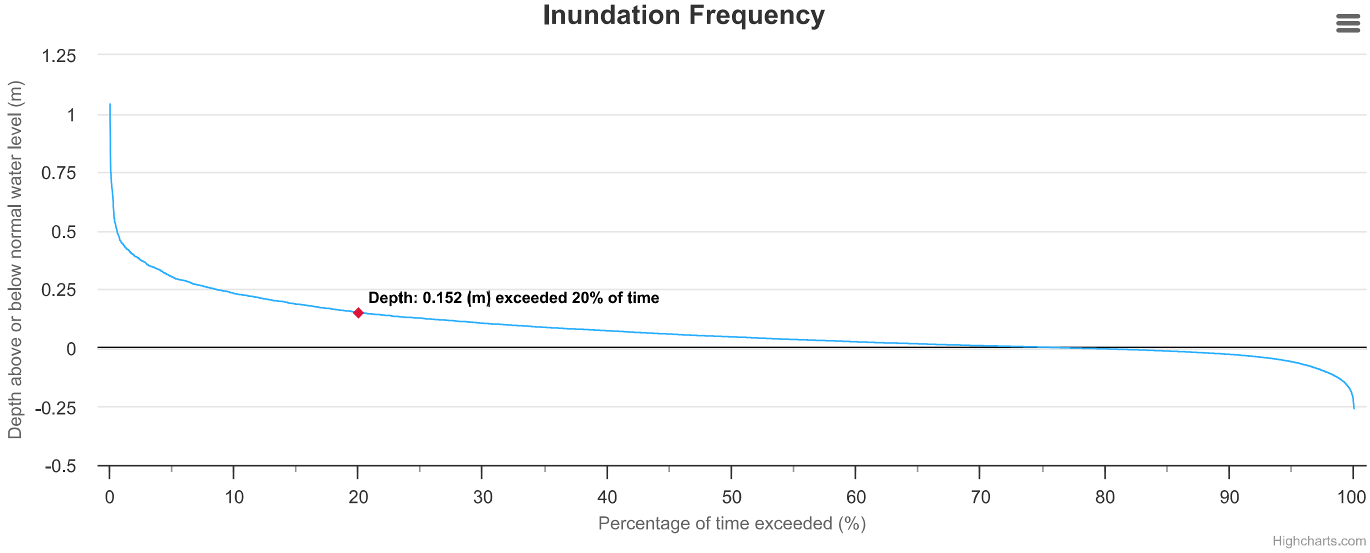
Shallow Planting Depth 0.15 m
Deep Planting Depth 0.35 m

Please enter the permanent pool volume.

Permanent Pool Volume 1166 m�

Please select the daily flux file generated in MUSIC for a wetland.
The file must be generated with MUSIC Version 6 and be a �DAILY� flux file.

Choose File MUSIC FLU...- WL RB.csv
FILE IS UPLOADED



Please select at least 3 plants for each of the shallow and deep marsh zones.

Clear Selection

Name	Average plant height (m)	Shallow marsh plants	Deep marsh plants	Suitability
Sea Club-rush *Bolboschoenus caldwellii*	1	☑		Shallow Only
Water Ribbons *Triglochin procerum*	1	☑	☐	

Jointed Club-rush *Baumea articulata*	1.8	☑	☐	
Tall Club-rush *Bolboschoenus fluviatilis*	1.8	☐	☑	
Marsh Club-rush *Bolboschoenus medianus*	1.5	☐	☑	Shallow and Deep
Leafy Twig-rush *Cladium procerum*	2	☐	☑	
River Club-rush *Schoenoplectus tabernaemontani*	1.8	☐	☐	
Tall Spike-rush *Eleocharis sphacelata*	1.5		☐	Deep Only
Common reed *Phragmites australis*	2.5		☐	
Common Spike-rush *Eleocharis acuta*	0.5	☐		Unsuitable

[+] Add user defined plant

Report

File: MUSIC FLUX v8h3b - WL RB.csv
Shallow marsh zone meets deemed to comply criteria

Deep marsh zone meets deemed to comply criteria

Water level exceeded for 20% of time: 0.152 m

Water level exceeded for 50% of time: 0.0472 m
·Effective water level is within 50 mm of normal water level and is acceptable.

90th Percentile Residence Time: 1 days

☑ Spells Analysis?

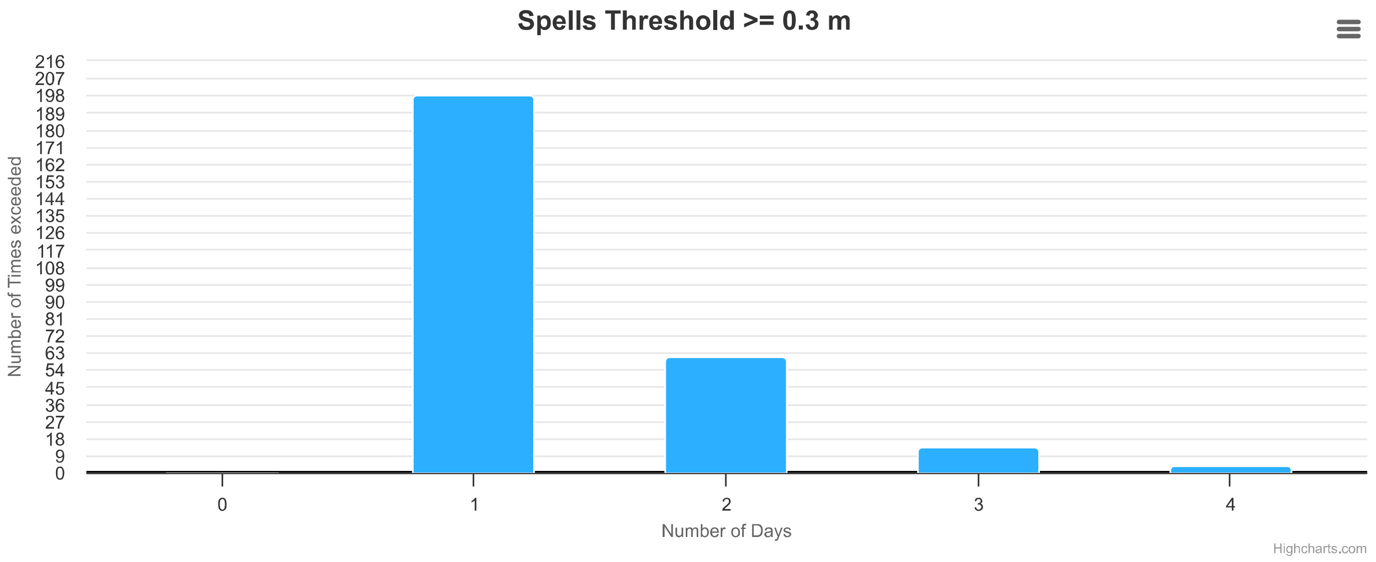
Spells Analysis

Please see water levels and spells for wetland plants for preliminary guidance on suitable water level spells
Note: depth threshold of 0 corresponds to normal water level
Find spells with water level that is:

Depth threshold(m)
>= 0.3

[+] Add depth threshold

[Clear Selection]
Spells Threshold Results


Spells Threshold >= 0.3 m
Number of Times exceeded
216
207
198
189
180
171
162
153
144
135
126
117
108
99
90
81
72
63
54
45
36
27
18
9
0
0
1
2
3
4
Number of Days
Highcharts.com

APPENDIX E
WETLAND VELOCITY COMPUTATIONS

Initial Wetland Flow Velocity Checks

Initial Velocity Checks

Q_{100} =	6.4	m^3/s (RORB)
Q_{10} =	2.88	m^3/s (RORB scaled)
Q_5 =	2.24	m^3/s (RORB scaled)
Q1 year	1.0500	m^3/s (RORB)
Q_{3mth} =	0.4200	Scaled from Q1
Q_{3mth} =	0.4200	**Adopted**

	0.35	m EDD	
Wetland Normal Water Level (NWL) =	10.000	m AHD	Generic NWL adopted for comps -
Wetland Top of Extended Detention (TED) =	10.350	m AHD	not representive of deign levels
Base level at wetland narrowest width =	9.850	m AHD	Not adopted for computations
	(0.15 deep conservatively)		

1a	Peak 10 yr flow through sediment pond =	2.88	m^3/s (RORB)	
	Peak 100 yr flow through sediment pond =	6.4	m^3/s (RORB)	
1b	Bypass around Macrophyte zone =	0.00	m^3/s	5 yr capacity - wetland
	Macrophyte zone inlet capacity =	0.42	m^3/s	
	Peak 3 month flow through macrophyte zone =	0.4	m^3/s	
	Peak 10 yr flow through macrophyte zone =	2.9	m^3/s (accounts for bypass)	
	Peak 100 yr flow through macrophyte zone =	6.4	m^3/s (accounts for bypass)	

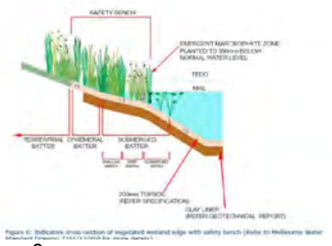
Initial Sediment Pond Velocity Check

	assumed depth above EDD	0.260	
2	RORB 10 yr WL =	10.610	m AHD
	At narrowest part of the sediment pond:		
3a	NWL width =	**20**	m
3b	Width at 10 yr WL =	26.1	m
4	10 yr WL - NWL =	0.6	m
	Average width =	23.05	m
	Cross section flow area =	14.0605	m^2
5	100 yr Flow velocity =	0.46	m/s < 0.5 m/s OK

