Hydrological and Environmental Engineering

Berwick Waterways

Storm Water Management Plan

DRAFT

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Contents

1. Introduction

Berwick Waterways refers to the area bounded by the levee banks of Hallam Valley Contour Drain and Berwick Town Drain to the west, Greaves Road to the south and existing residential development to the east and north. MWC have an existing drainage strategy incorporating a linear wetland and filling of land to facilitate development.

Various reports detail the drainage requirements of Berwick Waterways. In particular:

1. Homestead Road Drainage Area Study, Proposed Wetland Pondage System, Stage 2 Final Concept Design, Prepared for: Melbourne Water Corporation and City of Casey, 31 January 2005, Neil M Craigie and Pat Condina (Report 1).
2. September 2009 report by Stormy Water Solutions (SWS) and Neil M Craigie Pty Ltd "Berwick Waterways, Summary Issues Paper, 14 September 2009" (Report 2).
3. The Growth Areas Authority (GAA) produced a report by SWS and NM Craigie Pty Ltd entitled "Berwick Waterways, Drainage Assessment, Options Development and Appraisal, November 2009" (Report 3).
4. The SWS report "Berwick Waterways, Drainage System Requirements, 24 February 2011" (Report 4), and
5. The "Hallam Valley Contour Drain, Flood Investigation, Narre Warren/Berwick, *Draft,*10, SWS, August 2011" (Report 5), and
6. The "Hallam Valley Flood Plain, Master Plan, Narre Warren/Berwick,SWS, 16 April 2012" (Report 6).

The reports :

- Formulated a drainage strategy for the DSS area which accounted for both the internal and external drainage implications of the land,
- Formulated a drainage strategy for the DSS area which minimised filling and drainage costs associated with the future development of the area to help determine if development of the DSS area was actually feasible (physically and financially), and
- Investigated the implications in regard to the extreme flood event which occurred in the area in February 2011 in regard to development feasibility.

This report assumes that the reader has some background knowledge in regard to the drainage configuration and operation in the area, and in regard to the formulation of various drainage strategies in the past.

At this time, Moremac Property Group and their consultant team (KLM Spatial Pty Ltd) have formulated a proposed subdivision layout and retarding basin/wetland reserve configuration.

This Storm Water Management Plan (SWMP) developed by Stormy Water Solutions (SWS) aims to show that all drainage requirements as previously and currently specified by Council and Melbourne Water can be met within the current proposals.

Although this is a SWMP, given the complex nature of this system, the proposals detailed in this report are almost at a functional design level at this stage. That is, system extents have been determined given consideration of all physical constraints such downstream invert levels, existing upstream outfall invert levels, wetland edge treatment proposed etc). Details such as final subdivisional pipe sizes and alignments have yet to be set. Details such as these will be affected by the proposals in regard to appropriate wetland outfall locations etc.

The flood event of 4 and 5th February 2011 indicated that additional system outlet and Hallam Valley Contour drain (HVCD) levee investigations and works are required to ensure the proposals to date are adequate to protect any future development from flooding by the Hallam Valley Contour Drain. The requirements of future investigations (assumed to be performed at the functional design stage of the project) are detailed in this report.

It is requested that MWC and Council provide approval in principal for the SWMP at this time so that timely and coordinated planning of Berwick Waterways can occur in the short term.

Unlike previous reports, this work concentrates on the technical requirements of the drainage system. Financial consideration in regard to development feasibility given cost constraints etc are outside the scope of this report.

2. Background

Figure 1 details the major drainage features in the area.

