WATER TECHNOLOGY
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Wetland Functional Design Report

Central Road Drysdale Wetlands

St Quentin Consulting

July 2018

Document Status

Version	Doc type	Reviewed by	Approved by	Date issued
01	Draft Report	Aaron Vendargon	Aaron Vendargon	05/04/2018
02	Final Report	Bertrand Salmi	Aaron Vendargon	02/05/2018
03	Final Report	Bertrand Salmi	Aaron Vendargon	18/05/2018
04	Final Report	Aaron Vendargon	Thomas Cousland	23/07/2018

Project Details

Project Name	Central Road Drysdale Wetlands
Client	St Quentin Consulting
Client Project Manager	Chris Manson
Water Technology Project Manager	Julian Skipworth
Water Technology Project Director	Ben Tate
Authors	Cintia Dotto
Document Number	4663-01_R01v02_Wetland_FD

ST. QUENTIN
Surveyors • Town Planners • Engineers

15 Business Park Drive
Notting Hill VIC 3168
Telephone (03) 8526 0800
Fax (03) 9558 9365
ACN 093 377 283
ABN 60 093 377 283

Certified System Quality ISO 9001 SAI GLOBAL
Certified System Environment ISO 14001 SAI GLOBAL
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CONTENTS

APPENDICES

LIST OF FIGURES

LIST OF TABLES

1 INTRODUCTION

This report describes the functional design of two combined sediment basin/wetland/retarding basin systems (WL/RB) that forms the Stormwater Management Plan (SWMP) for a proposed development at Central Road, Drysdale. The development proposes the sub-division of the site for private sale and residential development.

The proposed systems are designed to achieve the following objectives:

- Ensure the development attenuates post-development runoff to pre-development levels;
- Achieve best practice water quality targets; and
- Alleviate flood impacts of the development on downstream environment.

This Functional Design report demonstrates that the proposed wetlands and retarding basins for the site are founded on best practice principles and meet Best Practice Guidelines for water quality treatment. The design has been based on the current Melbourne Water Guidelines[1] for constructed wetlands. While complying with most of the items, there were few non-compliance due to site specific conditions.

Calculations used in the functional design are included in Appendix A.

[1] Melbourne Water (2015). Melbourne Water, Design, Construction and Establishment of Constructed Wetlands: Design Manual, updated 2017.

2 BACKGROUND

The subject site consists of 27 parcels of land located within Drysdale, as shown in Figure 2-1. The subject site is located east of Jetty Road, Drysdale and includes Central Road and Thomas Street. Wyndham Street is located immediately south of the site and Ada Street immediately north. The land is currently zoned Rural Living Zone (RLZ) and has been identified for future residential development.

Current flood mapping available from the City of Greater Geelong (CoGG) has identified the site as being subject to flooding from Griggs Creek and its tributaries. The Corangamite Catchment Management Authority (CCMA) has also provided information which indicates that two designated waterways pass through this location (but not through the site).

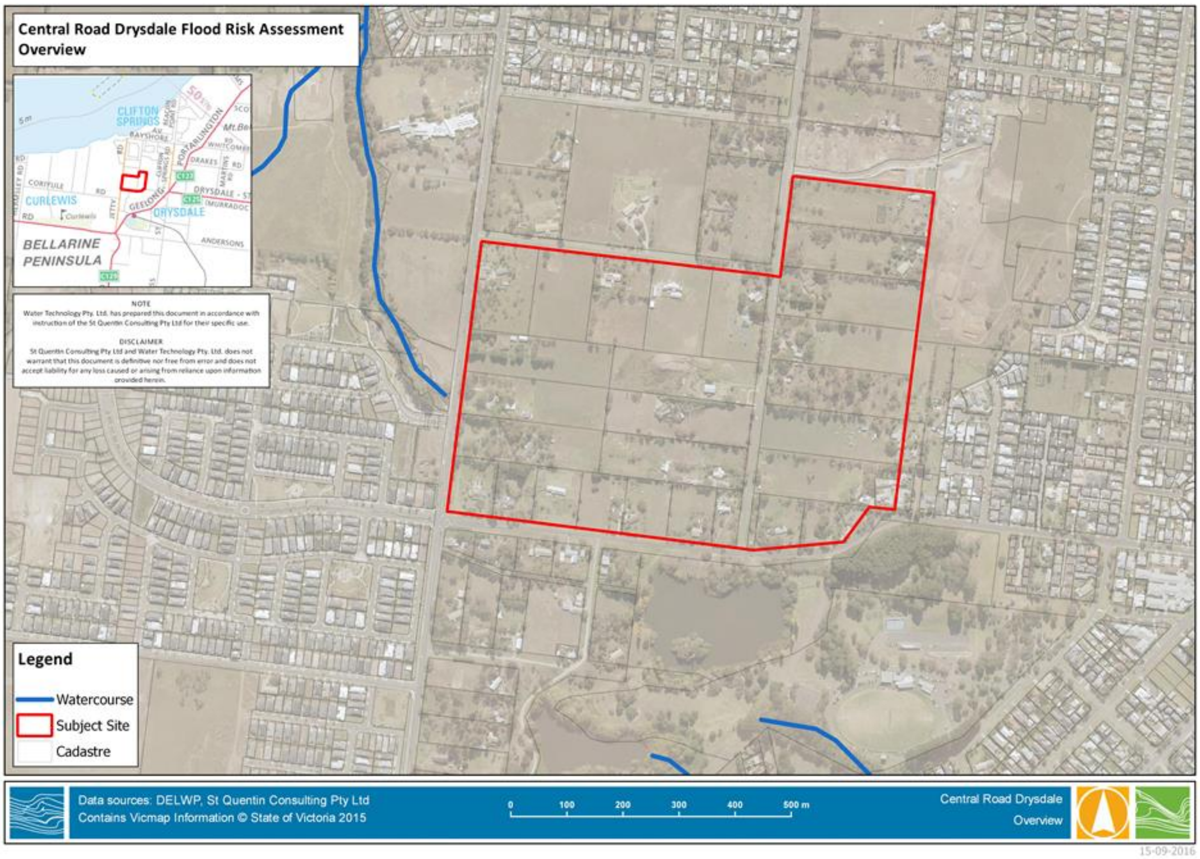

Figure 2-1 Subject Site

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2.1 Existing Flood Risk

Hydrological (RORB) and hydraulic modelling (TUFLOW) were previously undertaken to assess the existing flood risk at the site (Water Technology, September 2016). This modelling allowed for catchment delineation and estimated fraction imperviousness, based on existing planning zones and planned development.

The baseline TUFLOW model was created using LiDAR topography and survey collected in 2007. The model build was guided by current best practice approaches and standards, including the Infrastructure Design Manual.

Additional detail of the methodology and assumptions used to model existing flood risk can be found in Water Technology's report dated September 2016.

The modelling undertaken shows the site to be at risk of flooding during the 1% Annual Exceedance Probability (AEP), as shown in Figure 2-2.

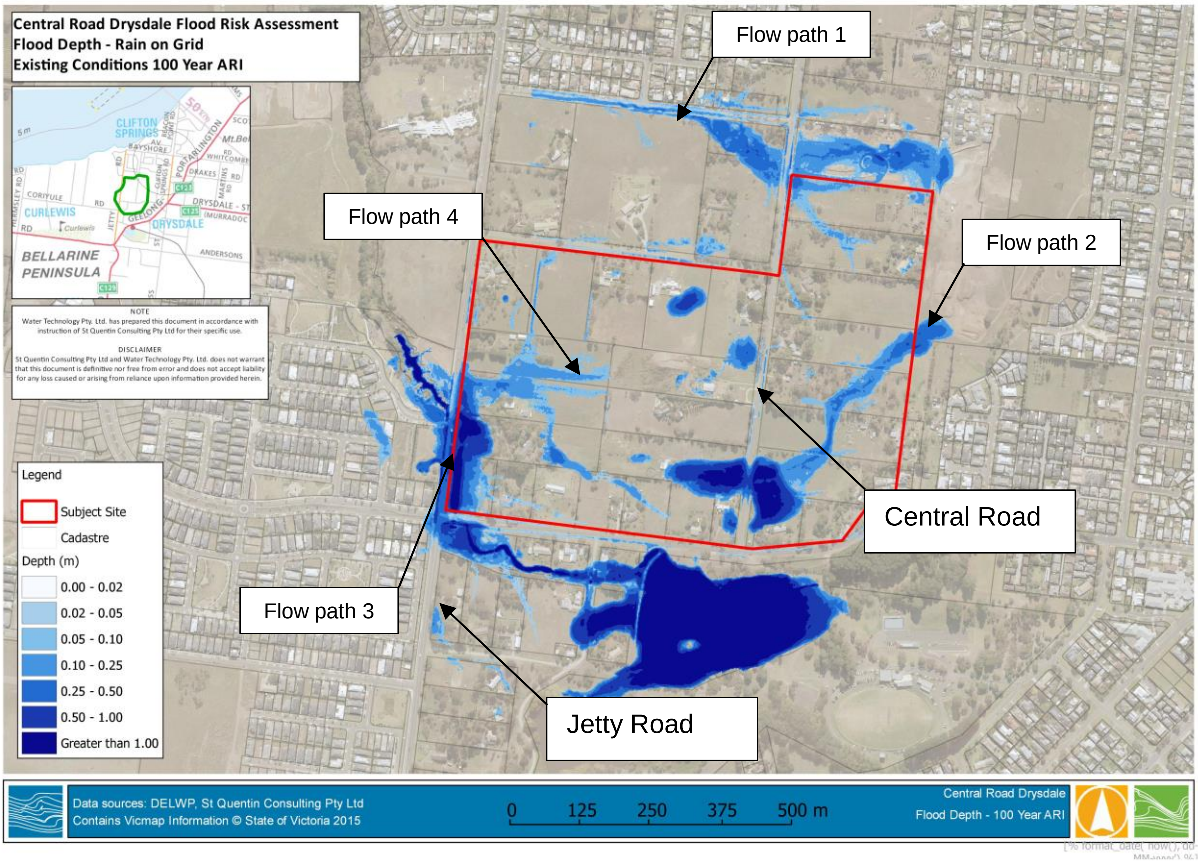

Figure 2-2 Preliminary Flood Modelling Results Extent (1%AEP)

Four overland flow paths were identified in the catchment within or around the site. Three of the flow paths (flow path 1 to 3) are associated with external catchments, while one appears to be the result of local run off (flow path 4). Two regions of significant ponding were also identified within the site, including:

- Central Road (150 m north of Wyndham Street) - Flooding as a result of water pooling is on both sides of the road and (excluding the farm dam) shows low areas inundated to depths greater than 1.5 m; and

- Jetty Road (From Wyndham Street to the Griggs Creek Crossing under Jetty Road) - Most flooding in this area is found on the east side of the road (inside the study area) and is greater than 1.2 m deep in channelized sections of the flow path.