Initial Macrophyte zone Velocity Check

	assumed depth above EDD	0.260	
6	RORB 10 yr WL =	10.610	m AHD
	At narrowest part of the macrophyte zone:		
7a	NWL width =	**22.5**	m
7b	TED width =	26	m
7c	Width at 10 yr WL =	28.6	m
8a	TED - base level at narrowest wi	0.35	m (very conservative - at narrowest width base is actually 1.0
	Average width =	24.25	m
	Cross section flow area =	8.4875	m^2
8b	10 yr WL - NWL =	0.6	m
	Average width =	27.3	m
	Cross section flow area =	16.653	m^2
9	3 month Flow velocity =	0.049	m/s < 0.05 m/s OK
10	100 Year ARI Flow velocity =	0.38	m/s < 0.5 m/s OK

Minimum width bathymetry



assumed pool base	2.000	m			
half	1	m			
	delta width	delta depth			
Base	1	0			
TOP of 1 in 3	3.45	1.150	1 in	3	slope
Top of Safety Benc	2.8	0.35	1 in	8	slope
				0.35	
total	**7.25**	1.500			
Minimum width	**14.5**				

Summary

Minimum WL width (WL only) **22.5** m — Based on 3 month velocity max of 0.05m/s

Initial Wetland Flow Velocity Checks

Initial Velocity Checks

Q_{100} (West catchment only) =	2.99	m^3/s (Rational method)
Q_{100} =	8.67	m^3/s (RORB)
Q_{10} =	3.9015	m^3/s (RORB scaled)
Q_5 =	3.0345	m^3/s (RORB scaled)
Q1 year (total U/S waterway flows) =	1.3600	m^3/s (RORB- refer Gayani comment - right snip>>)
Q_{3mth} (total U/S waterway flows) =	0.5440	m^3/s (RORB scaled)
Q_{3mth} (west 18.2ha catchment only) =	0.2900	m^3/s
Q_{3mth} =	**0.2900**	**Adopted** <<Assumes U/S Q3month waterway flows bypass RB WL

	0.35	m EDD	
Wetland Normal Water Level (NWL) =	10.000	m AHD	Generic NWL adopted for comps - not
Wetland Top of Extended Detention (TED) =	10.350	m AHD	representive of deign levels
Base level at wetland narrowest width =	9.900	m AHD	Not adopted for computations
	(0.1 deep conservatively)		

1a	Peak 10 yr flow through sediment pond =	3.9015	m^3/s (RORB)
	Peak 100 yr flow through sediment pond =	8.67	m^3/s (RORB)
1b	Bypass around Macrophyte zone =	0.00	m^3/s 5 yr capacity - wetland inflow
	Macrophyte zone inlet capacity =	0.29	m^3/s
	Peak 3 month flow through macrophyte zone =	0.3	m^3/s
	Peak 10 yr flow through macrophyte zone =	3.9	m^3/s (accounts for bypass)
	Peak 100 yr flow through macrophyte zone =	8.7	m^3/s (accounts for bypass)

Initial Sediment Pond Velocity Check

	assumed depth above EDD	0.900	
2	RORB 10 yr WL =	11.250	m AHD
	At narrowest part of the sediment pond:		
3a	NWL width =	**8**	m
3b	Width at 10 yr WL =	20.5	m
4	10 yr WL - NWL =	1.3	m
	Average width =	14.25	m
	Cross section flow area =	17.8125	m^2
5	100 yr Flow velocity (WW flow)	0.49	m/s < 0.5 m/s OK
	100 yr Flow velocity (West catcl	0.17	m/s < 0.5 m/s OK

Initial Macrophyte zone Velocity Check

	assumed depth above EDD	0.900	
6	RORB 10 yr WL =	11.250	m AHD
	At narrowest part of the macrophyte zone:		
7a	NWL width =	**15**	m
7b	TED width =	18.5	m
7c	Width at 10 yr WL =	27.5	m
		0	
8a	TED - base level at narrowest w	0.35	m (very conservative - at narrowest width base is actually 1.0
	Average width =	16.75	m
	Cross section flow area =	5.8625	m^2
8b	10 yr WL - NWL =	1.3	m
	Average width =	23	m
	Cross section flow area =	28.75	m^2
9	3 month Flow velocity =	0.049	m/s < 0.05 m/s OK
10	100 Year ARI Flow velocity =	0.30	m/s < 0.5 m/s OK

28

Minimum width bathymetry

assumed pool base width min		2.000 m			
	half	1 m			
	delta widtł	delta depth			
Base	1	0			
TOP of 1 in 3	3.45	1.150	1 in 3		slope
Top of Safety Bench	2.8	0.35	1 in 8		slope
				0.35	
total	**7.25**	1.500			
Minimum width	**14.5**				

Summary

Minimum WL width (WL only) **15** m Based on 3 month velocity max of 0.05m/s

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

APPENDIX F
MUSIC H-S-Q

The below tables summarise the stage-storage relationship adopted for the RORB modelling design. Depth 0 is at the NWL.

Table 0-1 RORB U/S Cell Stage-Storage-Discharge Relationship

Stage	H (Depth m)	S (Storage m^3)	Q (Discharge m^3/s)
58.55	0	0.00	0.000
58.65	0.1	285.85	0.002
58.75	0.2	589.01	0.006
58.85	0.3	909.70	0.012
58.95	0.4	1248.20	0.076
59.05	0.5	1605.09	0.327
59.15	0.6	1980.65	0.682
59.25	0.7	2375.11	1.116
59.35	0.8	2788.70	1.617
59.45	0.9	3221.75	2.136
59.55	1	3674.55	2.492
59.65	1.1	4149.57	2.875
59.75	1.2	4659.28	3.283
59.85	1.3	5201.18	3.713
59.95	1.4	5770.62	4.166
60.05	1.5	6363.73	4.639
60.15	1.6	6978.58	5.133
60.25	1.7	7613.08	5.645
60.35	1.8	8264.41	6.176
60.45	1.9	8929.99	6.724
60.55	2	9609.68	7.290
60.65	2.1	10300.02	7.872
60.75	2.2	10997.86	8.471
60.85	2.3	11700.60	9.085
60.95	2.4	12406.65	9.714
61.05	2.5	13115.80	10.359
61.15	2.6	13827.97	11.017
61.25	2.7	14542.30	11.691
61.35	2.8	15257.33	12.378
61.45	2.9	15972.40	13.079
61.55	3	16687.47	13.793

Stage	H (Depth m)	S (Storage m^3)	Q (Discharge m^3/s)
61.65	3.1	17402.54	14.520
61.75	3.2	18117.62	15.260
61.85	3.3	18832.69	16.013
61.95	3.4	19547.76	16.779
62.05	3.5	20262.83	17.556

Table 0-2 RORB MID Cell Stage-Storage-Discharge Relationship

Stage	H (Depth m)	S (Storage m^3)	Q (Discharge m^3/s)
55.95	0	0.00	0.000
56.05	0.1	214.92	0.003
56.15	0.2	442.65	0.007
56.25	0.3	683.13	0.013
56.35	0.4	936.49	0.043
56.45	0.5	1198.88	0.215
56.55	0.6	1466.67	0.648
56.65	0.7	1737.52	1.235
56.75	0.8	2009.58	1.941
56.85	0.9	2281.76	2.748
56.95	1	2553.94	3.443
57.05	1.1	2826.11	4.175
57.15	1.2	3098.29	4.963
57.25	1.3	3370.47	5.804
57.35	1.4	3642.64	6.695
57.45	1.5	3914.82	7.632
57.55	1.6	4187.00	8.613
57.65	1.7	4459.18	9.638
57.75	1.8	4731.35	10.703
57.85	1.9	5003.53	11.807
57.95	2	5275.71	12.949
58.05	2.1	5547.88	14.127
58.15	2.2	5820.06	15.341
58.25	2.3	6092.24	16.590
58.35	2.4	6364.41	17.872
58.45	2.5	6636.59	19.187

Stage	H (Depth m)	S (Storage m^3)	Q (Discharge m^3/s)
58.55	2.6	6908.77	20.533

Table 0-3 RORB D/S Cell Stage-Storage-Discharge Relationship

Stage	H (Depth m)	S (Storage m^3)	Q (Discharge m^3/s)
53.95	0	0.00	0.000
54.05	0.1	287.80	0.003
54.15	0.2	588.71	0.007
54.25	0.3	902.94	0.013
54.35	0.4	1230.72	0.020
54.45	0.5	1572.30	0.254
54.55	0.6	1927.89	0.676
54.65	0.7	2297.72	1.221
54.75	0.8	2681.86	1.864
54.85	0.9	3080.00	2.593
54.95	1	3491.62	3.399
55.05	1.1	3912.25	4.275
55.15	1.2	4334.43	5.215
55.25	1.3	4756.61	6.216
55.35	1.4	5178.78	7.274
55.45	1.5	5600.96	8.386
55.55	1.6	6023.13	9.549
55.65	1.7	6445.31	10.762
55.75	1.8	6867.49	12.022
55.85	1.9	7289.66	13.328
55.95	2	7711.84	14.679
56.05	2.1	8134.01	16.072
56.15	2.2	8556.19	17.506
56.25	2.3	8978.37	18.981
56.35	2.4	9400.54	20.495
56.45	2.5	9822.72	22.048

Table 0-4 RORB RB/WL Cell Stage-Storage-Discharge Relationship

Stage	H (Depth m)	S (Storage m^3)	Q (Discharge m^3/s)
51	0	0.00	0.000
51.1	0.1	363.95	0.002
51.2	0.2	747.65	0.005
51.3	0.3	1151.55	0.009
51.4	0.4	1578.58	0.023
51.5	0.5	2027.60	0.069
51.6	0.6	2495.80	0.132
51.7	0.7	2983.40	0.208
51.8	0.8	3490.62	0.296
51.9	0.9	4017.66	0.394
52	1	4564.75	0.501
52.1	1.1	5132.09	0.616
52.2	1.2	5719.89	0.739
52.3	1.3	6328.38	0.869
52.4	1.4	6957.76	1.018
52.5	1.5	7608.25	1.111
52.6	1.6	8280.06	1.204
52.7	1.7	8973.40	1.313
52.8	1.8	9688.50	1.437
52.9	1.9	10425.55	1.572
53	2	11184.79	1.718
53.1	2.1	11966.34	1.874
53.2	2.2	12769.61	2.039
53.3	2.3	13592.99	2.213
53.4	2.4	14434.72	2.396
53.5	2.5	15294.57	2.586
53.6	2.6	16163.01	2.784
53.7	2.7	17038.72	2.989
53.8	2.8	17921.54	3.201
53.9	2.9	18811.08	3.420
54	3	19706.38	3.645
54.1	3.1	20606.80	3.876
54.2	3.2	21511.20	4.114
54.3	3.3	22417.73	4.357

Stage	H (Depth m)	S (Storage m^3)	Q (Discharge m^3/s)
54.4	3.4	23325.57	4.606
54.5	3.5	24234.45	4.861
54.6	3.6	25144.12	5.121
54.7	3.7	26054.22	5.387
54.8	3.8	26964.38	5.658
54.9	3.9	27874.54	5.934
55	4	28784.70	6.215
55.1	4.1	29694.86	6.501

The below tables summarise the stage-storage relationship adopted for the MUSIC modelling design. Depth 0 is at the NWL.