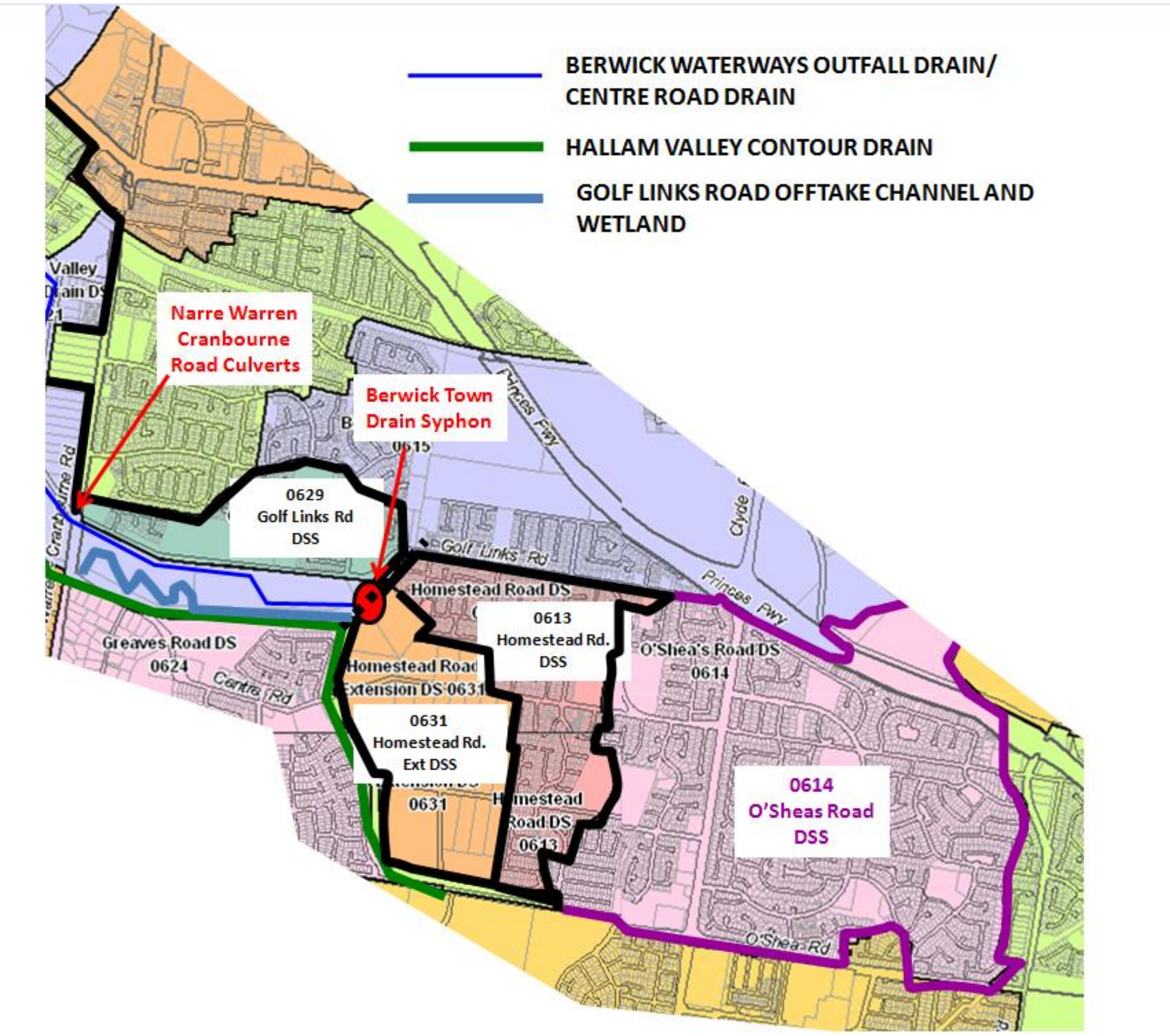


Figure 1 **Drainage Configuration**

In Figure 1:

- The area shown at the DSS 0631 (Homestead Road Extension DSS) is the Berwick Waterways development area,
- The O'Sheas Road DSS discharges directly to the HVCD,
- The Berwick Town Drain syphon and the downstream Centre Road Drain form the outfall system from Berwick Waterways (0631) and the existing Homestead Road DSS (0613),
- HVCD, the Golf Links Road wetland off take and the Centre Road Drain are separated by levees downstream of Berwick Waterways, resulting in each of the systems operating independently in rainfall events.