2.2 Designated Waterways

A review of the current state government watercourse layers, planning overlays, imagery and LiDAR suggests that there are no obvious watercourses within the subject site, and discussions were held with the CCMA to confirm this. The outcomes of these discussions were considered as part of this study to ensure that any detrimental impact from the development of the site are mitigated.

2.3 Site constraints and conditions

Based on the results of the preliminary flood modelling and subsequent discussions, it was agreed with the CCMA that:

- There are no designated waterways located on the site, however, the development layout must maintain conveyance capacity of Flow path 3;
- The proposed development must not result in afflux for a range of events up to and including the 1% AEP event;
- The proposed development layout ensures there is no net loss of flood storage within the site; and
- No new lots can be created within the post developed mapped 1% AEP flood extent.

The above items have been considered as part of the Stormwater Management Strategy and additional flood modelling was undertaking to ensure that the concept design considered the listed conditions.

3 PREVIOUS REPORTS AND RELEVANT UPDATES

The hydrology and hydraulic modelling that determined the size of the two retarding basins and wetlands were conducted and reported by Water Technology in December 2017. The 2017 report also included the water quality analysis conducted in MUSIC and the sizing of the two wetlands. A summary of the report is included in this section. This summary also covers the updated modelling where the shapes of the two wetlands were amended to match the 100 year ARI flood extent. It also includes the updates to the basin configurations undertaken as part of the functional design.

3.1 Hydrological Analysis

The hydrological modelling was based on a previous RORB model (Water Technology, 2016) and updated where necessary. The model estimated the rainfall runoff from the external catchments that impact the study area. It allowed for an assessment of the overland flows within the subject site.

3.1.1 Retarding Basin Sizing

The retarding basins were sized (in RORB) to ensure that the peak post-development flow rates do not exceed the pre-development discharges for the 100 year ARI event.

A flow comparison is presented in Table 3-1 below, showing that the post-development 100 year mitigated flows are successfully retarded back to pre-development flow rates.

Table 3-1 100 Year ARI Existing and Developed Retarded Flows (Critical Storm Duration)

Location	Existing Flows (m^3/s)	Mitigated Developed Flows (m^3/s)
Upstream Central Road	0.96 (2 hr)	0.61 (1 hr)
Upstream Jetty Road	2.02 (1 hr)	1.91 (1 hr)

The configurations of the two retarding basins are shown in Table 3-2. The outlet pipes for the basins are connected into the twin chamber outfall pit at the downstream end (downstream of the wetland control penstock in this pit). The inverts of the basin outlet pipes are set at the wetland NWL.

Table 3-2 Retarding Basin Details

Retarding Basin Details	Upstream Central Road	Upstream Jetty Road
Storage Volume (m^3)	1,610	4,820
Critical Duration (in the 100 year ARI)	1 hr	1 hr
Peak 100 year ARI Outflow (m^3/s)	0.61	1.91
No. of Pipes (primary outlet)	1	1
Pipe IL (primary outlet) (m AHD)	46.0	39.5
Pipe Diameter (mm)	525	900
Spillway Length (m)	25	25
Spillway Elevation (m AHD)	47.20	40.9

An indicative layout of these retarding basins is shown in Figure 3-1.

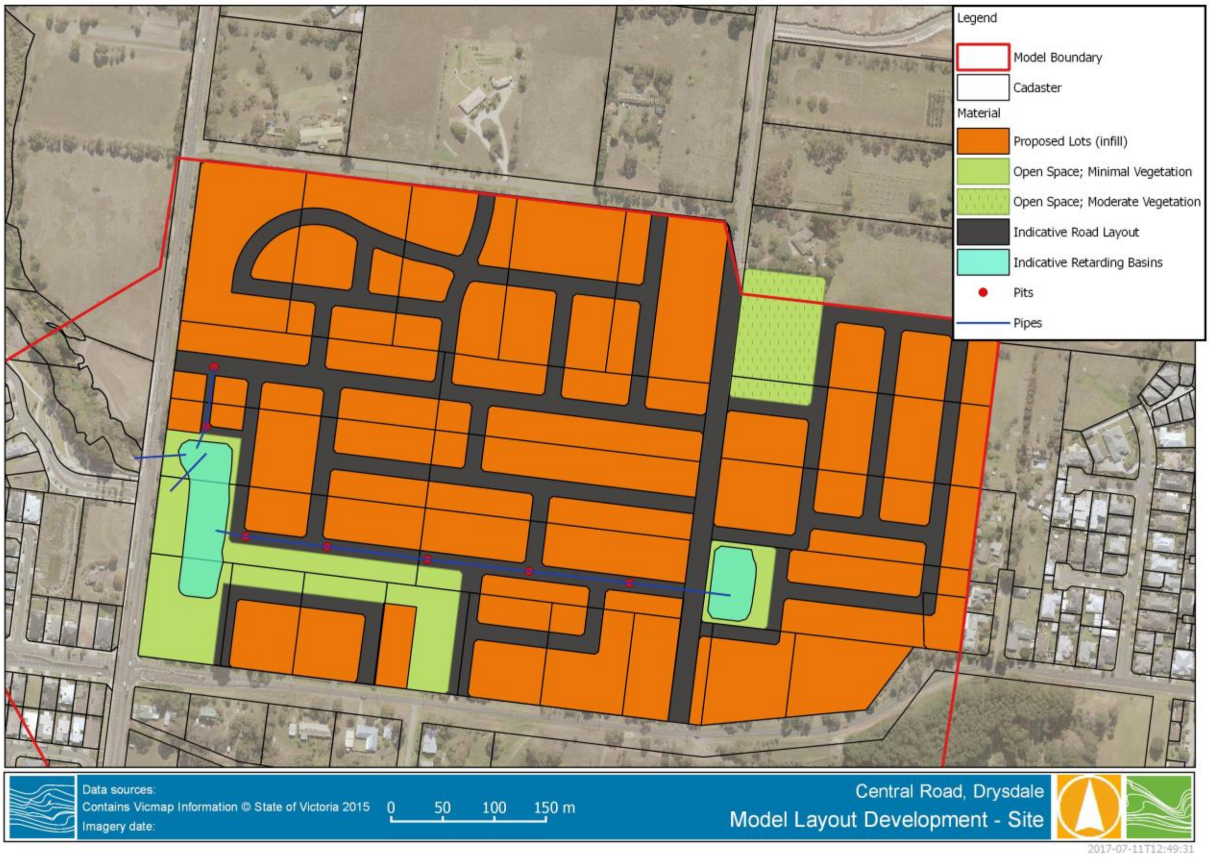

Figure 3-1 Indicative Retarding Basin Layouts and Locations

3.2 Flood Modelling

A broad-scale TUFLOW model of the entire study area was developed. The TUFLOW applies rainfall (1% Annual Exceedance Probability) and then routes flows through the catchment to assess overland flow paths during extreme rainfall events. The aim of the model was to assess the performance of the two proposed retarding basins. The extent of the model was delineated to ensure all runoff from the subject site was conveyed to the proposed retarding basins prior to discharge to the environment.

A development scenario model was built to assess the performance of the proposed retarding basins and was informed by a preliminary plan for the site, shown in Figure 3-2. The hydraulic modelling has demonstrated that the proposed retarding basins result in no adverse on-site impacts in the 1% AEP flood event and generally reduce flood risk compared to the existing conditions in respect to flow conveyance.

As agreed with Corangamite Catchment Management Authority, the proposed Stormwater Strategy shows that:

- The development layout maintains conveyance capacity of Flow path 3;
- The proposed development layout ensures there is no net loss of flood storage within the site; and
- No new lots are created within the post developed mapped 1% AEP flood extent.

Importantly, the downstream retarding basin (east of Jetty Road), is not located within the major flow path.

Whilst the proposed development results in localised afflux for the 1% AEP event (1 hour and 12 hour duration events), the proposed drainage system and retarding basin generally results in a significant reduction in flood depths along Flow path 3 and in an adjacent property. This afflux is limited to road infrastructure and the downstream waterway and does not impact on flood depths on downstream properties.

Based on the outcomes of this modelling, Water Technology concludes that the proposed development will not have any unacceptable impacts on flood safety. Additional details are provided in the Stormwater Management Plan (Water Technology, December 2017).

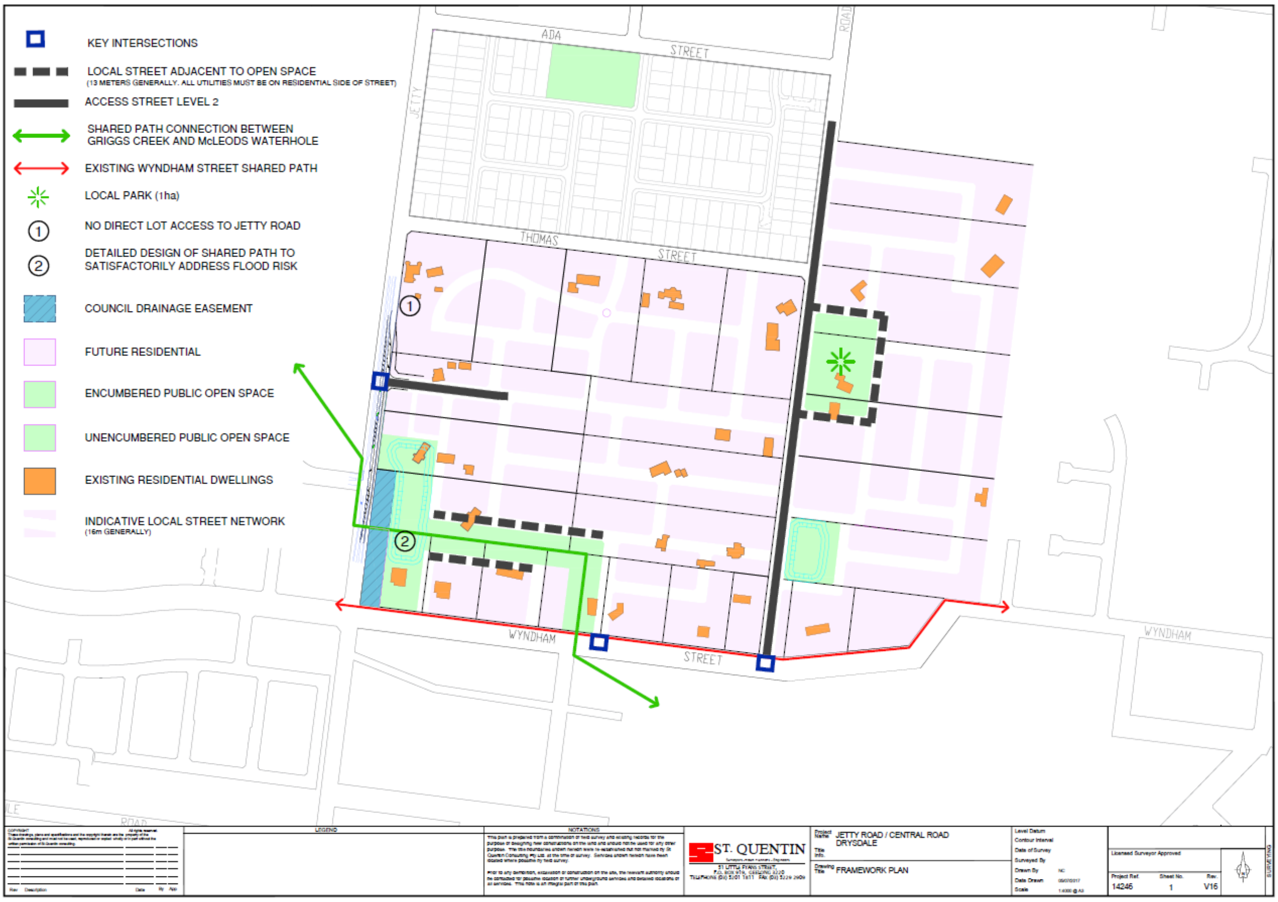

Figure 3-2 Draft Layout Plan (Source: St Quentin)

3.3 Water Quality Assets

Water quality analysis was conducted in MUSIC. Sizing of the proposed assets are shown in Table 3-3. High flow bypasses were also integrated in the model (above the 3-month ARI).