Table 0-5 U/S CELL MUSIC CUSTOM PARAMETERS

Pipe Flow		Weir Flow		Storage	
H (m)	Pipe Flow (m^3/s)	H (m)	Weir Flow (m^3/s)	H (m)	Storage (m^3)
0	0.000	0	0.000	0	800
0.1	0.0023	0.1	0.000	0.1	1086
0.2	0.006	0.2	0.000	0.2	1389
0.3	0.012	0.3	0.000	0.3	1710
0.4	0.018	0.4	0.058	0.4	2048
0.5	0.026	0.5	0.301	0.5	2405
0.6	0.034	0.6	0.648	0.6	2781
0.7	0.042	0.7	1.074	0.7	3175
0.8	0.052	0.8	1.565	0.8	3589
0.9	0.062	0.9	2.074	0.9	4022
1	0.072	1	2.420	1	4475
1.1	0.083	1.1	2.792	1.1	4950
1.2	0.095	1.2	3.188	1.2	5459
1.3	0.107	1.3	3.606	1.3	6001
1.4	0.120	1.4	4.046	1.4	6571
1.5	0.133	1.5	4.507	1.5	7164
1.6	0.146	1.6	4.987	1.6	7779
1.7	0.160	1.7	5.485	1.7	8413

Pipe Flow		Weir Flow		Storage	
1.8	0.174	1.8	6.001	1.8	9064
1.9	0.189	1.9	6.535	1.9	9730
2	0.204	2	7.086	2	10410
2.1	0.220	2.1	7.652	2.1	11100
2.2	0.236	2.2	8.235	2.2	11798
2.3	0.252	2.3	8.833	2.3	12501
2.4	0.269	2.4	9.446	2.4	13207

Table 0-6 MID CELL MUSIC CUSTOM PARAMETERS

Pipe Flow		Weir Flow		Storage	
H (m)	Pipe Flow (m^3/s)	H (m)	Weir Flow (m^3/s)	H (m)	Storage (m^3)
0	0.000	0	0.000	0	836
0.1	0.003	0.1	0.000	0.1	1051
0.2	0.007	0.2	0.000	0.2	1279
0.3	0.013	0.3	0.000	0.3	1519
0.4	0.020	0.4	0.023	0.4	1772
0.5	0.029	0.5	0.187	0.5	2035
0.6	0.038	0.6	0.611	0.6	2303
0.7	0.047	0.7	1.187	0.7	2574
0.8	0.058	0.8	1.883	0.8	2846
0.9	0.069	0.9	2.680	0.9	3118
1	0.081	1	3.362	1	3390
1.1	0.093	1.1	4.082	1.1	3662
1.2	0.106	1.2	4.857	1.2	3934
1.3	0.120	1.3	5.684	1.3	4206
1.4	0.134	1.4	6.561	1.4	4479
1.5	0.148	1.5	7.483	1.5	4751
1.6	0.163	1.6	8.450	1.6	5023
1.7	0.179	1.7	9.459	1.7	5295
1.8	0.195	1.8	10.508	1.8	5567
1.9	0.211	1.9	11.595	1.9	5840
2	0.228	2	12.720	2	6112

Pipe Flow		Weir Flow		Storage	
2.1	0.246	2.1	13.882	2.1	6384
2.2	0.263	2.2	15.078	2.2	6656
2.3	0.282	2.3	16.308	2.3	6928
2.4	0.300	2.4	17.572	2.4	7200

Table 0-7 D/S CELL MUSIC CUSTOM PARAMETERS

Pipe Flow		Weir Flow		Storage	
H (m)	Pipe Flow (m^3/s)	H (m)	Weir Flow (m^3/s)	H (m)	Storage (m^3)
0	0.000	0	0.000	0	1126
0.1	0.003	0.1	0.000	0.1	1414
0.2	0.007	0.2	0.000	0.2	1715
0.3	0.013	0.3	0.000	0.3	2029
0.4	0.020	0.4	0.000	0.4	2357
0.5	0.029	0.5	0.226	0.5	2698
0.6	0.038	0.6	0.639	0.6	3054
0.7	0.047	0.7	1.173	0.7	3424
0.8	0.058	0.8	1.806	0.8	3808
0.9	0.069	0.9	2.524	0.9	4206
1	0.081	1	3.318	1	4618
1.1	0.093	1.1	4.182	1.1	5038
1.2	0.106	1.2	5.109	1.2	5460
1.3	0.120	1.3	6.096	1.3	5883
1.4	0.134	1.4	7.140	1.4	6305
1.5	0.148	1.5	8.237	1.5	6727
1.6	0.163	1.6	9.386	1.6	7149
1.7	0.179	1.7	10.583	1.7	7571
1.8	0.195	1.8	11.827	1.8	7993
1.9	0.211	1.9	13.117	1.9	8416
2	0.228	2	14.450	2	8838
2.1	0.246	2.1	15.826	2.1	9260
2.2	0.263	2.2	17.243	2.2	9682
2.3	0.282	2.3	18.699	2.3	10104

Pipe Flow		Weir Flow		Storage	
2.4	0.300	2.4	20.195	2.4	10527

Table 0-8 RB/WL MUSIC CUSTOM PARAMETERS

Pipe Flow		Weir Flow		Storage	
H (m)	Pipe Flow (m^3/s)	H (m)	Weir Flow (m^3/s)	H (m)	Storage (m^3)
0	0.000	0	0.000	0	1166
0.1	0.002	0.1	0.000	0.1	1530
0.2	0.005	0.2	0.000	0.2	1914
0.3	0.009	0.3	0.000	0.3	2318
0.4	0.014	0.4	0.010	0.4	2745
0.5	0.020	0.5	0.049	0.5	3194
0.6	0.026	0.6	0.106	0.6	3662
0.7	0.032	0.7	0.176	0.7	4149
0.8	0.040	0.8	0.257	0.8	4657
0.9	0.047	0.9	0.347	0.9	5184
1	0.055	1	0.445	1	5731
1.1	0.064	1.1	0.552	1.1	6298
1.2	0.073	1.2	0.666	1.2	6886
1.3	0.082	1.3	0.787	1.3	7494
1.4	0.092	1.4	0.927	1.4	8124
1.5	0.102	1.5	1.010	1.5	8774
1.6	0.112	1.6	1.092	1.6	9446
1.7	0.122	1.7	1.191	1.7	10139
1.8	0.133	1.8	1.303	1.8	10854
1.9	0.145	1.9	1.427	1.9	11592
2	0.156	2	1.561	2	12351
2.1	0.168	2.1	1.706	2.1	13132
2.2	0.180	2.2	1.859	2.2	13936
2.3	0.193	2.3	2.021	2.3	14759
2.4	0.205	2.4	2.190	2.4	15601

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APPENDIX G
CONTROLLING OUTLETS

U/S Cell:

Outlet Arrangement adopted for concept H-S-Q:

- Twin chamber outlet pit for EDD control and frequent storm weir outlet
- Controlling weir pit on U/S end of 1% culverts downstream of the wetland Cell
- H-S relationship determined from SMEC 3D design tin.

Refer to Figure 9 for indicative layout, and below spreadsheet for H-Q computations

It is suitable for the controlling outlets to be refined at detail design stage.