2.1 Hallam Valley Contour Drain

The Hallam Valley Contour Drain (HVCD) is a large man made drain which forms the western boundary of Berwick Waterways. The HVCD has a large external catchment extending south to Cranbourne from Berwick.

In a “normal” mode of operation, the Hallam Valley Contour Drain (HVCD) discharges from the Greaves Road Retarding Basin to the confluence with the Berwick Town Drain (BTD). From this point flow is conveyed west to Narre Warren Cranbourne Road.

Being a contour drain, the HVCD utilises a levee on its right bank (looking downstream) to protect properties east and north of the drain from flooding. This includes the Berwick Waterways development area. Flooding is designed to preferentially occur to the west and south of the drain. The levee also extends along the eastern bank of the BTD, and along the northern edge of Greaves Road to fully enclose the Homestead Road/Berwick Waterways area with a flood protection levee.

2.2 Berwick Town Drain Off take and Golf Links Road Wetland

In 2004, the Golf Links Road wetland system was constructed north of the HVCD, east of Narre Warren Cranbourne Road. A low flow connection was constructed on the BTD (just downstream of the Berwick Waterways outlet syphon) to feed the wetland system with BTD flows. This low flow connection consists of:

- A rock weir in the BTD (just upstream of the HVCD confluence),
- Triple 1050 off take pipes to the low flow channel,
- A sluice gate system on the off take pipes to restrict flow from the BTD into the wetland system, especially during flood events (MWC have previously advised that the sluice gates are currently almost closed to mitigate against unexpected high inflows into the wetland system in flood events, while allowing low flows to almost continuously feed the Golf Links Road wetland system),
- A low flow channel and wetland system which exhibits the same normal water level (NWL = 16.9 m AHD) and Top of Extended Detention (TED= 17.5 m AHD), as defined by the wetland system, along this entire length, and
- An orifice outlet system and flood gate which connects the treated wetland flow back into the HVCD upstream of Narre Warren Cranbourne Road.

The low flow wetland inflow conveyance system is separated from the HVCD via a levee between the two waterways.

A levee on the north side of the Golf Links Road wetland off take channel separates the Berwick Waterways syphon outflow system (Centre Road Drain) from the Golf Links Road wetland system.

2.3 Berwick Waterways/ Homestead Road Outfall System

Discharge points from the Homestead Rd/Berwick Waterways outlet system were modified in about 2004 as part of the Golf Links Road wetland works.

The drainage system within the Homestead Road/Berwick Waterways catchment has no “natural” outfall. In low flow events, this catchment is drained via a 900 mm dia outfall connection the BTD/HVCD intersection. A flood gate on this outfall closes when there are high flows in the HVCD/BTD.

In high flow events the Homestead Road catchment drains via a syhpon system (1200 mm and 600 mm diameter syphons) to a small open drainage system located on the northern side of the Golf Link Road Wetland connection channel levee. Flows are conveyed in a small swale drain and the shallow Golf Links Road table drain, around the Golf Links Road wetland system. Flows then discharge via a culvert under Narre Warren Cranbourne Road. This minor system is the Centre Road Drain shown in Figure 1 above.

That is, this system has a separate outfall to the majority of the HVCD/BTD catchment at Narre Warren Cranbourne Road. Report 6 (Section 1) describes the Centre Road Drain system in detail.

3 Identified Issues, Constraints and Requirements

There are significant existing drainage and flooding issues relating to the Homestead Road/Berwick Waterways area. These have been identified/highlighted over many years via:

- MWC officer and consultant team experience in the area over the last few decades,
- Various technical investigations as detailed in Section 1 above, and
- The flood event of 4th February 2011

Identified issues are described briefly below.