Table 3-3 Treatment Train Sizing

System	Location	Surface Area at the NWL (m^2)	Extended Detention Depth (m)	Permanent Pool Volume at the NWL (m^3)	High flow bypass (m^3/s)
Upstream Sedimentation Basin	Upstream Central Road	400	0.5	355	0.15
Upstream Wetland	Upstream Central Road	770	0.5	236	0.15
Downstream Sedimentation Basin	Upstream Jetty Road	748	0.5	609	0.295
Downstream Wetland	Upstream Jetty Road	2,211	0.5	790	0.295

Sedimentation ponds were designed for a notional detention time of approximately 12 hours, and the wetlands for 48 – 72 hours.

Any difference between the overall design values presented in the SWMP report and this report is due to the refinement of the design at this functional stage.

4 OVERALL LAYOUT

The functional design of the treatment trains and major drainage was based on the previous analyses presented in Section 3. The overall layout of the systems is shown in Figure 4-1. The downstream system is located by Jetty Road and the upstream system is located by Central Road as shown in Figure 4-1.

The digital files of the design (12d) and preliminary layout plans are delivered with this report as part of the functional design package. The preliminary layout plans together with this report package provide the design elements for St Quentin to produce the final Functional Drawings.

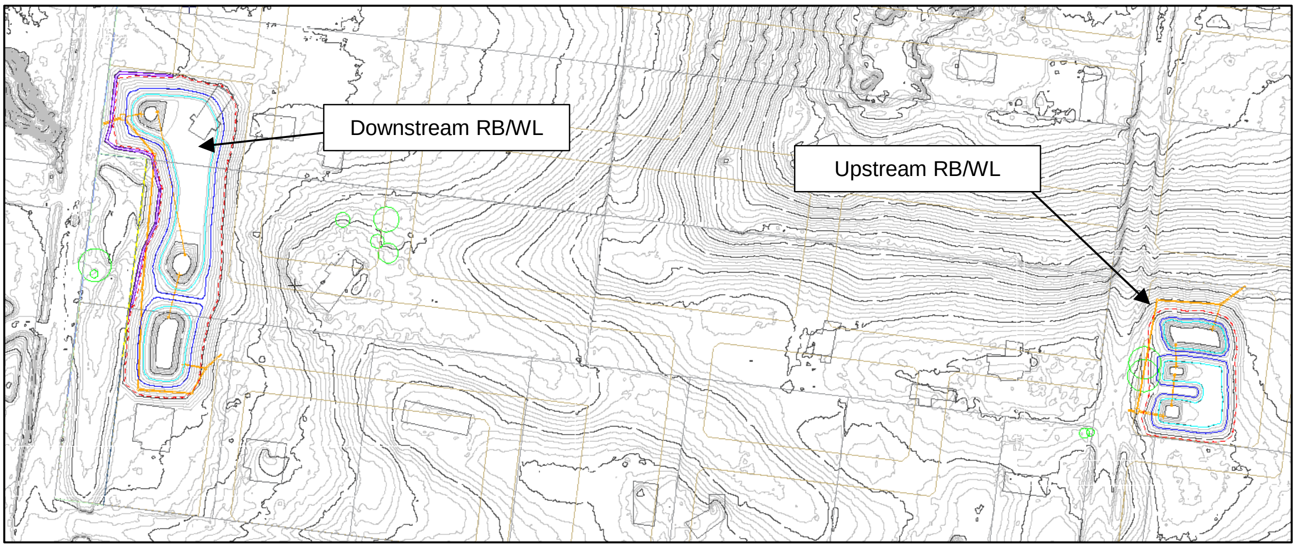

Figure 4-1 Overall Plan of the Proposed Stormwater Management Systems

4.1 Design Flow Rates

The design flow rates used in the design of the systems are shown in Table 4-1.

Table 4-1 Design Flow Rates

Storm Event	US Wetland Flow Rate (m^3/s)	DS Wetland Flow Rate (m^3/s)
3 month ARI inflow	0.15	0.30
1 year ARI Inflow	0.36	0.72
5 year ARI Inflow	0.75	1.49
100 year ARI Inflow	1.88	3.65

The current design arrangement has up to the 3 month ARI flow entering the sediment basin and macrophyte zone. It has been assumed that flows above the 3 month ARI and up to 5 year ARI will be conveyed via a pipe system and will bypass the wetland system. Flows above the 5 year ARI and up to 100 year ARI will enter the retarding basin as overland flows.

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5 UPSTREAM SYSTEM

5.1 Sediment Basin

Sediment basins have been sized via two methods, the Fair and Geyer formula (Equation 10.3 in WSUD Engineering Procedures, 2004) to determine minimum surface area required to ensure the settling velocity criteria for sediments (with a target of 95% removal rate for 125 µm particles), and a volume calculation to determine the accumulated sediment volume required within the Sediment Basin for a cleanout frequency of every 5 years.

The 3 month ARI flow rate of 0.15 m^3/s was used to determine the required basin via the Fair and Geyer formula, with the minimum area of **180 m^2** required. The accumulated volume calculation assumed a developed upstream catchment area of 9.6 ha and a sediment loading rate of 2.0 m^3/ha/year, which sized the required sediment storage volume to be **100 m^3**. It was found that the accumulated volume storage requirement was the critical factor in sizing of the sediment basin, not sediment settling velocities. It has been assumed that no GPT is located upstream of the sediment basin. The design of the sediment basin has an area of **400 m^2** and **185 m^3** of volume 500 mm below the NWL for accumulated sediment volume, and therefore is adequately sized for the upstream catchment.

An Extended Detention Depth of **500 mm** has been incorporated into the sediment basin.

Twin 225 mm diameter pipes have been sized to convey the 3 month ARI flow from the sediment basin to the wetland macrophyte zone. Whilst the pipe size is less than the normal Council minimum requirement (300 mm), it was found that a single 300 mm pipe would too small, dual 300 mm pipes would be too large and a single 375 mm pipe would also be too large, therefore twin 225 mm pipes were selected. The sediment basin outlet pit (**1.2 m by 1.2 m**) has been sized (is large enough) to ensure that the pipe outlet forms the hydraulic control. Blockage factors within the transfer pit have been factored into the calculation.

The 100 year velocity in the sediment basin has been calculated following the approach outlined in the Melbourne Water Guidelines[2]. The calculation (using the cross sectional area between NWL and the 10 year ARI level in the basin) indicates that the maximum 100 year velocity in the basin would be equal to 0.11 m/s, which is within the requirement of 0.5 m/s to prevent re-suspension of deposited sediment during a major storm event.

5.2 Wetland Macrophyte Zone

Table 5-1 shows a summary of the macrophyte zonation areas within the Upstream Wetland.

An Extended Detention Depth of 500 mm has been incorporated into the macrophyte zone. To determine the maximum 3 month velocities within the macrophyte zone, the calculation approach in the Melbourne Water Guidelines[1] has been adopted. The estimated 3 month flow velocity was 0.03 m/s which is within the requirement of 0.05 m/s. The estimated 100 year flow velocity was 0.08 m/s which is within the requirement of 0.5 m/s.

[2] Melbourne Water (2015). Melbourne Water, Design, Construction and Establishment of Constructed Wetlands: Design Manual, updated 2017.

Table 5-1 Macrophyte Zonation Details

	Depth below NWL (m)	Area (m^2)	Percentage of Macrophyte Zone
Open Water	> 0.7	113	15%
Submerged Marsh	0.35 - 0.7	74	10%
Deep Marsh	0.15 - 0.35	251	33%
Shallow Marsh	0.15	332	43%

A **70 mm** weir has been sized as the wetland outlet, to control Extended Detention Depths within the macrophyte zone. The residence time of the wetland has been calculated to equal **2 days** (for the 90th percentile of events), which represents the time taken for a particle of water to travel through the wetland system for all storm events. While the exact 90th percentile determined from the daily timeseries of residence times corresponds to 2 days, it has been determined that the macrophyte zone provides an 84th percentile residence time of 72 hours, which is in close proximity to Melbourne Water's requirement and is considered to be an acceptable result. See Figure 5-1 for a graphical representation of the macrophyte zone residence times.

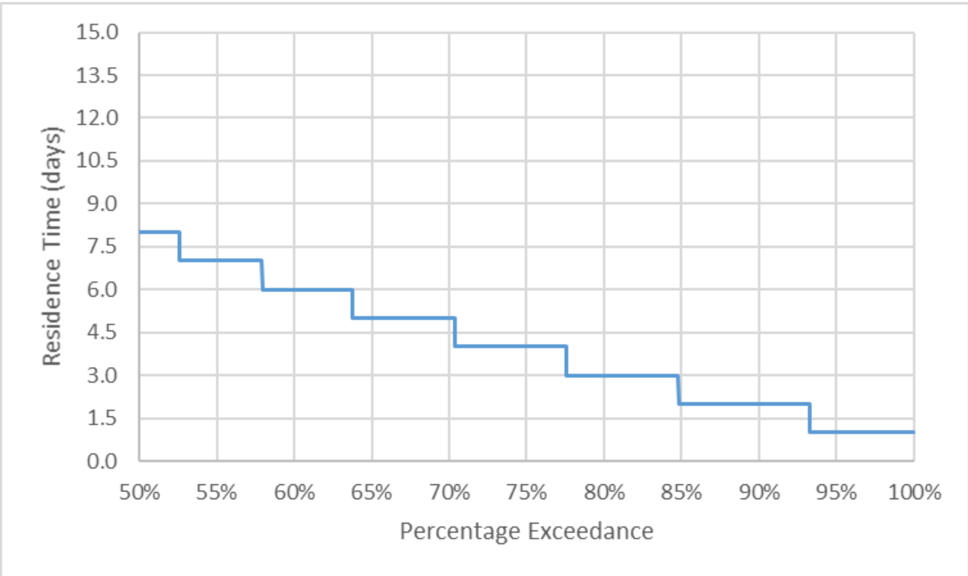

Figure 5-1 Macrophyte Residence Time

The macrophyte zone inundation frequency curve is shown below in Figure 5-2 which shows the distribution of expected water levels within the macrophyte zone. The effective water level is within 50 mm of the Normal Water Level and is acceptable.