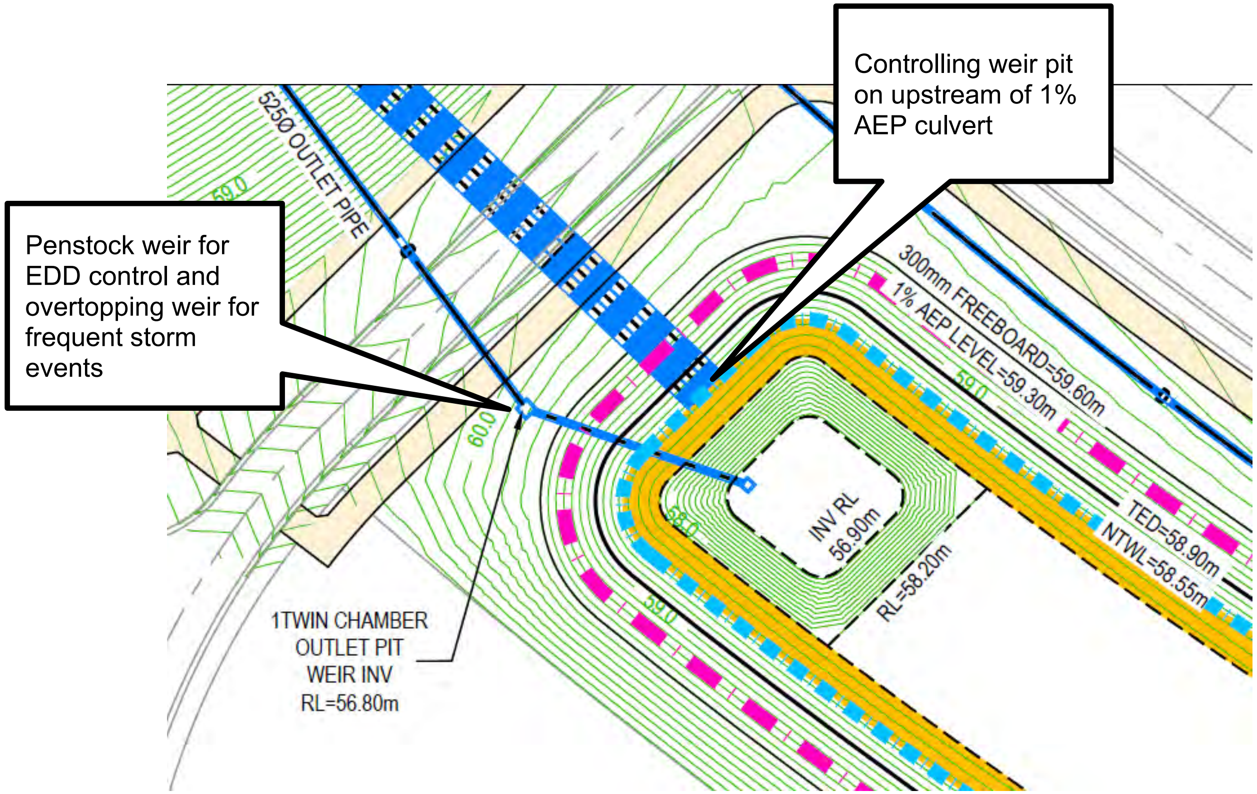

Figure 9 U/S Wetland Cell Concept Outlet Sketch

Ignore the pit?? Yes/No		No	Ignore the pit?? Yes/No	Yes												Are you Assuming INLET CONTROL??? Yes/No			Yes		
PIT			PIT			0	m h		0.35	m h			m h			PIPE	INLET CONTROL		OUTLET CONTROL		
Pit Sill Level	58.55	m AHD	Pit Sill Level	57.5 m AHD		58.55	m AHD		58.9	m AHD		58.9	m AHD			Invert Level	56.9 m AHD		Ke	0.2	Side or slope tapered inlet
Width	0	m	Width	0.9 m												Dia	0.525 m		Kex	1.0	
Length	0	m	Length	0.9 m		0.095	m		1.35	m		1.7	m			barrels	1		Length	35.0	m
Area	0	m²	Area	0.81 m²		0.085										Area	0.216475369 m²		Tailwater Level	28.800	m AHD
portion not taken by b	62	%	portion not taken by b	62 %			0.046					Note - It is assumed a pit will be on U/S end of Q100 culvert - and the pit lip will be controlling capacity. And therefore the D/S									
Grill blockage factor	50	%	Grill blockage factor	50 %			0.017			1.37									wetted perim	1.65	m
All Weir blockage facto	0	%	Weir blockage factor	0 %		EDD Weir blockage	50												Hyd Radius	0.13	m

NOT USED High Flow outlet pit - U/S side of Pens			WL Offtake/Outlet Pit			PenStock Weir			Weir Over Pen stock			Spillway									
Height	Head	Orifice Flow	Head	Orifice Flow	total flow into U/S twin chamber pit	Head	Weir Flow 1	EDD Control Flow	Head	Weir Flow 2	total weir flow including EDD weir	Head	Weir Flow 3	total flow into twin chamber pit D/S chamber of Baffel/weir	total weir flow excuding EDD weir and excluding emergency spill	Head	Orifice Flow	head loss	flow	TOTAL FLOW RATE	
58.55	0	0.000	1.05	0.752	0.752	0	0.000	0.000	0	0.000	0.000	0	0.000	0.000	0.000	1.388	0.745	29.750	NA	0.00	m³/s
58.65	0.1	0.000	1.15	0.787	0.787	0.1	0.003	0.003	0	0.000	0.003	0	0.000	0.003	0.000	1.488	0.772	29.850	NA	0.00	m³/s
58.75	0.2	0.000	1.25	0.821	0.821	0.2	0.007	0.007	0	0.000	0.007	0	0.000	0.007	0.000	1.588	0.797	29.950	NA	0.01	m³/s
58.85	0.3	0.000	1.35	0.853	0.853	0.3	0.013	0.013	0	0.000	0.013	0	0.000	0.013	0.000	1.688	0.822	30.050	NA	0.01	m³/s
58.95	0.4	0.000	1.45	0.884	0.884	0.4	0.020	0.020	0.05	0.026	0.046	0.05	0.032	0.046	0.026	1.788	0.846	30.150	NA	0.08	m³/s
59.05	0.5	0.000	1.55	0.914	0.914	0.5	0.029	0.029	0.15	0.133	0.162	0.15	0.168	0.162	0.133	1.888	0.869	30.250	NA	0.33	m³/s
59.15	0.6	0.000	1.65	0.943	0.943	0.6	0.038	0.038	0.25	0.287	0.324	0.25	0.361	0.324	0.287	1.988	0.892	30.350	NA	0.69	m³/s
59.25	0.7	0.000	1.75	0.971	0.971	0.7	0.047	0.047	0.35	0.475	0.523	0.35	0.598	0.523	0.475	2.088	0.914	30.450	NA	1.12	m³/s
59.35	0.8	0.000	1.85	0.998	0.998	0.8	0.058	0.058	0.45	0.693	0.751	0.45	0.872	0.751	0.693	2.188	0.936	30.550	NA	1.62	m³/s
59.45	0.9	0.000	1.95	1.025	1.025	0.9	0.069	0.069	0.55	0.936	1.005	0.55	1.179	1.005	0.936	2.288	0.957	30.650	NA	2.14	m³/s
59.55	1	0.000	2.05	1.051	1.051	1	0.081	0.081	0.65	1.203	1.283	0.65	1.514	1.051	0.970	2.388	0.978	30.750	NA	2.49	m³/s
59.65	1.1	0.000	2.15	1.076	1.076	1.1	0.093	0.093	0.75	1.491	1.584	0.75	1.877	1.076	0.983	2.488	0.998	30.850	NA	2.88	m³/s
59.75	1.2	0.000	2.25	1.101	1.101	1.2	0.106	0.106	0.85	1.799	1.905	0.85	2.265	1.101	0.995	2.588	1.018	30.950	NA	3.28	m³/s
59.85	1.3	0.000	2.35	1.125	1.125	1.3	0.120	0.120	0.95	2.125	2.245	0.95	2.676	1.125	1.006	2.688	1.037	31.050	NA	3.71	m³/s
59.95	1.4	0.000	2.45	1.149	1.149	1.4	0.134	0.134	1.05	2.469	2.603	1.05	3.109	1.149	1.015	2.788	1.057	31.150	NA	4.17	m³/s
60.05	1.5	0.000	2.55	1.172	1.172	1.5	0.148	0.148	1.15	2.830	2.979	1.15	3.564	1.172	1.024	2.888	1.075	31.250	NA	4.64	m³/s
60.15	1.6	0.000	2.65	1.195	1.195	1.6	0.163	0.163	1.25	3.207	3.371	1.25	4.039	1.195	1.032	2.988	1.094	31.350	NA	5.13	m³/s
60.25	1.7	0.000	2.75	1.217	1.217	1.7	0.179	0.179	1.35	3.600	3.779	1.35	4.533	1.217	1.038	3.088	1.112	31.450	NA	5.65	m³/s
60.35	1.8	0.000	2.85	1.239	1.239	1.8	0.195	0.195	1.45	4.007	4.202	1.45	5.046	1.239	1.044	3.188	1.130	31.550	NA	6.18	m³/s
60.45	1.9	0.000	2.95	1.261	1.261	1.9	0.211	0.211	1.55	4.429	4.640	1.55	5.577	1.261	1.049	3.288	1.147	31.650	NA	6.72	m³/s
60.55	2	0.000	3.05	1.282	1.282	2	0.228	0.228	1.65	4.864	5.093	1.65	6.125	1.282	1.054	3.388	1.165	31.750	NA	7.29	m³/s
60.65	2.1	0.000	3.15	1.303	1.303	2.1	0.246	0.246	1.75	5.313	5.559	1.75	6.690	1.303	1.057	3.488	1.182	31.850	NA	7.87	m³/s
60.75	2.2	0.000	3.25	1.323	1.323	2.2	0.263	0.263	1.85	5.775	6.038	1.85	7.272	1.323	1.060	3.588	1.199	31.950	NA	8.47	m³/s
60.85	2.3	0.000	3.35	1.344	1.344	2.3	0.282	0.282	1.95	6.249	6.531	1.95	7.870	1.344	1.062	3.688	1.215	32.050	NA	9.08	m³/s
60.95	**2.4**	**0.000**	**3.45**	**1.363**	**1.363**	**2.4**	0.300	**0.300**	**2.05**	**6.736**	**7.036**	2.05	**8.483**	**1.363**	**1.063**	**3.788**	**1.232**	**32.150**	**NA**	**9.71**	**m³/s**

Mid Cell:

Outlet Arrangement adopted for concept H-S-Q:

- Twin chamber outlet pit for EDD control and frequent storm weir outlet
- Controlling weir on U/S end of 1% spillway
- H-S relationship determined from SMEC 3D design tin.