3.1 Outfall Invert Level Constraints

Realignment of the outfall drain from the Berwick Waterways syphons as part of the Golf Link Road wetland construction has resulted in raising of downstream tail water controls. Prior to 2004 is it understood that the syphons could operate whenever water levels in the Homestead Road drainage system exceeded about 16.70 m. The new outfall drain now prevents water flow through the syphon until water levels upstream of the levees exceed about 17.30 m (the approximate existing invert level of the outfall catch drain downstream of the syphons). Therefore, in low flow events, water is required to pond back in the existing drain all the way to Centre Road before ANY outflow can occur from the Berwick Waterways area.

High flow events are also affected. The construction of the downstream Golf Link Road wetland and minor nature of the Centre Road Drain around this system severely impedes the dissipation of high flows in between the syphons and Narre Warren Cranbourne Road.

3.2 Hallam Valley Contour Drain Levee Overtopping

In a normal course of events, the Homestead Road area should be protected from flooding (from to flows within the HVCD) via the leveed system described in Section 2.1.

Flooding still occurs within the Berwick Waterways/Homestead Road area, but only from the impact of local flows, not from external HVCD flows. Flows from the local catchment have previously been estimated to result in an existing 100 Year ARI water level in the order of 18.15 m AHD.

If ANY overtopping of the HVCD or BTD levee occurs, there is the potential for the flood level within the Berwick Waterways/Homestead Road area to be much higher than the expected 100 Year ARI water level of 18.15 m AHD.

Any SWMP developed for the Berwick Waterways area must ensure that the 100 year ARI flood level due to runoff originating from DSS 0631 and 0613 must be below 18.15 m AHD, unless any rise in this flood level does not affect any existing development of adjacent property owners.

3.3 Flood Event – February 2011

Report 5 (Section 1) described and analysed the unexpected flooding which occurred within the Homestead Road area in the flood event of 4th February 2011. This analysis indicated that the event recorded could have been in the order of a 100 - 200 Year Average Recurrence Interval (ARI) event in the HVCD. Figure 2 details recorded flood levels which were well in excess of the expected 100 Year flood level in the Berwick Waterway area