The connection between the outlet pool and the twin chamber outfall pit has been sized to be a **525 mm** pipe, which was sized to convey the peak 100 year ARI flow given a head equal to the 100 year ARI level. The connection between the twin chamber outfall pit and the downstream drainage (assumed free flow condition) forms the high flow control for the basin.

A 300 mm diameter maintenance pipe has been adopted to drawdown the wetland.

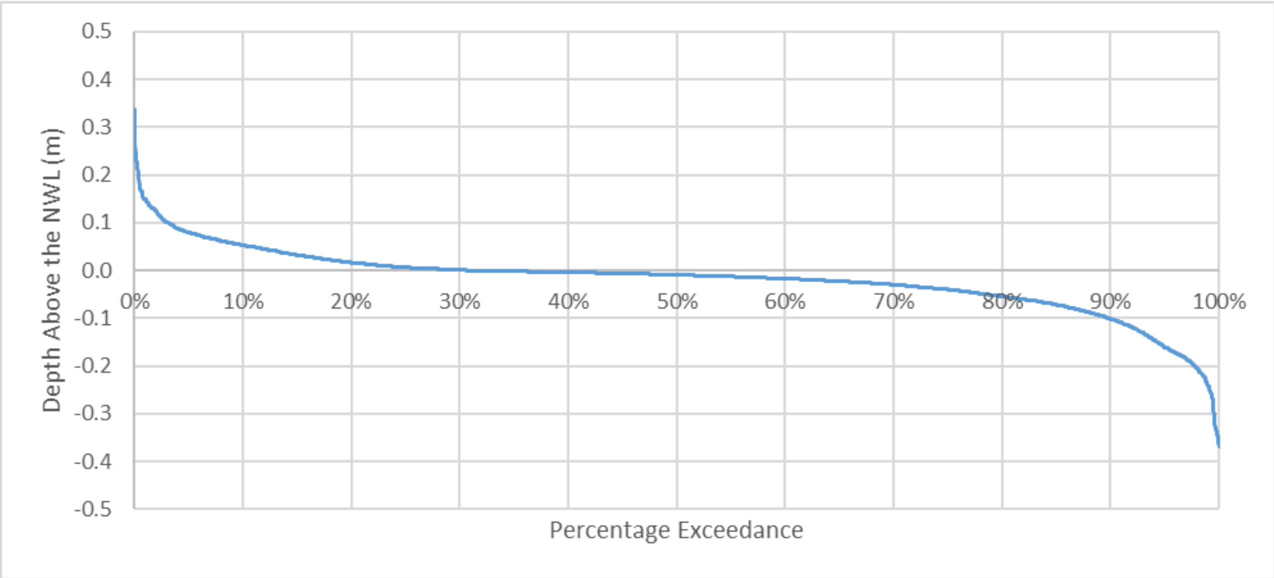

Figure 5-2 Macrophyte Zone Inundation Frequency Curve

5.3 Drainage Reserve

5.3.1 Edge Treatments

The approach batters to the NWL are approximately 1 in 6, with a 1 in 8 safety bench below the NWL's for a depth of 350 mm.

The sediment pond batter from TEDD to the base is 1 in 3. This will require dense impenetrable planting.

Remaining slopes within the retarding basin are no steeper than 1 in 6.

5.3.2 Maintenance Area

Informal maintenance access is available to the reserve area via the abutting subdivisional road network with a formal 4 m wide ramp included to provide access to the base of the sediment basin.

5.3.3 Drying Area

A silt storage area of approximately 250 m^2 is located at the south-west of the drainage reserve. This size will adequately cater for the 6 year ARI accumulated desilting volume (124 m^3) when distributed over an average depth of 500 mm. The location of the drying area is adjacent to the sediment basin maintenance track.

5.4 Flood Storage

Storage requirements for the peak 100 year ARI flood event will be met at an elevation of 47.2 m AHD as shown in Table 5-2, with a total volume of 1,620 m^3.

Table 5-2 Volume Above TED in the Proposed Retarding Basin

Elevations (m AHD)	Volume above TED (m^3)	Comment
46.5	0	RB storage base (TEDD level)
47.2	1,620	100 year WL

The 100 year flood level in the basin has been calculated excluding any storage volume below the TEDD in the wetland. The basin's design 100 year ARI flood level will be used to inform the fill and freeboard requirements around the basins. A minimum freeboard of 300 mm is expected to be required, to be confirmed by Council. No embankments are envisioned to be necessary for the basin construction. The spillway is proposed to provide an overland flow path for flows above the peak 100 year ARI flow.

The design is based on battering to existing levels. The basin design will need to tie into the surrounding design levels. No steep batters (steeper than 1:6) are proposed within the reserve. The future levels surrounding the system should also be set to minimise the need for steep interface batters and to allow post-development overland flows to enter the system as proposed.

6 DOWNSTREAM SYSTEM

6.1 Sediment Basin

Sediment basins have been sized via two methods, the Fair and Geyer formula (Equation 10.3 in WSUD Engineering Procedures, 2004) to determine minimum surface area required to ensure the settling velocity criteria for sediments (with a target of 95% removal rate for 125 µm particles), and a volume calculation to determine the accumulated sediment volume required within the Sediment Basin for a cleanout frequency of every 5 years.

The 3 month ARI flow rate of 0.30 m^3/s was used to determine the required basin via the Fair and Geyer formula, with the minimum area of **400 m^2** required. The accumulated volume calculation assumed a developed upstream catchment area of 9.6 ha and a sediment loading rate of 2.0 m^3/ha/year, which sized the required sediment storage volume to be **220 m^3**. It was found that the accumulated volume storage requirement was the critical factor in sizing of the sediment basin, not sediment settling velocities. It has been assumed that no GPT is located upstream of the sediment basin. The design of the sediment basin has an area of **748 m^2** and **325 m^3** of volume 500 mm below the NWL for accumulated volume and therefore is adequately sized for the upstream catchment.

An Extended Detention Depth of **500 mm** has been incorporated into the sediment basin.

Twin 300 mm diameter pipes has been sized to convey the 3 month ARI flow from the sediment basin to the wetland macrophyte zone. The sediment basin outlet pit (**1.2 m by 1.2 m**) has been sized (is large enough) to ensure that the pipe outlet forms the hydraulic control. Blockage factors within the transfer pit have been factored into the calculation.

The 100 year velocity in the sediment basin has been calculated following the approach outlined in the Melbourne Water Guidelines[3]. The calculation (using the cross sectional area between NWL and the 10 year ARI level in the basin) indicates that the maximum velocity in the basin would be equal to 0.12 m/s which is within the requirement of 0.5 m/s to prevent re-suspension of deposited sediment during a major storm event.

6.2 Wetland Macrophyte Zone

Table 6-1 shows a summary of the macrophyte zonation areas within the Upstream Wetland.

An Extended Detention Depth of 500 mm has been incorporated into the macrophyte zone. To determine the maximum 3 month velocities within the macrophyte zone, the calculation recommended in the Melbourne Water Guidelines[1] has been adopted. The estimated flow velocity was 0.01 m/s which is within the requirement of 0.05 m/s. The estimated 100 year flow velocity was 0.12 m/s which is within the requirement of 0.5 m/s.

[3] Melbourne Water (2015). Melbourne Water, Design, Construction and Establishment of Constructed Wetlands: Design Manual, updated 2017.

Table 6-1 Macrophyte Zonation Details

	Depth below NWL (m)	Area (m^2)	Percentage of Macrophyte Zone
Open Water	> 0.7	347	16%
Submerged Marsh	0.35 - 0.7	141	6%
Deep Marsh	0.15 - 0.35	819	37%
Shallow Marsh	0.15	903	41%

A **110 mm** weir has been sized as the wetland outlet, to control Extended Detention Depths within the macrophyte zone. The residence time of the wetland has been calculated to equal **2 days** (for the 90th percentile of events), which represents the time taken for a particle of water to travel through the wetland system for all storm events. While the exact 90th percentile determined from the daily timeseries of residence times corresponds to 2 days, it has been determined that the macrophyte zone provides an 85th percentile residence time of 72 hours, which is in close proximity to Melbourne Water's requirement and is considered to be an acceptable result. See Figure 6-1 for a graphical representation of the macrophyte zone residence times.

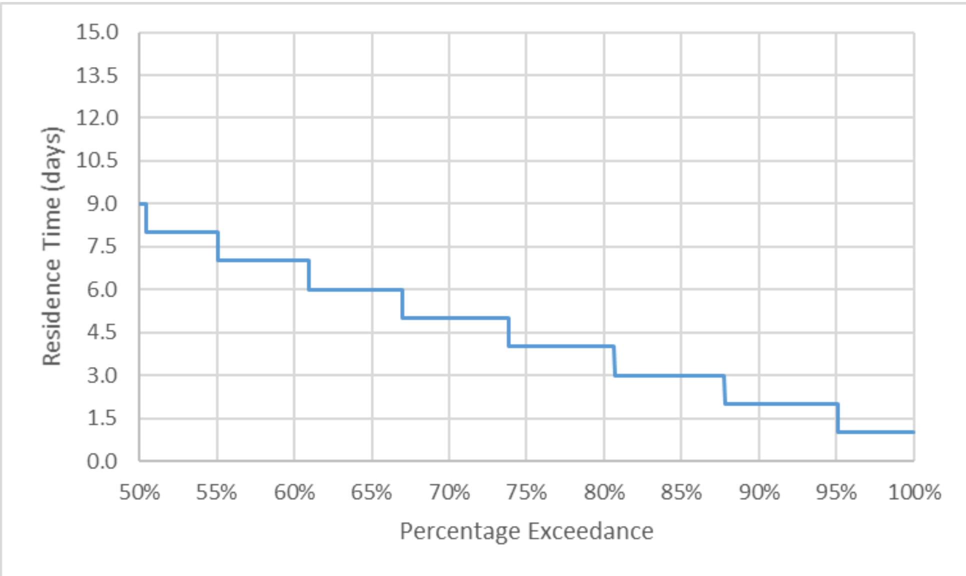

Figure 6-1 Macrophyte Residence Time

The macrophyte zone inundation frequency curve is shown below in Figure 6-2 which shows the distribution of expected water levels within the macrophyte zone. The effective water level is within 50 mm of the Normal Water Level and is acceptable.

The connection between the outlet pool and the twin chamber outfall pit has been sized to be an **825 mm** diameter pipe, which was calculated to convey the peak 100 year ARI flow given a head equal to the 100 year ARI level. The connection between the twin chamber outfall pit and the downstream drainage (assumed free flow condition) forms the high flow control for the basin.

A 300 mm diameter maintenance pipe has been adopted to drawdown the wetland.