Refer to Figure 10 for indicative layout, and below spreadsheet for H-Q computations

It is suitable for the controlling outlets to be refined at detail design stage.

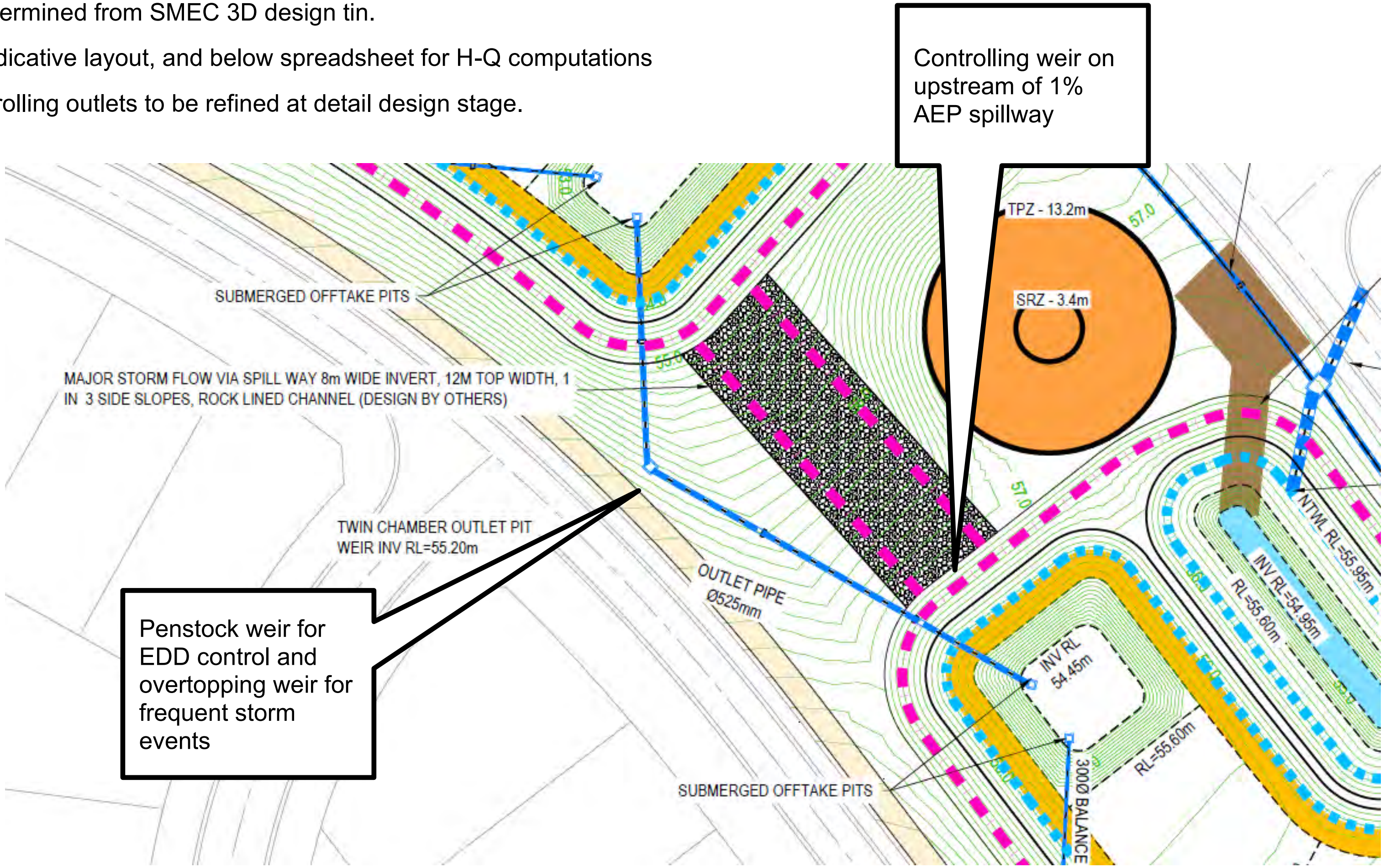

Figure 10 Mid Wetland Cell Concept Outlet Sketch

Ignore the pit?? Yes/No		No	Ignore the pit?? Yes/No		Yes											
PIT			PIT													
							0	m h		0.35	m h			m h		
Pit Sill Level	58.75	m AHD	Pit Sill Level	55	m AHD		55.95	m AHD		56.3	m AHD		56.4	m AHD		
Width	0	m	Width	0.9	m											
Length	0	m	Length	0.9	m		0.120	m		1.2	m		3.6	m		
Area	0	m^2	Area	0.81	m^2		0.095									
portion not taken by b	62	%	portion not taken by b	62	%			0.033								
Grill blockage factor	50	%	Grill blockage factor	50	%			0.021			1.60					
All Weir blockage facto	0	%	Weir blockage factor	0	%		EDD Weir blockage	50								
THIS PIT IS NOT USED) High Flow outlet pit - U/S			WL Offtake/Outlet Pit				PenStock Weir			Weir Over Pen stock			Spillway			

Are you Assuming INLET CONTROL??? Yes/No				Yes	
PIPE	INLET CONTROL		OUTLET CONTROL		
Invert Level	54.45	m AHD	Ke	0.2	Side or slope tapered inlet
Dia	0.525	m	Kex	1.0	
barrels	1		Length	35.0	m
Area	0.216475369	m^2	Tailwater Level	28.800	m AHD
			wetted perim	1.65	m
			Hyd Radius	0.13	m

Height	Head	Orifice Flow	Head	Orifice Flow	total flow into U/S twin chamber pit	Head	Weir Flow 1	EDD Control Flow	Head	Weir Flow 2	total weir flow including EDD weir	Head	Weir Flow 3	total flow into twin chamber pit D/S chamber of Baffel/weir	total weir flow excuding EDD weir and excluding emergency spill	Head	Orifice Flow	head loss	flow	TOTAL FLOW RATE	
55.95	-2.8	0.000	0.95	0.715	0.715	0	0.000	0.000	0	0.000	0.000	0	0.000	0.000	0.000	1.238	0.704	27.150	NA	0.00	m^3/s
56.05	-2.7	0.000	1.05	0.752	0.752	0.1	0.003	0.003	0	0.000	0.003	0	0.000	0.003	0.000	1.338	0.732	27.250	NA	0.00	m^3/s
56.15	-2.6	0.000	1.15	0.787	0.787	0.2	0.009	0.009	0	0.000	0.009	0	0.000	0.009	0.000	1.438	0.759	27.350	NA	0.01	m^3/s
56.25	-2.5	0.000	1.25	0.821	0.821	0.3	0.017	0.017	0	0.000	0.017	0	0.000	0.017	0.000	1.538	0.785	27.450	NA	0.02	m^3/s
56.35	-2.4	0.000	1.35	0.853	0.853	0.4	0.026	0.026	0.05	0.023	0.049	0	0.000	0.049	0.023	1.638	0.810	27.550	NA	0.05	m^3/s
56.45	-2.3	0.000	1.45	0.884	0.884	0.5	0.036	0.036	0.15	0.119	0.155	0.05	0.068	0.155	0.119	1.738	0.834	27.650	NA	0.22	m^3/s
56.55	-2.2	0.000	1.55	0.914	0.914	0.6	0.047	0.047	0.25	0.255	0.302	0.15	0.356	0.302	0.255	1.838	0.858	27.750	NA	0.66	m^3/s
56.65	-2.1	0.000	1.65	0.943	0.943	0.7	0.060	0.060	0.35	0.422	0.482	0.25	0.765	0.482	0.422	1.938	0.881	27.850	NA	1.25	m^3/s
56.75	-2	0.000	1.75	0.971	0.971	0.8	0.073	0.073	0.45	0.616	0.689	0.35	1.267	0.689	0.616	2.038	0.903	27.950	NA	1.96	m^3/s
56.85	-1.9	0.000	1.85	0.998	0.998	0.9	0.087	0.087	0.55	0.832	0.919	0.45	1.847	0.919	0.832	2.138	0.925	28.050	NA	2.77	m^3/s
56.95	-1.8	0.000	1.95	1.025	1.025	1	0.102	0.102	0.65	1.069	1.171	0.55	2.496	1.025	0.923	2.238	0.947	28.150	NA	3.44	m^3/s
57.05	-1.7	0.000	2.05	1.051	1.051	1.1	0.118	0.118	0.75	1.325	1.443	0.65	3.207	1.051	0.933	2.338	0.968	28.250	NA	4.17	m^3/s
57.15	-1.6	0.000	2.15	1.076	1.076	1.2	0.134	0.134	0.85	1.599	1.733	0.75	3.975	1.076	0.942	2.438	0.988	28.350	NA	4.96	m^3/s
57.25	-1.5	0.000	2.25	1.101	1.101	1.3	0.151	0.151	0.95	1.889	2.040	0.85	4.796	1.101	0.950	2.538	1.008	28.450	NA	5.80	m^3/s
57.35	-1.4	0.000	2.35	1.125	1.125	1.4	0.169	0.169	1.05	2.195	2.364	0.95	5.667	1.125	0.956	2.638	1.028	28.550	NA	6.69	m^3/s
57.45	-1.3	0.000	2.45	1.149	1.149	1.5	0.187	0.187	1.15	2.516	2.703	1.05	6.585	1.149	0.962	2.738	1.047	28.650	NA	7.63	m^3/s
57.55	-1.2	0.000	2.55	1.172	1.172	1.6	0.206	0.206	1.25	2.851	3.057	1.15	7.547	1.172	0.966	2.838	1.066	28.750	NA	8.61	m^3/s
57.65	-1.1	0.000	2.65	1.195	1.195	1.7	0.226	0.226	1.35	3.200	3.426	1.25	8.553	1.195	0.969	2.938	1.085	28.850	NA	9.64	m^3/s
57.75	-1	0.000	2.75	1.217	1.217	1.8	0.246	0.246	1.45	3.562	3.808	1.35	9.600	1.217	0.971	3.038	1.103	28.950	NA	10.70	m^3/s
57.85	-0.9	0.000	2.85	1.239	1.239	1.9	0.267	0.267	1.55	3.937	4.204	1.45	10.686	1.239	0.972	3.138	1.121	29.050	NA	11.81	m^3/s
57.95	-0.8	0.000	2.95	1.261	1.261	2	0.288	0.288	1.65	4.324	4.612	1.55	11.810	1.261	0.972	3.238	1.139	29.150	NA	12.95	m^3/s
58.05	-0.7	0.000	3.05	1.282	1.282	2.1	0.310	0.310	1.75	4.723	5.033	1.65	12.971	1.282	0.972	3.338	1.156	29.250	NA	14.13	m^3/s
58.15	-0.6	0.000	3.15	1.303	1.303	2.2	0.333	0.333	1.85	5.133	5.466	1.75	14.168	1.303	0.970	3.438	1.173	29.350	NA	15.34	m^3/s
58.25	-0.5	0.000	3.25	1.323	1.323	2.3	0.356	0.356	1.95	5.555	5.911	1.85	15.400	1.323	0.968	3.538	1.190	29.450	NA	16.59	m^3/s
58.35	**-0.4**	**0.000**	**3.35**	**1.344**	**1.344**	**2.4**	0.379	**0.379**	**2.05**	**5.988**	**6.367**	1.95	**16.665**	**1.344**	**0.964**	**3.638**	**1.207**	**29.550**	**NA**	**17.87**	**m^3/s**