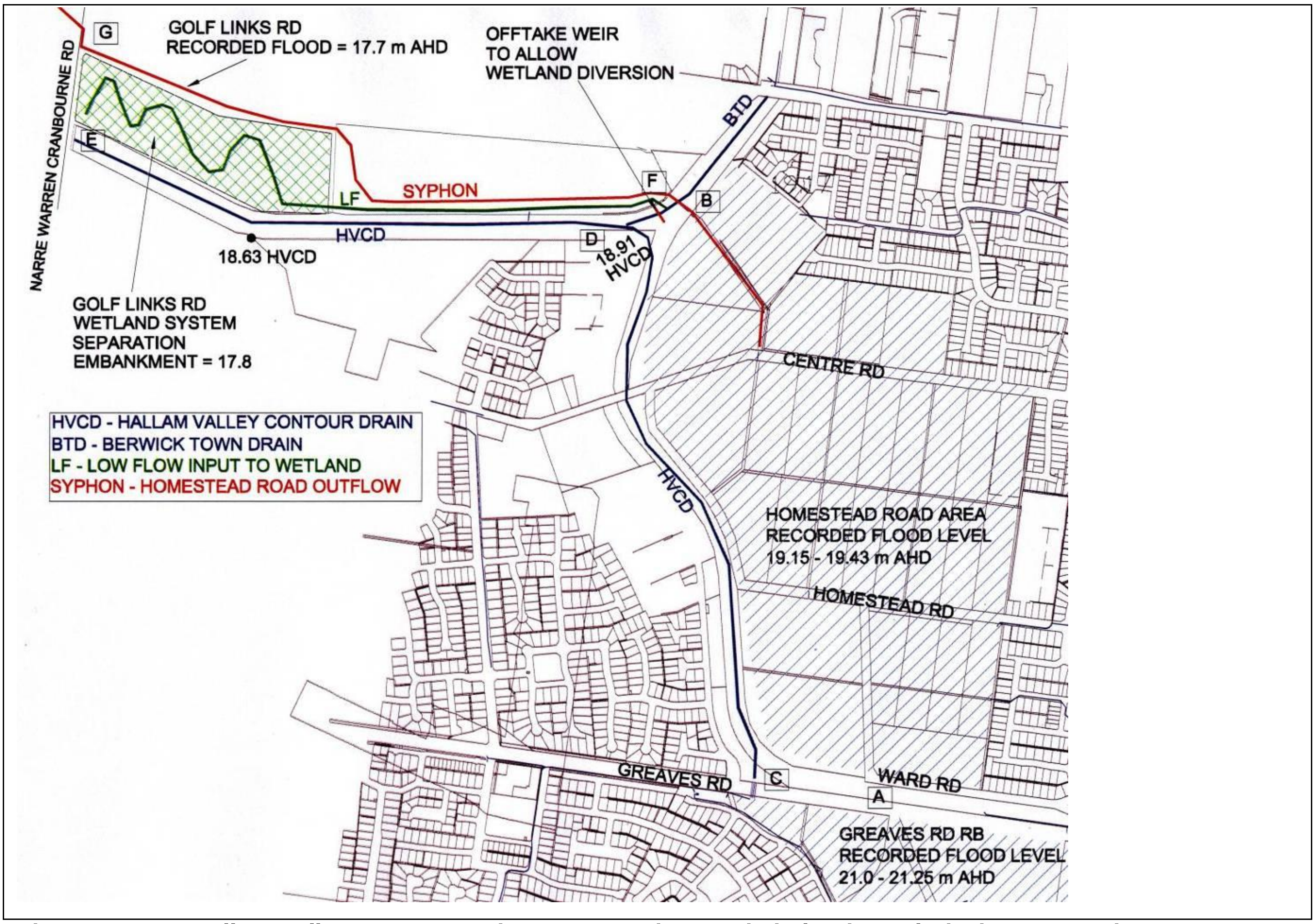


Figure 2 Hallam Valley Contour Drain System and Recorded Flood Levels (February 2011)

The analysis in the Report 5 (2011) indicated that:

- Levee overtopping probably occurred in the vicinity of Greaves Road/Ward Road and between Homestead Road and Centre Road,
- This overtopping was probably the main cause of these extreme flood levels in the Homestead Road area (although other issues such as backwater effects from the Centre Road Drain (Syphon outfall) could have contributed).

The flooding in Golf Links Road east of Narre Warren Cranbourne Road (to 17.7 m AHD) did not dissipate for many days. This is a real problem for the Berwick Waterways area. If flood levels are retained at 17.7 m AHD in Golf Links Road for extended periods outflow from upstream of the syphon will also be severely restricted for this duration.

3.4 Sewer Overflow Issues

Neil Craigie (Report 1) previously identified a main sewer pit on the west side of the Berwick Town Drain levee bank is prone to frequent surcharging. Neil Craigie specified that fencing had been erected around the pit area to keep stock and people away from the splash zone.

Neil also stated that the current drainage layout allows surcharge flows from the sewer to backflow into the Homestead Road system via the syphons, and thence into the Hallam Valley Contour Drain via the 900 mm diameter low flow pipe. This issue is required to be addressed by the SWMP.

3.5 Geotechnical Constraints

In 2009 AECOM undertook a limited geotechnical and environmental assessment mainly focusing on potential soil contamination.

The primary findings were that:

- There is some minor excavation activity in the area and stockpiles will need to be removed.
- There is some potential for acid sulphate soils which may affect excavation works associated with the wetland system. Acid sulphate investigations suggest levels are not high, but may be a concern in regard to future wetland construction requirements.
- Reusing the wetland excavation spoil on site for fill of adjacent development may require moisture conditioning and lime treatment. This will add to the construction cost but be cheaper than removing onsite soils and bringing in clean fill. MWC have previously indicated that they would allow soils removed during construction of the wetlands/retarding basin to reused by developers within the Berwick Waterways area as fill.
- Construction of the wetlands may need to occur at the same time as the development of adjacent landholdings in order to reuse the soils as soon as possible in order to reduce the risk of acid sulphate soils.