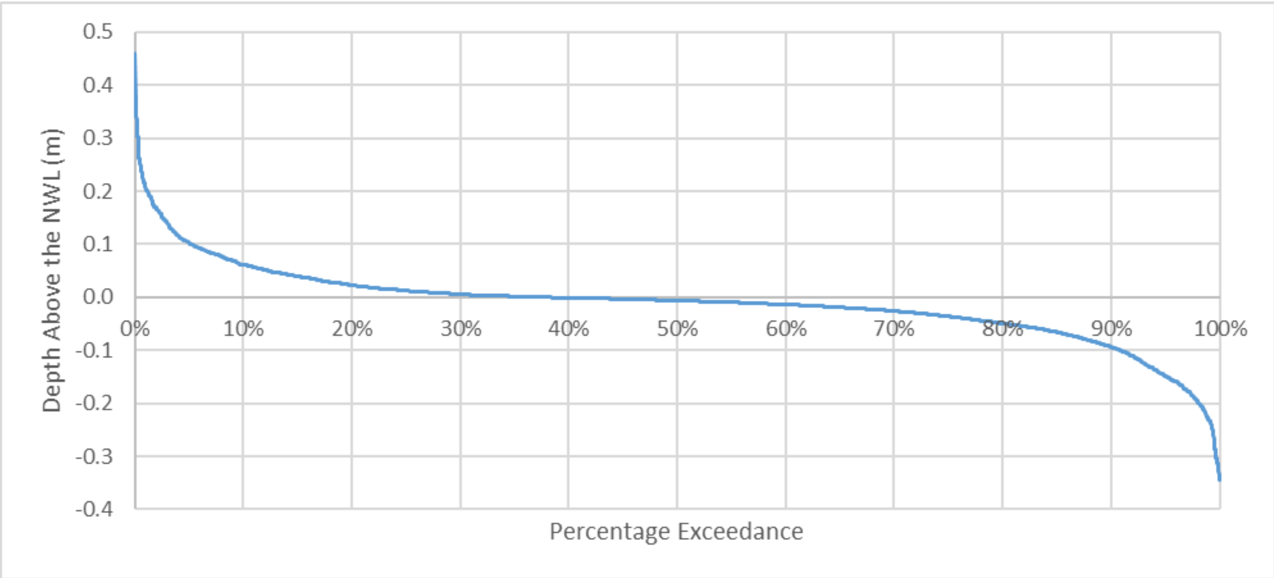

Figure 6-2 Macrophyte Zone Inundation Frequency Curve

6.3 Drainage Reserve

6.3.1 Edge Treatments

The approach batters to the NWL are approximately 1 in 6, with a 1 in 8 safety bench below the NWL's for a depth of 350 mm

Remaining slopes within the retarding basin are no steeper than 1 in 6.

6.3.2 Maintenance Area

Informal maintenance access is available to the reserve area via the abutting subdivisional road network with a formal 4 m wide ramp included to provide access to the base of the sediment basin.

6.3.3 Drying Area

A silt storage area of approximately 500 m^2 is located at the south-west of the drainage reserve. The size will adequately cater for the 6 year ARI accumulated desilting volume (250 m^3) when distributed over an average depth of 500 mm. The location of the drying area is adjacent to the sediment basin maintenance track.

6.4 Flood Storage

Storage requirements for the peak 100 year ARI flood event will be met at an elevation of 40.9 m AHD as shown in Table 6-2, with a total volume of 4,875 m^3.

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Table 6-2 Volume Above TED in the Proposed Retarding Basin

Elevations (m AHD)	Volume above TED (m^3)	Comment
40	0	RB storage base (TEDD level)
40.9	4,875	100 year WL

The 100 year flood level in the basin has been calculated excluding any storage volume below the TEDD in the wetland. The basin's design 100 year ARI flood level will be used to inform the fill and freeboard requirements around the basin. A minimum freeboard of 300 mm is expected to be required, to be confirmed by Council. A minor embankment (maximum height 1.3 m, but generally in the order of 0.5 m) is required to ensure the 100 year flood waters are contained within the proposed reserve. With this design, the 100 year flood level is still below the road crown level in Jetty Road. A spillway is proposed to provide an overland flow path for flows above the peak 100 year ARI flow.

The design is based on battering to existing levels. The basin design will need to tie into the surrounding design levels. No steep batters (steeper than 1:6) are proposed within the basin. The future levels surrounding the system should also be set to minimise the need for steep interface batters and to allow post-development overland flows to enter the system as proposed.

7 INTERIM STAGE

The above retarding basins allow for the whole subject site to be developed, with allowance for area where development is likely to be restricted by the existing floodplain. It is however, likely that the development will be staged. It will be possible to stage the construction of the westernmost retarding basin to be in-line with how the residential development will unfold.

This will also ensure that the construction of the ultimate water quality asset is timed with regard to construction sediment loading. Excessive sediment may damage macrophyte vegetation and compromise long-term treatment performance of a constructed wetland.

It was assumed, for the interim stage, that:

- The asset immediately upstream of Central Road may be constructed as a standalone asset, to service area east of Central Road (~9.5 ha);
- The water quality assets immediately upstream of Jetty Road may be constructed independently from the asset immediately upstream of Central Road;
  - The interim sediment basin would be constructed on 171 and 161 Jetty Road only.

The proposed interim sediment basin (shown in Appendix C) will be located on 171 and 161 Jetty Road, where the ultimate design wetland is to be located. Its south embankment will batter up to the southern boundary of 171 Jetty Road. It is proposed that the interim sediment basin will be in place as the upstream catchment develops.

The design of the asset will need to tie into the surrounding design levels, including to future development roads and parcels to the north and east.

7.1 Water Quality

This interim arrangement allows for the excavation and construction of the temporary sedimentation basin, and the wetlands being fully planted later, when sediment loading from upstream catchment would not result in adverse impacts:

- The sediment pond consists of two pools linked with a balance pipe:
  - The inlet and outlet pools of the ultimate wetland would act as sediment basin;
  - The entire pool is not 1 m deep as the levels and shape of the system are largely based on the ultimate design for the macrophyte zone. As such the entire interim volume provided exceeded volume requirements required to treat fully developed catchment.
- The macrophyte area between these two pools would be filled above TEDD and planted with terrestrial plants. The resulting mound will:
  - Be dry in low flow conditions, but engaged in larger events;
  - Provide additional aesthetics during the interim stage.

The volume provided exceeded volume requirements required to treat fully developed catchment. As a result, the sediment basin will remove in excess of 99% of sediment entering the system. It must be noted that sediment loading is higher during construction stage (between 50 and 200 m^3 per ha per year). The frequency of clean-up operations would therefore need to be adapted to staging of construction. As an example, clean-up operations would be required on a 6-monthly basis (assuming 100 m^3 per ha per year) if the construction progressed in 2.5 ha increments.

As per Best Practice Environmental Management Guidelines (BPEMG) for construction stages, the sediment basin will ensure 50% reduction in TSS.

7.2 Interim Flood Storage

The sediment basin is located at the base of a retarding basin. The reduced asset footprint results in a reduced flood volume. The site-specific RORB model (detailed in Section 3.1) was adjusted to model interim arrangement and estimate the maximum development area that can be catered by the interim retarding basin.

For the interim scenario, the model parameters for the subject site were iteratively adjusted to assess the extent of urbanisation:

- A revised fraction impervious value was used for the development area within subject site (corresponding to residential lots);
- Reaches were modelled with a reach type 3 (lined channels) if downstream of developed areas; and
- A lower initial loss was adopted, as per area developed.

This iterative process indicated that up to 16 ha would be developable under the interim scenario without exceeding pre-development discharge rate (2.02 m^3/s), or 25.5 ha if the assets immediately upstream of Central Road is also constructed. The configuration of the interim retarding basin is shown in Table 7-1. It was assumed that the ultimate outlet pipe (900 mm) is to be utilised as the interim arrangement outlet.

Table 7-1 Interim Retarding Basin Details

Retarding Basin Details	Upstream Jetty Road
Storage Volume (m^3)	3,580
Critical Duration (in the 100 year ARI)	1 hr
Peak 100 year ARI Outflow (m^3/s)	2.02
No. of Pipes (primary outlet)	1
Pipe IL (m AHD)	39.5
Pipe Diameter (mm)	900
Spillway Length (m)	15
Spillway Elevation (m AHD)	41.05

This 900 mm diameter pipe has been assumed to control the interim sediment pond detention depth. The exact arrangements of the outlet (twin chamber outlet pit) and how it is best incorporated with the ultimate design will need be refined when interim construction commences.

APPENDIX A
DESIGN CALCULATIONS

A-1 Sediment Basin Calculations

A-1-1 Sediment Basin Sizing

Upstream WL/RB

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 =	0.50	m	Extended Detention Depth			*max 0.35 for MW*					
d_p =	1.0	m	Permanent Pool Vollume Depth			*1.5 m is a common depth for standard residential developments*					
d* =	1.0	m	(lower of 1 m and dp)								
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.00										
Q =	0.15	m^3/s	Use rational method to obtain 1 Year ARI flow for sub catchment - then determine the 3 month ARI								
A =	**400**	m^2	Area of the sediment basin at NWL								
L/W =	2.0		Length/Width Ratio (assuming rectangular shape)								
$\frac{V_s}{Q/A}$ =	29.33										
λ =	0.18		Pond shape assumption (see figure 10.5 above)								
n =	1.22										
Fraction of Initial Solids Removed											
R =	**98%**										
Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle											
Cleanout Frequency											
Catchment area =			9.6	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						
Option 1	***Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***										
Actual basin depth =			0.50	m	500 mm below the NWL (i.e. dp-0.5)						
Actual Basin volume =			185.00	m^3	Basin volume at 500 mm below the NWL						
Therefore, cleanout frequency required =				$\frac{(1.6+0.4)A_{catchment}}{ActualBasinVolume}$ =		0.10	per year		Clean out every		9.6 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 =				96.00	m^3	volume of sediment acumulated up to 0.5 m below the NWL					
Required Dewatering area =				192.00	m^2						
Provided dewatering area =				250.0	m^2	**OK**					

Downstream WL/RB

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 =	0.50	m	Extended Detention Depth	*max 0.35 for MW*
d_p =	1.0	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)	
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.00			
Q =	0.30	m^3/s	Use rational method to obtain 1 Year ARI flow for sub catchment - then determine the 3 month ARI	
A =	748	m^2	Area of the sediment basin at NWL	
L/W =	1.0		Length/Width Ratio (assuming rectangular shape)	
$\frac{V_s}{Q/A}$ =	27.43			
λ =	0.11		Pond shape assumption (see figure 10.5 above)	
n =	1.12			
Fraction of Initial Solids Removed				
R =	97%			
Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle				
Cleanout Frequency				
Catchment area =	22.2	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	
Option 1	***Assumes clean out when sediment level is 500mm below NWL (MW Wetland Guidelines 2015)***			
Actual basin depth =	0.50	m	500 mm below the NWL (i.e. dp-0.5)	
Actual Basin volume =	325.00	m^3	Basin volume at 500 mm below the NWL	## 250 from 12D
Therefore, cleanout frequency required =	$\frac{(1.6+0.4)A_{catchment}}{ActualBasinVolume}$ =	0.14 per year	Clean out every	7.3 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 =	222.00	m^3	volume of sediment acumulated up to 0.5 m below the NWL	
Required Dewatering area =	444.00	m^2		
Provided dewatering area =	500.0	m^2	OK	