D/S Cell:

Outlet Arrangement adopted for concept H-S-Q:

- Twin chamber outlet pit for EDD control
- Controlling weir on U/S end of 1% AEP culverts into RB
- H-S relationship determined from SMEC 3D design tin.

Refer to Figure 11 for indicative layout, and below spreadsheet for H-Q computations

It is suitable for the controlling outlets to be refined at detail design stage.

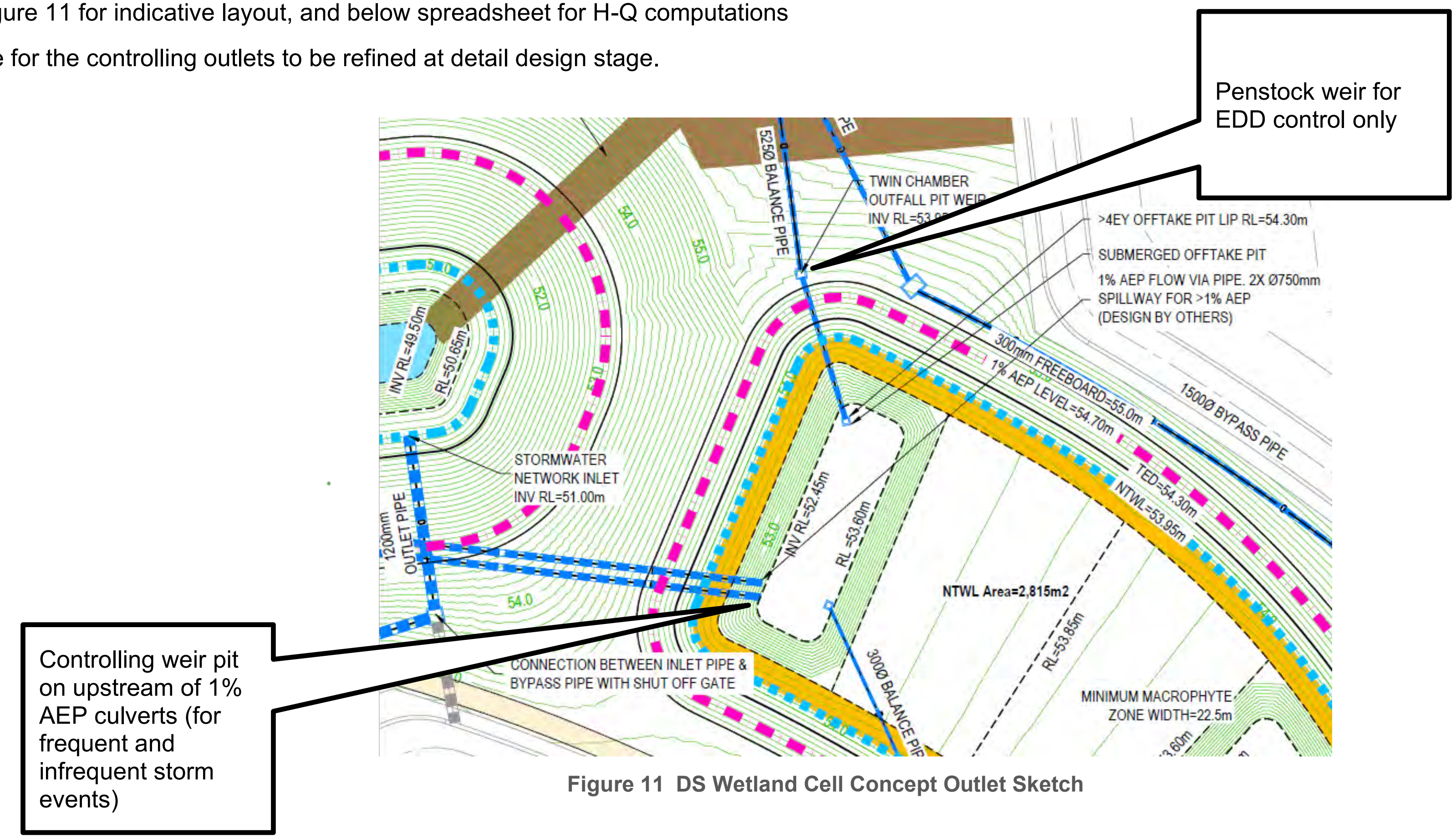

Figure 11 DS Wetland Cell Concept Outlet Sketch

Ignore the pit?? Yes/No		No	Ignore the pit?? Yes/No		Yes
PIT			PIT		
	Pit Sill Level	58.75 m AHD		Pit Sill Level	53 m AHD
	Width	0 m		Width	0.9 m
	Length	0 m		Length	0.9 m
	Area	0 m^2		Area	0.81 m^2
	portion not taken by b	62 %		portion not taken by b	62 %
	Grill blockage factor	50 %		Grill blockage factor	50 %
	All Weir blockage factc	0 %		Weir blockage factor	0 %

THIS PIT IS NOT USED) High Flow outlet pit - U/S | **WL Offtake/Outlet Pit**

PenStock Weir

0	m h	
53.95	m AHD	
0.095		
0.135	m	
Note - It is assumed the twin chamber outlet pit will only be set for EDD control - i.e. the pit baffel height is greater than Q100		0.024
		0.024
EDD Weir blockage	50	

Spillway

0.05	m h
54.35	m AHD
4.2	m

Note - It is assumed a pit will be on U/S end of Q100 culvert - and the pit lip will be controlling capacity. And therefore the D/S

Are you Assuming INLET CONTROL??? Yes/No				Yes	
PIPE	INLET CONTROL		OUTLET CONTROL		
Invert Level	52.45	m AHD	Ke	0.2	Side or slope tapered inlet
Dia	0.525	m	Kex	1.0	
barrels	1		Length	35.0	m
Area	0.216475369	m^2	Tailwater Level	28.800	m AHD
			wetted perim	1.65	m
			Hyd Radius	0.13	m