Further work is required to check the flow rates from groundwater and stormwater to fully understand the interaction between groundwater and surface water within the wetland systems.

3.6 Ecological Issues

In 2010 Ecology Partners undertook an ecological assessment of the Berwick Waterways area for the GAA. Primarily the assessment found the major issue to be the existing swamp scrub located along the road reserves and original drainage lines. The existing swamp scrub is an endangered EVC, however, it is considered easy to replace and grow.

The proposed drainage strategy as detailed in this report, is required to be assessed and commented on in regard to both requirements of implementation and the opportunities the strategy provides in regard to possibly increased ecological diversity in the area in the future. For instance, significant ephemeral marsh areas in wetland areas will offer a significant habitat opportunity for species such as Latham's Snipe. Deep pool areas will provide diversity in wet areas for other fauna. Also, significant swamp scrub (existing and future) areas can be accommodated within the wetland reserve areas.

Dwarf Galaxias **HAVE NOT** been observed within the Berwick Waterways boundary. However, this species has been observed downstream of the study area. Previously (Reports 3 and 4, Section 1) it was determined that the opportunity for Dwarf Galaxias to migrate upstream into Berwick Waterways (both now and in the future) is extremely low. This is because of the structural barriers afforded by the existing low flow outlet to the Hallam Valley Contour drain and the syhpon outlet to the Golf Links Road drainage system.

NO fish of this species have been observed in the area of interest despite the existing habitat available. It appears this structural barrier has, and will probably continue to be, an impassable barrier for this species. As such, the existing drainage system, although exhibiting good habitat, probably has a low chance of being used by this fish species.

Given the above, the SWMP has been formulated to replace any lost habitat with new wetland habitat areas. This approach is considered reasonable given the probably negligible impact on the dwarf galaxias and the significant future ecological attributes of the wetland systems proposed.

MWC have previously advised that they support the concept that the wetland portion to be located north of Centre Road to have high ecological values, and as such can be designed deeper than the normal MWC requirements (e.g. 0.5 – 1.0 m deep below NWL on average). (MWC feedback provided in a meeting with the GAA on 15th December 2010).

3.7 Existing Services

There are significant South East Water sewer and water supply assets in the Berwick Waterways area.

Water supply assets generally run along existing road systems and are generally 100 – 150 mm dia asbestos cement pipes. Minor alteration of these assets may be required for wetland/balance culvert works.

Large sewer mains are located south of Ward Road and in Centre Road and also affect the major wetland site in the north west portion of Berwick Waterways. The SWMP has been formulated to minimise the impact on these assets and to ensure access routes etc in the future.

The implications in regard to the SWMP proposals are discussed in Section 4.

All services should be accurately proven in the detailed design stage of the project and modifications made to the concept design proposed in this report as required. South East Water should be provided with a copy of this SWMP to ensure they approve of the concept in relation to impact on their assets.

3.8 Fill Levels Adjacent to HVCD

Fill levels and the form of the development adjacent to HVCD must account for adequate flood protection in regard to flooding within the HVCD. Any flood mitigation/flood protection mechanism for protection of Berwick waterways must not increase flood levels on existing development to the west of HVCD.

KLM Spatial understand that MWC have advised Casey that:

- Presently 50 yr protection (with some freeboard) is afforded to Berwick Waterways and MWC indicate that this is appropriate with existing Low Density Residential Zoning,
- If Berwick Waterways is to be rezoned to allow residential normal density use then the HVCD levee (or equivalent system) our area must ultimately provide 100 Year ARI flood protection with 600 mm freeboard or 500 yr protection whichever is the highest.

3.9 Subdivisional Drainage System Design

In general, although large and mutli-purpose in design intent (i.e. incorporating more than just a traditional WSUD role), the wetland and flood storage area is similar to many existing systems which have been shown to function successfully. However, some technical challenges must be clearly understood in regard to the subdivisional pipeline design.