A-1-2 Sediment Basin Transfer Pit/Pipe

Upstream WL/RB

Ignore the pit?? Yes/No			No		Are you Assuming INLET CONTROL??? Yes/No				No		
PIT					PIPE	INLET CONTROL		OUTLET CONTROL			
	Pit Sill Level	46	m AHD		Invert Level	45.3	m AHD	Ke	0.5	Square edge	
	Width	1.2	m		Dia	0.225	m	Kex	1.0		
	Length	1.2	m		barrels	2		Length	18.0	m	
	Area	1.44	m^2		Area	0.080	m^2	Tailwater Level	46.0	m AHD	
	portion not taken by b	62	%								
	Grill blockage factor	50	%					wetted perim	0.71	m	
	Weir blockage factor	50	%					Hyd Radius	0.11	m	
Height	Head	Orifice Flow	Weir Flow	total flow into pit	Head	Orifice Flow		head loss	flow	TOTAL FLOW RATE	
46	0	0.000	0.000	0.000	0.588	0.178		0.000	0.000	0.00	m^3/s
46.1	0.1	0.413	0.129	0.129	0.688	0.193		0.100	0.069	0.07	m^3/s
46.2	0.2	0.584	0.365	0.365	0.788	0.206		0.200	0.098	0.10	m^3/s
46.3	0.3	0.715	0.670	0.670	0.888	0.219		0.300	0.120	0.12	m^3/s
46.4	0.4	0.825	1.032	0.825	0.988	0.231		0.400	0.138	0.14	m^3/s
46.5	**0.5**	**0.923**	**1.442**	**0.923**	**1.088**	**0.242**		**0.500**	**0.154**	**0.15**	**m^3/s**

Downstream WL/RB

Ignore the pit?? Yes/No			No		Are you Assuming INLET CONTROL??? Yes/No				No		
PIT					PIPE	INLET CONTROL		OUTLET CONTROL			
	Pit Sill Level	39.5	m AHD		Invert Level	38.8	m AHD	Ke	0.5	Square edge	
	Width	1.2	m		Dia	0.3	m	Kex	1.0		
	Length	1.2	m		barrels	2		Length	18.0	m	
	Area	1.44	m^2		Area	0.141371669	m^2	Tailwater Level	39.5	m AHD	
	portion not taken by b	62	%								
	Grill blockage factor	50	%					wetted perim	0.94	m	
	Weir blockage factor	50	%					Hyd Radius	0.15	m	
Height	Head	Orifice Flow	Weir Flow	total flow into pit	Head	Orifice Flow		head loss	flow	TOTAL FLOW RATE	
39.5	0	0.000	0.000	0.000	0.550	0.307		0.000	0.000	0.00	m^3/s
39.6	0.1	0.413	0.129	0.129	0.650	0.333		0.100	0.132	0.13	m^3/s
39.7	0.2	0.584	0.365	0.365	0.750	0.358		0.200	0.187	0.19	m^3/s
39.8	0.3	0.715	0.670	0.670	0.850	0.381		0.300	0.229	0.23	m^3/s
39.9	0.4	0.825	1.032	0.825	0.950	0.403		0.400	0.264	0.26	m^3/s
40	**0.5**	**0.923**	**1.442**	**0.923**	**1.050**	**0.423**		**0.500**	**0.295**	**0.30**	**m^3/s**

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A-2 Macrophyte Zone Weir Outlet

Upstream WL/RB

Wetland EDD	0.5	m	Starting Water Depth	Detention Time	
Wetland Top Area	1044	m2	100mm	40	hrs
Wetland Bottom Area	773	m2	200mm	43	hrs
Volume at EDD	454.3	m3	300mm	45	hrs
			400mm	46	hrs
			500mm	45	hrs
			Average Detention Depth	43.8	hrs

orifice configuration

Weir Coefficient	0.6
Depth (m)	weir Width (mm)
0	70
0.2	0
0.3	0

Height Discharge Table

Height	Weir 1	Weir 2	Weir 3	TOTAL FLOW		
0	0	0	0	0	0.000	m3/s
0.1	0.003922	0	0	0.1	0.004	m3/s
0.5	0.043849	0	0	0.5	0.044	m3/s
1	0.124025	0	0	1	0.124	m3/s
1.5	0.227848	0	0	1.5	0.228	m3/s
2	0.350794	0	0	2	0.351	m3/s
2.5	0.49025	0	0	2.5	0.490	m3/s

Downstream WL/RB

Wetland EDD	0.5	m
Wetland Top Area	2497	m2
Wetland Bottom Area	2124	m2
Volume at EDD	1155.3	m3

Starting Water Depth	Detention Time	
100mm	64	hrs
200mm	70	hrs
300mm	73	hrs
400mm	75	hrs
500mm	76	hrs
Average Detention Depth	71.6	hrs

orifice configuration

Weir Coefficient	0.6

Depth (m)	weir Width (mm)
0	110
0.2	0
0.3	0

0

Height Discharge Table

Height	Weir 1	Weir 2	Weir 3		TOTAL FLOW	
0	0	0	0	0	0.000	m3/s
0.1	0.006163	0	0	0.1	0.006	m3/s
0.5	0.068906	0	0	0.5	0.069	m3/s
1	0.194896	0	0	1	0.195	m3/s
1.5	0.358046	0	0	1.5	0.358	m3/s
2	0.551248	0	0	2	0.551	m3/s
2.5	0.770393	0	0	2.5	0.770	m3/s

Height – Discharge table input into MUSIC, from which Flux files were extracted to determine residence times.

A-3 Macrophyte Zone Outlet Pipes

A-3-1 Outlet to Twin Chamber

Upstream WL/RB

Ignore the pit?? Yes/No			Yes		Are you Assuming INLET CONTROL??? Yes/No				No		
PIT					PIPE	INLET CONTROL		OUTLET CONTROL			
	Pit Sill Level	46.5	m AHD		Invert Level	45.3	m AHD	Ke	0.5	Square edge	
	Width	1.4	m		Dia	0.525	m	Kex	1.0		
	Length	1.4	m		barrels	1		Length	11.0	m	
	Area	1.96	m^2		Area	0.216475	m^2	Tailwater Level	46.0	m AHD	
	portion not taken by b	62	%								
	Grill blockage factor	50	%					wetted perim	1.65	m	
	Weir blockage factor	50	%					Hyd Radius	0.13	m	
Height	Head	Orifice Flow	Weir Flow	total flow into pit	Head	Orifice Flow		head loss	flow	TOTAL FLOW RATE	
46	-0.5	0.000	0.000	NA	0.438	0.419		0.000	0.000	0.00	m^3/s
46.1	-0.4	0.000	0.000	NA	0.538	0.464		0.100	0.212	0.21	m^3/s
46.2	-0.3	0.000	0.000	NA	0.638	0.505		0.200	0.300	0.30	m^3/s
46.3	-0.2	0.000	0.000	NA	0.738	0.543		0.300	0.367	0.37	m^3/s
46.4	-0.1	0.000	0.000	NA	0.838	0.579		0.400	0.424	0.42	m^3/s
46.5	0	0.000	0.000	NA	0.938	0.613		0.500	0.474	0.47	m^3/s
46.6	0.1	0.562	0.151	NA	1.038	0.645		0.600	0.519	0.52	m^3/s
46.7	0.2	0.794	0.426	NA	1.138	0.675		0.700	0.561	0.56	m^3/s
46.8	0.3	0.973	0.782	NA	1.238	0.704		0.800	0.599	0.60	m^3/s
46.9	0.4	1.123	1.204	NA	1.338	0.732		0.900	0.636	0.64	m^3/s
47	0.5	1.256	1.683	NA	1.438	0.759		1.000	0.670	0.67	m^3/s
47.1	0.6	1.376	2.212	NA	1.538	0.785		1.100	0.703	0.70	m^3/s
47.2	**0.7**	**1.486**	**2.788**	**NA**	**1.638**	**0.810**		**1.200**	**0.734**	**0.73**	**m^3/s**

Downstream WL/RB

Ignore the pit?? Yes/No			Yes		Are you Assuming INLET CONTROL??? Yes/No				No		
PIT					PIPE	INLET CONTROL		OUTLET CONTROL			
	Pit Sill Level	40	m AHD		Invert Level	38.8	m AHD	Ke	0.5	Square edge	
	Width	1.4	m		Dia	0.825	m	Kex	1.0		
	Length	1.4	m		barrels	1		Length	15.0	m	
	Area	1.96	m^2		Area	0.53	m^2	Tailwater Level	39.5	m AHD	
	portion not taken by b	62	%								
	Grill blockage factor	50	%					wetted perim	2.59	m	
	Weir blockage factor	50	%					Hyd Radius	0.21	m	
Height	Head	Orifice Flow	Weir Flow	total flow into pit	Head	Orifice Flow		head loss	flow	TOTAL FLOW RATE	
39.5	-0.5	0.000	0.000	NA	0.288	0.838		0.000	0.000	0.00	m^3/s
39.6	-0.4	0.000	0.000	NA	0.388	0.973		0.100	0.542	0.54	m^3/s
39.7	-0.3	0.000	0.000	NA	0.488	1.091		0.200	0.767	0.77	m^3/s
39.8	-0.2	0.000	0.000	NA	0.588	1.198		0.300	0.939	0.94	m^3/s
39.9	-0.1	0.000	0.000	NA	0.688	1.296		0.400	1.084	1.08	m^3/s
40	0	0.000	0.000	NA	0.788	1.387		0.500	1.212	1.21	m^3/s
40.1	0.1	0.562	0.151	NA	0.888	1.472		0.600	1.328	1.33	m^3/s
40.2	0.2	0.794	0.426	NA	0.988	1.553		0.700	1.434	1.43	m^3/s
40.3	0.3	0.973	0.782	NA	1.088	1.630		0.800	1.533	1.53	m^3/s
40.4	0.4	1.123	1.204	NA	1.188	1.703		0.900	1.626	1.63	m^3/s
40.5	0.5	1.256	1.683	NA	1.288	1.773		1.000	1.714	1.71	m^3/s
40.6	0.6	1.376	2.212	NA	1.388	1.841		1.100	1.798	1.80	m^3/s
40.7	0.7	1.486	2.788	NA	1.488	1.906		1.200	1.878	1.88	m^3/s
40.8	0.8	1.589	3.406	NA	1.588	1.969		1.300	1.954	1.95	m^3/s
40.9	0.9	1.685	4.064	NA	1.688	2.030		1.400	2.028	2.03	m^3/s
41	**1**	**1.776**	**4.760**	**NA**	**1.788**	**2.089**		**1.500**	**2.099**	**2.09**	**m^3/s**