Height	Head	Orifice Flow	Head	Orifice Flow	total flow into U/S twin chamber pit	Head	Weir Flow 1	EDD Control Flow	total weir flow including EDD weir	Head	Weir Flow 3	total flow into twin chamber pit D/S chamber of Baffel/weir	total weir flow excuding EDD weir and excluding emergency spill	Head	Orifice Flow	head loss	flow	TOTAL FLOW RATE	
53.95	-4.8	0.000	0.95	0.715	0.715	0	0.000	0.000	0.000	0	0.000	0.000	0.000	1.238	0.704	25.150	NA	0.00	m^3/s
54.05	-4.7	0.000	1.05	0.752	0.752	0.1	0.004	0.004	0.004	0	0.000	0.004	0.000	1.338	0.732	25.250	NA	0.00	m^3/s
54.15	-4.6	0.000	1.15	0.787	0.787	0.2	0.010	0.010	0.010	0	0.000	0.010	0.000	1.438	0.759	25.350	NA	0.01	m^3/s
54.25	-4.5	0.000	1.25	0.821	0.821	0.3	0.019	0.019	0.019	0	0.000	0.019	0.000	1.538	0.785	25.450	NA	0.02	m^3/s
54.35	-4.4	0.000	1.35	0.853	0.853	0.4	0.029	0.029	0.029	7.10543E-15	0.000	0.029	0.000	1.638	0.810	25.550	NA	0.03	m^3/s
54.45	-4.3	0.000	1.45	0.884	0.884	0.5	0.041	0.041	0.041	0.1	0.226	0.041	0.000	1.738	0.834	25.650	NA	0.27	m^3/s
54.55	-4.2	0.000	1.55	0.914	0.914	0.6	0.053	0.053	0.053	0.2	0.639	0.053	0.000	1.838	0.858	25.750	NA	0.69	m^3/s
54.65	-4.1	0.000	1.65	0.943	0.943	0.7	0.067	0.067	0.067	0.3	1.173	0.067	0.000	1.938	0.881	25.850	NA	1.24	m^3/s
54.75	-4	0.000	1.75	0.971	0.971	0.8	0.082	0.082	0.082	0.4	1.806	0.082	0.000	2.038	0.903	25.950	NA	1.89	m^3/s
54.85	-3.9	0.000	1.85	0.998	0.998	0.9	0.098	0.098	0.098	0.5	2.524	0.098	0.000	2.138	0.925	26.050	NA	2.62	m^3/s
54.95	-3.8	0.000	1.95	1.025	1.025	1	0.115	0.115	0.115	0.6	3.318	0.115	0.000	2.238	0.947	26.150	NA	3.43	m^3/s
55.05	-3.7	0.000	2.05	1.051	1.051	1.1	0.132	0.132	0.132	0.7	4.182	0.132	0.000	2.338	0.968	26.250	NA	4.31	m^3/s
55.15	-3.6	0.000	2.15	1.076	1.076	1.2	0.151	0.151	0.151	0.8	5.109	0.151	0.000	2.438	0.988	26.350	NA	5.26	m^3/s
55.25	-3.5	0.000	2.25	1.101	1.101	1.3	0.170	0.170	0.170	0.9	6.096	0.170	0.000	2.538	1.008	26.450	NA	6.27	m^3/s
55.35	-3.4	0.000	2.35	1.125	1.125	1.4	0.190	0.190	0.190	1	7.140	0.190	0.000	2.638	1.028	26.550	NA	7.33	m^3/s
55.45	-3.3	0.000	2.45	1.149	1.149	1.5	0.211	0.211	0.211	1.1	8.237	0.211	0.000	2.738	1.047	26.650	NA	8.45	m^3/s
55.55	-3.2	0.000	2.55	1.172	1.172	1.6	0.232	0.232	0.232	1.2	9.386	0.232	0.000	2.838	1.066	26.750	NA	9.62	m^3/s
55.65	-3.1	0.000	2.65	1.195	1.195	1.7	0.254	0.254	0.254	1.3	10.583	0.254	0.000	2.938	1.085	26.850	NA	10.84	m^3/s
55.75	-3	0.000	2.75	1.217	1.217	1.8	0.277	0.277	0.277	1.4	11.827	0.277	0.000	3.038	1.103	26.950	NA	12.10	m^3/s
55.85	-2.9	0.000	2.85	1.239	1.239	1.9	0.301	0.301	0.301	1.5	13.117	0.301	0.000	3.138	1.121	27.050	NA	13.42	m^3/s
55.95	-2.8	0.000	2.95	1.261	1.261	2	0.325	0.325	0.325	1.6	14.450	0.325	0.000	3.238	1.139	27.150	NA	14.77	m^3/s
56.05	-2.7	0.000	3.05	1.282	1.282	2.1	0.349	0.349	0.349	1.7	15.826	0.349	0.000	3.338	1.156	27.250	NA	16.18	m^3/s
56.15	-2.6	0.000	3.15	1.303	1.303	2.2	0.374	0.374	0.374	1.8	17.243	0.374	0.000	3.438	1.173	27.350	NA	17.62	m^3/s
56.25	-2.5	0.000	3.25	1.323	1.323	2.3	0.400	0.400	0.400	1.9	18.699	0.400	0.000	3.538	1.190	27.450	NA	19.10	m^3/s
56.35	-2.4	0.000	3.35	1.344	1.344	2.4	0.427	0.427	0.427	2	20.195	0.427	0.000	3.638	1.207	27.550	NA	20.62	m^3/s

RB/WL Cell

Outlet Arrangement adopted for concept H-S-Q:

- Twin chamber outlet pit for EDD control and frequent storm weir outlet
- Controlling weir pit on U/S end of 1% culvert outlet
- H-S relationship determined from SMEC 3D design tin.

Refer to Figure 12 for indicative layout, and below spreadsheet for H-Q computations.

It is suitable for the controlling outlets to be refined at detail design stage.

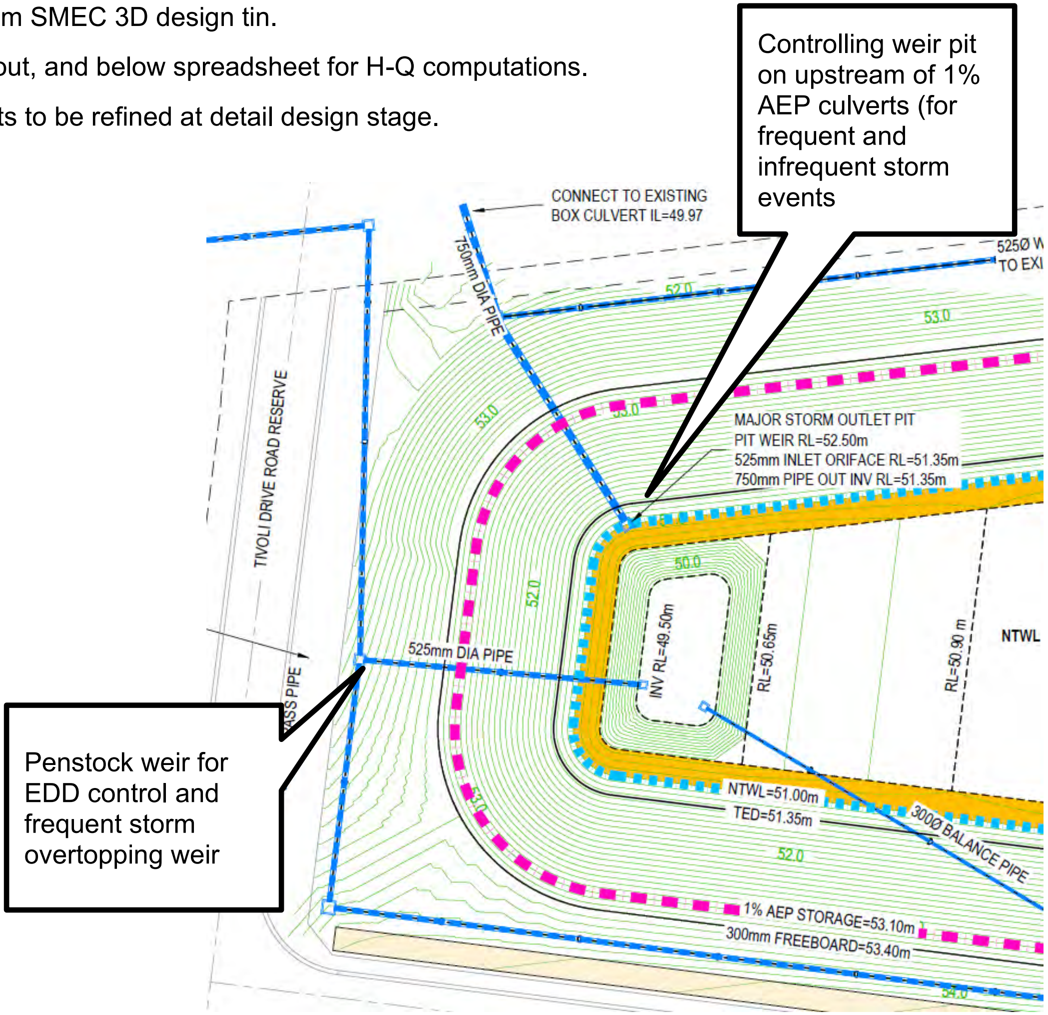

Figure 12 Mid Wetland Cell Concept Outlet Sketch

High Flow outlet pit - U/S			WL Offtake/Outlet Pit		
Ignore the pit?? Yes/No	No		Ignore the pit?? Yes/No	Yes	
PIT			PIT		
Pit Sill Level	58.75	m AHD	Pit Sill Level	50	m AHD
Width	0	m	Width	0.9	m
Length	0	m	Length	0.9	m
Area	0	m^2	Area	0.81	m^2
portion not taken by ba	62	%	portion not taken by ba	62	%
Grill blockage factor	50	%	Grill blockage factor	50	%
All Weir blockage facto	0	%	Weir blockage factor	0	%

THIS PIT IS NOT USED) High Flow outlet pit - U/S

PenStock Weir			Weir Over Pen stock			Spillway	
0	m h		0.35	m h		1	m h
51	m AHD		51.35	m AHD		52.35	m AHD
0.0650	m		0.5	m		0.65	m
0.065							
	0.016						
	0.012		full RB level ->	1.87			
EDD Weir blockage	50		Lesser RB level->	1.20			

Note - It is assumed a pit will be on U/S end of Q100 culvert - and the pit lip will be controlling capacity. And therefore the D/S

	53.1		
	0.75		
S(m3)	4564.747	5719.891	9688.498
Q	0.50	0.74	1.44
Stage(mAHD)	52	52.2	52.8

Are you Assuming INLET CONTROL??? Yes/No				Yes	
PIPE	INLET CONTROL		OUTLET CONTROL		
Invert Level	49.5	m AHD	Ke	0.2	Side or slope tapered inlet
Dia	0.525	m	Kex	1.0	
barrels	1		Length	35.0	m
Area	0.216475369	m^2	Tailwater Level	28.800	m AHD
			wetted perim	1.65	m
			Hyd Radius	0.13	m