One issue within Berwick Waterways itself is to ensure all piped drainage systems can in fact outfall to the wetland system incorporating either a 1 in 600 slope (minimum , as previously agreed to be MWC and Council) or adequate pipe slopes for flushing. Care must be taken to ensure:

- Pipes are not fully drowned out at the wetland,
- Pipes incorporate adequate cover which may impact fill level requirements in the final subdivision design.

Notwithstanding the above, existing MWC DSS pipelines north of Centre Road incorporate relatively invert levels (16.3 m AHD and 16.6 m AHD respectively). These levels are well below the existing outfall invert level at the downstream end of the syphon outfall. Therefore all effort should be made in the SWMP to minimise the outfall invert level conditions to ensure minimal ponding in these existing systems.

3.10 Council Requirements

In 2010 Casey City Council reviewed Report 3 (Section 1) and raised requirements in regard to the development of Berwick Waterways.

It is understood that, in general Council require:

- Roads and active parkland to be above the existing and future 100 Year flood level,
- Any roads adjacent to the Hallam Valley Levee to be raised to match the levee protection works proposed by MWC,
- That proposed to gravity outlet for the wetlands into the existing MWC syhpon and low flow outlet located at the north western edge of Berwick Waterways (i.e. the top end of the wetland system) must be shown to be feasible,
- That the strategy must account for existing road levels and also how development will drain towards the road pipe network,
- That, given the flood event of 4 and 5th February 2011, proposals to be designed to ensure protection any future development outside the HVCD leveed system from flooding.
- That a pipe full velocity of above 1m/s must be achieved in subdivision pipes. In 2011 SWS confirmed that in 2009 agreement was obtained from MWC and Council in a workshop 1/600 pipe grades are suitable within Berwick Waterways provided this velocity can be achieved. This was part of the strategy to minimise development fill costs. Previous preliminary calculations showed this velocity can be achieved.

As previously confirmed by MWC and council, all pipes of less than 60 ha catchment will ultimately become council's responsibility. Stormy Water Solutions understand that Moremac Property Group have obtained agreement from Council that they will be responsible for the wetland reserve area not encompassed by a wetland system such as any structural retaining walls and surrounding grassed and landscaped areas etc.

3.11 Melbourne Water Requirements

In 2010 MWC reviewed Report 3 (Section 1) and raised requirements in regard to the development of Berwick Waterways.

- o MWC require the wetland system south of Centre Road (apart from sediment pond zones) to meet current water quality depth requirements to achieve 80% vegetation coverage.

- o MWC require the wetland NWL to be at least 15 metres to road reserves and 20 m to lot boundaries.

- o All final DSS pipelines within Berwick waterways will be Council's, and as such may be required to be Rubber Ring Jointed.

- o MWC will require upfront construction of the wetland north of Centre Road and the HVCD levee protection works.

In general, MWC have previously indicated that they prefer to get the engineering and town planning correct and in place prior to organizing financing issues

In addition, as per current MWC requirements, the combined wetland systems must ensure:

- o The 100 Year ARI velocity over the wetland areas are less than 0.5 m/s,
- o The 3 month ARI velocity over the wetland areas are less than 0.05 m/s,
- o The Regular Inundation Check relating to wetland vegetation health is met (considering at least 10 years of continuous rainfall and evaporation data), and
- o The Excessive Inundation Check relating to wetland vegetation health is met (considering at least 10 years of continuous rainfall and evaporation data).

Recently MWC confirmed that they have no objection to the drainage concept proposed with a number of water bodies connected by an underground drainage system provided:

- Hydrologic and hydraulic calculations are undertaken that satisfy Melbourne Water's concern for events greater than the 1 in 100 year event.
- Functional drawings are prepared showing that the water bodies adequately address the stormwater quality requirements and are readily maintainable with suitable sediment dry out areas and bypass facilities.
- Arrangements are made with Council for the maintenance of the walling around the water bodies as well as the open space areas proposed.
- An acceptable staging plan is developed for both the stages within the precinct as well as the drainage works proposed.