A-3-2 Outlet from Twin Chamber

Upstream WL/RB

Ignore the pit?? Yes/No			No		Are you Assuming INLET CONTROL??? Yes/No				Yes		
PIT					PIPE	INLET CONTROL		OUTLET CONTROL			
	Pit Sill Level	46.5	m AHD		Invert Level	46	m AHD	Ke	0.5	Square edge	
	Width	1.4	m		Dia	0.525	m	Kex	1.0		
	Length	1.4	m		barrels	1		Length	11.0	m	
	Area	1.96	m^2		Area	0.216	m^2	Tailwater Level	46.5	m AHD	
	portion not taken by b	62	%								
	Grill blockage factor	50	%					wetted perim	1.65	m	
	Weir blockage factor	50	%					Hyd Radius	0.13	m	
Height	Head	Orifice Flow	Weir Flow	total flow into pit	Head	Orifice Flow		head loss	flow	TOTAL FLOW RATE	
46	-0.5	0.000	0.000	0.000	-0.263	0.000		-0.500	NA	0.00	m^3/s
46.1	-0.4	0.000	0.000	0.000	-0.163	0.000		-0.400	NA	0.00	m^3/s
46.2	-0.3	0.000	0.000	0.000	-0.063	0.000		-0.300	NA	0.00	m^3/s
46.3	-0.2	0.000	0.000	0.000	0.038	0.123		-0.200	NA	0.00	m^3/s
46.4	-0.1	0.000	0.000	0.000	0.138	0.235		-0.100	NA	0.00	m^3/s
46.5	0	0.000	0.000	0.000	0.238	0.308		0.000	NA	0.00	m^3/s
46.6	0.1	0.562	0.151	0.151	0.338	0.368		0.100	NA	0.15	m^3/s
46.7	0.2	0.794	0.426	0.426	0.438	0.419		0.200	NA	0.42	m^3/s
46.8	0.3	0.973	0.782	0.782	0.538	0.464		0.300	NA	0.46	m^3/s
46.9	0.4	1.123	1.204	1.123	0.638	0.505		0.400	NA	0.51	m^3/s
47	0.5	1.256	1.683	1.256	0.738	0.543		0.500	NA	0.54	m^3/s
47.1	0.6	1.376	2.212	1.376	0.838	0.579		0.600	NA	0.58	m^3/s
47.2	**0.7**	**1.486**	**2.788**	**1.486**	**0.938**	**0.613**		**0.700**	**NA**	**0.61**	**m^3/s**

Downstream WL/RB

Ignore the pit?? Yes/No			No		Are you Assuming INLET CONTROL??? Yes/No				Yes		
PIT					PIPE	INLET CONTROL		OUTLET CONTROL			
	Pit Sill Level	40	m AHD		Invert Level	39.4	m AHD	Ke	0.5	Square edge	
	Width	1.4	m		Dia	0.9	m	Kex	1.0		
	Length	1.4	m		barrels	1		Length	20.0	m	
	Area	1.96	m^2		Area	0.636	m^2	Tailwater Level	39.5	m AHD	
	portion not taken by b	70	%								
	Grill blockage factor	50	%					wetted perim	2.83	m	
	Weir blockage factor	50	%					Hyd Radius	0.23	m	
Height	Head	Orifice Flow	Weir Flow	total flow into pit	Head	Orifice Flow		head loss	flow	TOTAL FLOW RATE	
39.5	-0.5	0.000	0.000	0.000	-0.350	0.000		0.000	NA	0.00	m^3/s
39.6	-0.4	0.000	0.000	0.000	-0.250	0.000		0.100	NA	0.00	m^3/s
39.7	-0.3	0.000	0.000	0.000	-0.150	0.000		0.200	NA	0.00	m^3/s
39.8	-0.2	0.000	0.000	0.000	-0.050	0.000		0.300	NA	0.00	m^3/s
39.9	-0.1	0.000	0.000	0.000	0.050	0.416		0.400	NA	0.00	m^3/s
40	0	0.000	0.000	0.000	0.150	0.720		0.500	NA	0.00	m^3/s
40.1	0.1	0.634	0.151	0.151	0.250	0.930		0.600	NA	0.15	m^3/s
40.2	0.2	0.897	0.426	0.426	0.350	1.100		0.700	NA	0.43	m^3/s
40.3	0.3	1.098	0.782	0.782	0.450	1.248		0.800	NA	0.78	m^3/s
40.4	0.4	1.268	1.204	1.204	0.550	1.379		0.900	NA	1.20	m^3/s
40.5	0.5	1.418	1.683	1.418	0.650	1.499		1.000	NA	1.42	m^3/s
40.6	0.6	1.553	2.212	1.553	0.750	1.611		1.100	NA	1.55	m^3/s
40.7	0.7	1.678	2.788	1.678	0.850	1.715		1.200	NA	1.68	m^3/s
40.8	0.8	1.794	3.406	1.794	0.950	1.813		1.300	NA	1.79	m^3/s
40.9	**0.9**	**1.903**	**4.064**	**1.903**	**1.050**	**1.906**		**1.400**	**NA**	**1.90**	**m^3/s**

A-4 Velocity Checks

US Sediment pond		
Q10yr =	0.92	m^3/s
Q100yr =	1.88	m^3/s
Width at the NWL =	14.15	m
Width at the10Yr =	21	m
Average width =	17.575	m
NWL	46	m AHD
Water level at 10yr =	47	m AHD
diff h =	1	m
Cross section flow area =	17.575	m^2
V100yr	0.11	< 0.5 m/s

US Macrophyte Zone		
Q3month =	0.15	m^3/s
Q10yr =	0.9	m^3/s
Q100yr =	1.88	m^3/s
Width at the NWL =	9.4	m
Width at the TED =	12.9	m
Width at the10Yr =	39.5	m
NWL =	46	m AHD
TED =	46.5	m AHD
Water level at 10yr =	47	m AHD
Cross section flow area 3month =	5.575	m^2
Cross section flow area 100yr =	24.45	m^2
V100yr	0.08	< 0.5 m/s
V3month	0.03	<0.05 m/s

DS Sediment pond		
Q10yr =	1.82	m^3/s
Q100yr =	3.65	m^3/s
Width at the NWL =	21.5	m
Width at the10Yr =	32.23	m
Average width =	26.865	m
NWL	39.5	m AHD
Water level at 10yr =	40.62	m AHD
diff h =	1.12	m
Cross section flow area =	30.0888	m^2
V100yr	0.12	< 0.5 m/s

DS Macrophyte Zone		
Q3month =	0.3	m^3/s
Q10yr =	1.8	m^3/s
Q100yr =	3.65	m^3/s
Width at the NWL =	23.39	m
Width at the TED =	26.25	m
Width at the10Yr =	32.85	m
NWL =	39.5	m AHD
TED =	40.5	m AHD
Water level at 10yr =	40.62	m AHD
Cross section flow area 3month =	24.82	m^2
Cross section flow area 100yr =	31.4944	m^2
V100yr	0.12	< 0.5 m/s
V3month	0.01	<0.05 m/s

APPENDIX B
LAYOUT PLANS

FUTURE LEVELS SURROUNDING THE BASIN RESERVE WILL DICTATE THE PRECISE INTERFACE EDGE TREATMENTS FOR THE SYSTEM.

THE CURRENT DESIGN KEEPS ALL BATTERS TO A MINIMUM OF 1 IN 6

Issue	Description	Approved	Date	Issue	Description	Checked	Approved	Date
03	REVISED RB BATTERS	TJC	09/07/2018					
02	REVISED RB BATTERS TO NO STEEPER THAN 1 in 6	BFS	01/05/2018					
01	PRELIMINARY ISSUE	CBD	05/04/2018					

Scale HORZ 1:200
VERT 1:200

Client ST QUENTIN

Drawn ADV/BYT
Designed BFS/ADV
Modelled BFS
Project: J4663-02

Legend
- NORMAL WATER LEVEL (NWL)
- TOP EXTENDED DETENTION DEPTH (TEDD)
- SEDIMENT DRYING AREA
- PROPSED PIPES
- GOOD RETENTION VALUE VEGETATION
- 1% AEP FLOOD LEVEL (47.2)

Water Technology Pty Ltd
15 Business Park Drive
Notting Hill
VIC 3168
ACN 056 019 713
Tel: +61 3 8526 0800
Fax: +61 3 9558 9365
Email: admin@watertech.com.au
Web: http://www.watertech.com.au

Title
CENTRAL ROAD, DRYSDALE
US WL/RB
FUNCTIONAL LAYOUT PLAN

Dwg. No. 1
Job No. J4663-02
Page Size A1

Issue	Description		Approved	Date	Issue	Description	Checked	Approved	Date
03	REVISED RB BATTERS		TJC	09/07/2018					
02	REVISED RB BATTERS TO NO STEEPER THAN 1 in 6		BFS	01/05/2018					
01	PRELIMINARY ISSUE		CBD	05/04/2018					