Height	Head	Orifice Flow	Head	Orifice Flow	total flow into U/S twin chamber pit	Head	Weir Flow 1	EDD Control Flow	Head	Weir Flow 2	total weir flow including EDD weir	Head	Weir Flow 3	total flow into twin chamber pit D/S chamber of Baffel/weir	total weir flow excuding EDD weir and excluding emergency spill	Head	Orifice Flow	head loss	flow	TOTAL FLOW RATE	
51	-7.75	0.000	1	0.734	0.734	0	0.000	0.000	0	0.000	0.000	0	0.000	0.000	0.000	1.238	0.704	22.200	NA	0.00	m^3/s
51.1	-7.65	0.000	1.1	0.770	0.770	0.1	0.002	0.002	0	0.000	0.002	0	0.000	0.002	0.000	1.338	0.732	22.300	NA	0.00	m^3/s
51.2	-7.55	0.000	1.2	0.804	0.804	0.2	0.005	0.005	0	0.000	0.005	0	0.000	0.005	0.000	1.438	0.759	22.400	NA	0.00	m^3/s
51.3	-7.45	0.000	1.3	0.837	0.837	0.3	0.009	0.009	0	0.000	0.009	0	0.000	0.009	0.000	1.538	0.785	22.500	NA	0.01	m^3/s
51.4	-7.35	0.000	1.4	0.869	0.869	0.4	0.014	0.014	0.05	0.010	0.023	0	0.000	0.023	0.010	1.638	0.810	22.600	NA	0.02	m^3/s
51.5	-7.25	0.000	1.5	0.899	0.899	0.5	0.020	0.020	0.15	0.049	0.069	0	0.000	0.069	0.049	1.738	0.834	22.700	NA	0.07	m^3/s
51.6	-7.15	0.000	1.6	0.929	0.929	0.6	0.026	0.026	0.25	0.106	0.132	0	0.000	0.132	0.106	1.838	0.858	22.800	NA	0.13	m^3/s
51.7	-7.05	0.000	1.7	0.957	0.957	0.7	0.032	0.032	0.35	0.176	0.208	0	0.000	0.208	0.176	1.938	0.881	22.900	NA	0.21	m^3/s
51.8	-6.95	0.000	1.8	0.985	0.985	0.8	0.040	0.040	0.45	0.257	0.296	0	0.000	0.296	0.257	2.038	0.903	23.000	NA	0.30	m^3/s
51.9	-6.85	0.000	1.9	1.012	1.012	0.9	0.047	0.047	0.55	0.347	0.394	0	0.000	0.394	0.347	2.138	0.925	23.100	NA	0.39	m^3/s
52	-6.75	0.000	2	1.038	1.038	1	0.055	0.055	0.65	0.445	0.501	0	0.000	0.501	0.445	2.238	0.947	23.200	NA	0.50	m^3/s
52.1	-6.65	0.000	2.1	1.064	1.064	1.1	0.064	0.064	0.75	0.552	0.616	0	0.000	0.616	0.552	2.338	0.968	23.300	NA	0.62	m^3/s
52.2	-6.55	0.000	2.2	1.089	1.089	1.2	0.073	0.073	0.85	0.666	0.739	0	0.000	0.739	0.666	2.438	0.988	23.400	NA	0.74	m^3/s
52.3	-6.45	0.000	2.3	1.113	1.113	1.3	0.082	0.082	0.95	0.787	0.869	0	0.000	0.869	0.787	2.538	1.008	23.500	NA	0.87	m^3/s
52.4	-6.35	0.000	2.4	1.137	1.137	1.4	0.092	0.092	1.05	0.915	1.006	0.05	0.012	1.006	0.915	2.638	1.028	23.600	NA	1.02	m^3/s
52.5	-6.25	0.000	2.5	1.161	1.161	1.5	0.102	0.102	1.15	1.048	1.150	0.15	0.064	1.150	1.048	2.738	1.047	23.700	NA	1.11	m^3/s
52.6	-6.15	0.000	2.6	1.184	1.184	1.6	0.112	0.112	1.25	1.188	1.300	0.25	0.138	1.184	1.072	2.838	1.066	23.800	NA	1.20	m^3/s
52.7	-6.05	0.000	2.7	1.206	1.206	1.7	0.122	0.122	1.35	1.333	1.456	0.35	0.229	1.206	1.084	2.938	1.085	23.900	NA	1.31	m^3/s
52.8	-5.95	0.000	2.8	1.228	1.228	1.8	0.133	0.133	1.45	1.484	1.618	0.45	0.334	1.228	1.095	3.038	1.103	24.000	NA	1.44	m^3/s
52.9	-5.85	0.000	2.9	1.250	1.250	1.9	0.145	0.145	1.55	1.640	1.785	0.55	0.451	1.250	1.105	3.138	1.121	24.100	NA	1.57	m^3/s
53	-5.75	0.000	3	1.271	1.271	2	0.156	0.156	1.65	1.802	1.958	0.65	0.579	1.271	1.115	3.238	1.139	24.200	NA	1.72	m^3/s
53.1	-5.65	0.000	3.1	1.292	1.292	2.1	0.168	0.168	1.75	1.968	2.136	0.75	0.718	1.292	1.124	3.338	1.156	24.300	NA	1.87	m^3/s
53.2	-5.55	0.000	3.2	1.313	1.313	2.2	0.180	0.180	1.85	2.139	2.319	0.85	0.866	1.313	1.133	3.438	1.173	24.400	NA	2.04	m^3/s
53.3	-5.45	0.000	3.3	1.334	1.334	2.3	0.193	0.193	1.95	2.315	2.507	0.95	1.023	1.334	1.141	3.538	1.190	24.500	NA	2.21	m^3/s
53.4	**-5.35**	**0.000**	**3.4**	**1.354**	**1.354**	**2.4**	0.205	**0.205**	**2.05**	**2.495**	**2.700**	1.05	**1.189**	**1.354**	**1.148**	**3.638**	**1.207**	**24.600**	**NA**	**2.40**	m^3/s
53.5	-5.25	0.000	3.5	1.373	1.373	2.5	0.218	0.218	2.15	2.680	2.898	1.15	1.363	1.373	1.155	3.738	1.223	24.700	NA	2.59	m^3/s
53.6	-5.15	0.000	3.6	1.393	1.393	2.6	0.232	0.232	2.25	2.869	3.100	1.25	1.544	1.393	1.161	3.838	1.240	24.800	NA	2.78	m^3/s
53.7	-5.05	0.000	3.7	1.412	1.412	2.7	0.245	0.245	2.35	3.062	3.307	1.35	1.733	1.412	1.167	3.938	1.256	24.900	NA	2.99	m^3/s
53.8	**-4.95**	**0.000**	**3.8**	**1.431**	**1.431**	**2.8**	0.259	**0.259**	**2.45**	**3.260**	**3.518**	1.45	**1.929**	**1.431**	**1.172**	**4.038**	**1.272**	**25.000**	**NA**	**3.20**	m^3/s
53.9	-4.85	0.000	3.9	1.450	1.450	2.9	0.273	0.273	2.55	3.461	3.734	1.55	2.132	1.450	1.177	4.138	1.287	25.100	NA	3.42	m^3/s
54	-4.75	0.000	4	1.468	1.468	3	0.287	0.287	2.65	3.667	3.954	1.65	2.342	1.468	1.181	4.238	1.303	25.200	NA	3.64	m^3/s
54.1	-4.65	0.000	4.1	1.486	1.486	3.1	0.302	0.302	2.75	3.876	4.178	1.75	2.558	1.486	1.185	4.338	1.318	25.300	NA	3.88	m^3/s
54.2	-4.55	0.000	4.2	1.504	1.504	3.2	0.316	0.316	2.85	4.090	4.406	1.85	2.780	1.504	1.188	4.438	1.333	25.400	NA	4.11	m^3/s
54.3	-4.45	0.000	4.3	1.522	1.522	3.3	0.331	0.331	2.95	4.307	4.638	1.95	3.009	1.522	1.191	4.538	1.348	25.500	NA	4.36	m^3/s
54.4	**-4.35**	**0.000**	**4.4**	**1.540**	**1.540**	**3.4**	0.346	**0.346**	**3.05**	**4.528**	**4.874**	2.05	**3.243**	**1.540**	**1.193**	**4.638**	**1.363**	**25.600**	**NA**	**4.61**	m^3/s
54.5	-4.25	0.000	4.5	1.557	1.557	3.5	0.362	0.362	3.15	4.752	5.114	2.15	3.484	1.557	1.195	4.738	1.377	25.700	NA	4.86	m^3/s
54.6	-4.15	0.000	4.6	1.574	1.574	3.6	0.377	0.377	3.25	4.980	5.358	2.25	3.729	1.574	1.197	4.838	1.392	25.800	NA	5.12	m^3/s
54.7	-4.05	0.000	4.7	1.591	1.591	3.7	0.393	0.393	3.35	5.212	5.605	2.35	3.981	1.591	1.198	4.938	1.406	25.900	NA	5.39	m^3/s
54.8	-3.95	0.000	4.8	1.608	1.608	3.8	0.409	0.409	3.45	5.447	5.856	2.45	4.238	1.608	1.199	5.038	1.420	26.000	NA	5.66	m^3/s
54.9	-3.85	0.000	4.9	1.625	1.625	3.9	0.426	0.426	3.55	5.685	6.111	2.55	4.500	1.625	1.199	5.138	1.434	26.100	NA	5.93	m^3/s
55	-3.75	0.000	5	1.641	1.641	4	0.442	0.442	3.65	5.927	6.369	2.65	4.767	1.641	1.199	5.238	1.448	26.200	NA	6.22	m^3/s

Melbourne

15 Business Park Drive
Notting Hill VIC 3168
Telephone (03) 8526 0800
Fax (03) 9558 9365

Brisbane

Level 5, 43 Peel Street
South Brisbane QLD 4101
Telephone (07) 3105 1460
Fax (07) 3846 5144

Adelaide

1/198 Greenhill Road
Eastwood SA 5063
Telephone (08) 8378 8000
Fax (08) 8357 8988

Perth

Ground Floor
430 Roberts Road
Subiaco WA 6008
Telephone 08 6555 0105

Geelong

PO Box 436
Geelong VIC 3220
Telephone 0458 015 664

Gippsland

154 Macleod Street
Bairnsdale VIC 3875
Telephone (03) 5152 5833

Wangaratta

First Floor, 40 Rowan Street
Wangaratta VIC 3677
Telephone (03) 5721 2650

Wimmera

PO Box 584
Stawell VIC 3380
Telephone 0438 510 240

www.watertech.com.au

info@watertech.com.au