Scale HORZ 1:400
VERT 1:400

Client ST QUENTIN

Drawn ADV/BYT
Designed BFS/ADV
Modelled BFS
Project: J4663-02

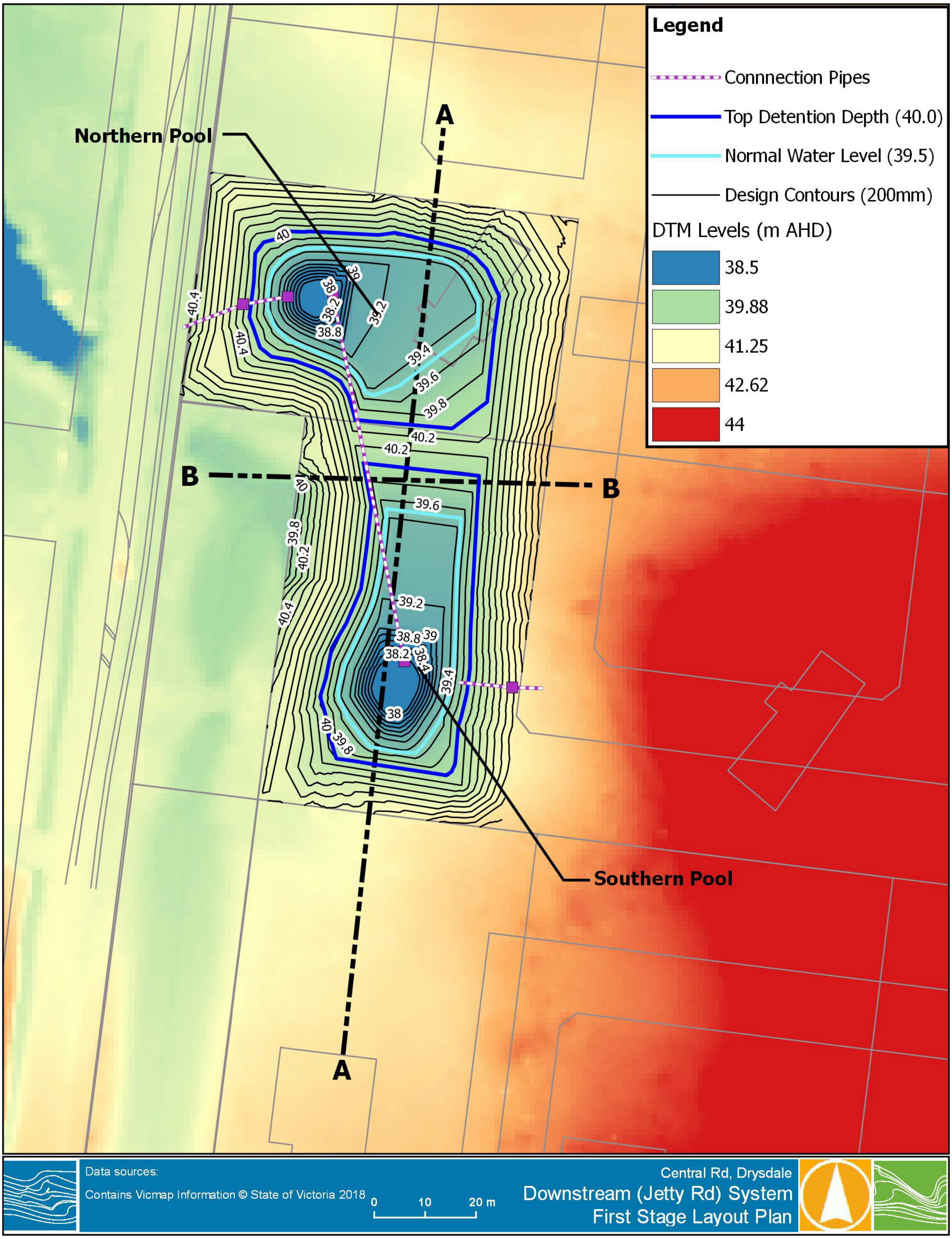
Legend
- NORMAL WATER LEVEL (NWL)
- TOP EXTENDED DETENTION DEPTH (TEDD)
- BASIN EMBANKMENT
- SEDIMENT DRYING AREA
- PROPSED PIPES
- GOOD RETENTION VALUE VEGETATION
- 1% AEP FLOOD LEVEL (40.9)

Water Technology Pty Ltd
WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS
15 Business Park Drive
Notting Hill
VIC 3168
ACN 056 019 713
Tel: +61 3 8526 0800
Fax: +61 3 9558 9365
Email: admin@watertech.com.au
Web: http://www.watertech.com.au

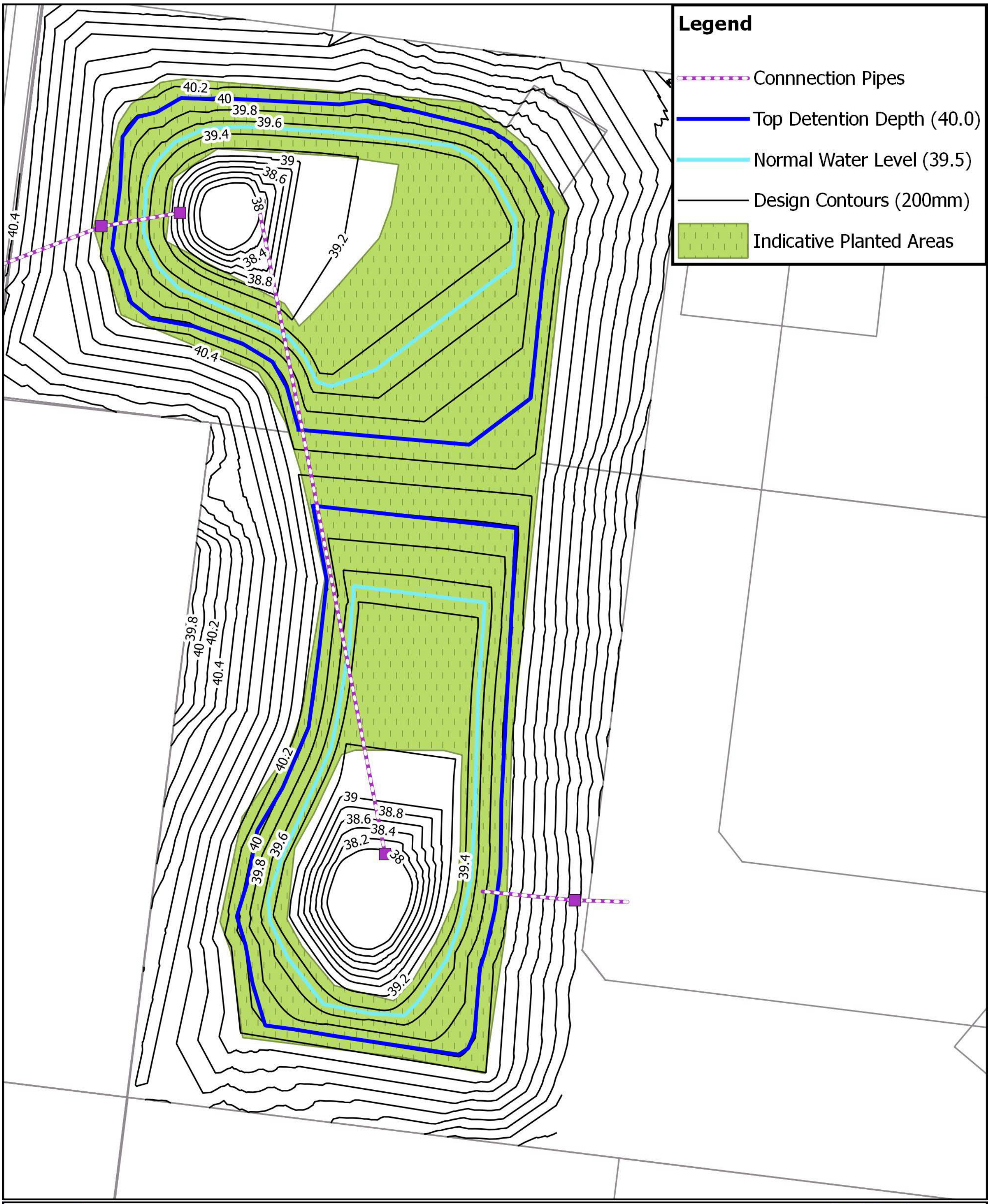
Title
CENTRAL ROAD, DRYSDALE
DS WL/RB
FUNCTIONAL LAYOUT PLAN

Dwg. No. 2
Job No. J4663-02
Page Size A1

APPENDIX C
INTERIM LAYOUT PLANS



2018-07-10T12:08:08



Data sources:

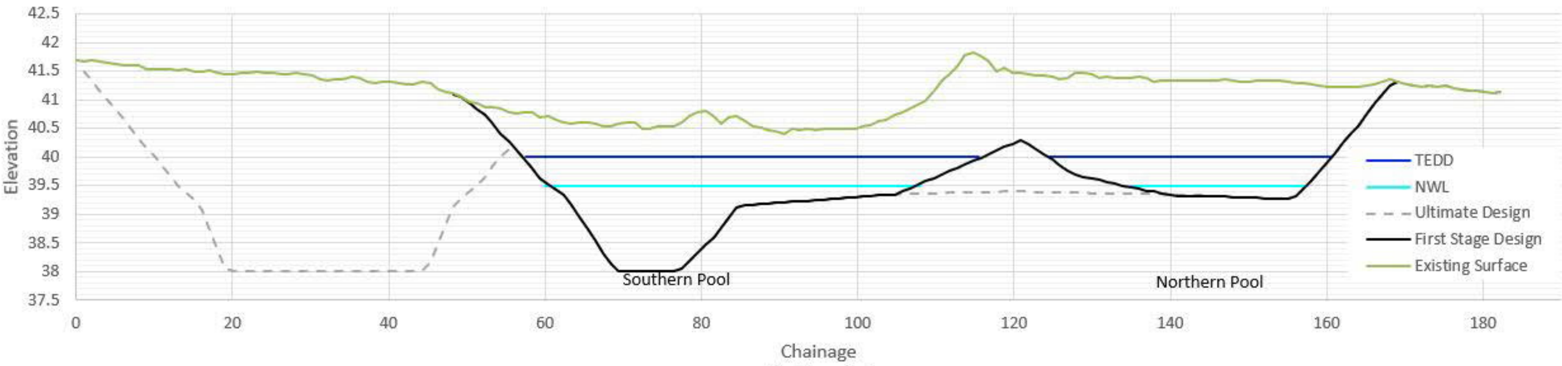
0 5 10 m

Central Rd, Drysdale

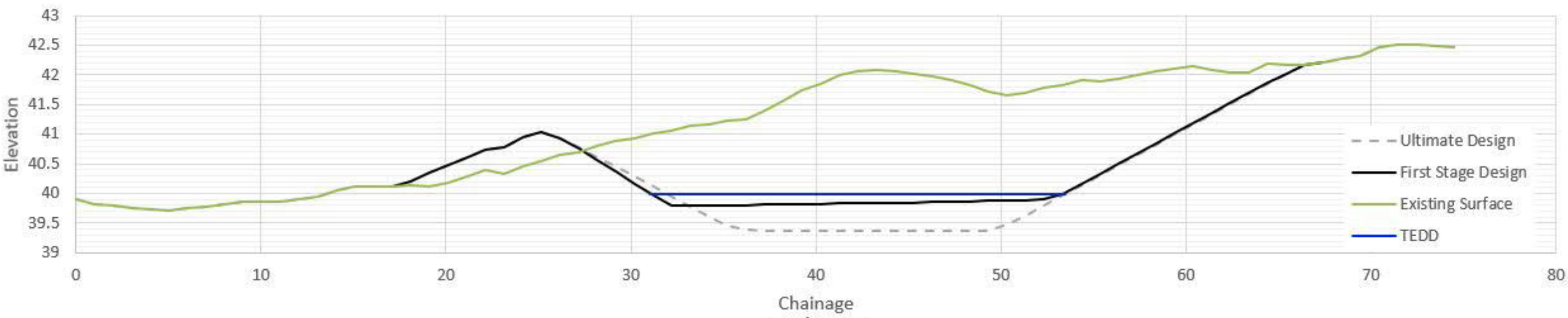
Downstream (Jetty Rd) System

First Stage Indicative Planted Areas

2018-07-10T12:06:04



Section A-A



Section B-B

Melbourne

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

Brisbane

Level 3, 43 Peel Street
South Brisbane QLD 4101
Telephone (07) 3105 1460
Fax (07) 3846 5144

Wangaratta

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

Perth

PO Box 362
Subiaco WA 6904
Telephone 0438 347 968

Geelong

PO Box 436
Geelong VIC 3220
Telephone 0458 015 664

Gippsland

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

Wimmera

PO Box 584
Stawell VIC 3380
Telephone 0438 510 240

www.watertech.com.au

info@watertech.com.au