4. Proposed Stormwater Management Plan

Stormy Water Solutions drawing Set 1411 details the SWMP developed for Berwick Waterways.

The concept designs detailed in the SWMP have been fully considered in regard to their applicability. Existing site levels and existing (and future) drainage system invert levels have been used to set normal water level etc. This has been done to ensure all elements can be constructed and will not be constrained by outfall invert levels, topography, site areas etc . The intent of the plan is to ensure that the developers, MWC and Council are clear in regard to the feasibility and requirements of the drainage system. Notwithstanding the above, the functional and detailed design process should ensure all design levels are reviewed against a detailed site survey

The vision for future development in Berwick Waterways provides for a wetland/pool floodplain development concept.

Following on from the original concept developed over the last ten years (See Section 1 reports), the SWMP provides for:

- creation of a wetland/pool system interlinked with balance culvert systems, for management of stormwater quality from existing developed areas in both the Homestead Road and O'Sheas Road Drainage Scheme catchments;
- incorporation of the existing Homestead Road Retarding Basin as part of this system by removal of the existing embankments;
- use of the airspace above the water surface area and over the lands in the balance of the drainage reserve areas for flood storage purposes;
- wetlands and balance culverts to replace all existing open drains within the study area, thereby resolving existing open drain performance and maintenance concerns;
- restoration of natural waterway and floodplain habitat values throughout the proposed reserve area;
- provision of a high value recreational corridor which will complete a missing link between the floodplain developments south of Greaves Road and the Golf Link Road wetland system.

This SWMP suggests the following to provide direction in regard to the subdivision and development design going forward:

- Preliminary locations for major pipes ,
- The location of the outfalls to the proposed wetlands and sediment ponds,
- The sediment pond extents at the wetland outfall location, and
- A fill level scenario within Berwick Waterways to ensure adequate catchment delineation and appropriate lot grades.

The SWMP detailed in SWS drawing set 1411 is described below.

- Four interlinked wetland systems (W1, W2, W3, W4) are proposed. These will act as one combined stormwater treatment wetland system in low flows and one flood storage system in extreme events.

- The NWL of the combined wetland system (W1 – W4) is 16.9 m AHD.

- The TED in the combined wetland system (W1 – W4) is 17.1 m AHD. This level has been reduced from previous proposals to maximise flood storage within Berwick Waterways.

- The total detention time for the "combined" wetland system is 72 hours

- The Centre Road Drain outfall constraints as detailed have been considered as part of the development of the SWMP. As such, implementation of outfall augmentation works as detailed in Section 6 has been further developed as part of the SWMP and are detailed in SWS Drawing 1411/2.

- 300 l/s will be diverted into the wetland system from the O'Sheas Road DSS to provide stormwater treatment for this catchment.

- Modifications to the existing HVCD 900 mm dia flood gated connection and the existing syphon as detailed in SWS Drawing 1411/5 are proposed to ensure adequate low flow and high flow controls within the Berwick Waterways wetland and flood storage system. This includes provision of a pump pit in the HVCD levee for wetland maintenance and emergency use if required.

- Twin 3000 mm by 1500 mm balance culverts are proposed to connect W2 and W3 and W3 and W4. These will ensure a minimal head loss though the system in high flow events and a maximum 100 year water level in W2, W3 or W4 of no more than 18.15 m AHD either when the wetland is full, or when the wetland system if filling (i.e. when inflows and head losses though the balance culvert system are high.

- One 3000 mm by 1500 mm balance culvert is proposed between W1 and W2. This will ensure a maximum 100 year water level in W1 no more than 18.5 m AHD (600 mm below the existing Homestead Road development fill level (19.2 m AHD)) either when the wetland is full, or when the wetland system is filling

- The preliminary fill levels adjacent to HVCD have been set given the extreme flood levels detailed in Report 5 (Section 1). Although not addressed in detail in this SWMP, it is assumed that design and investigation work in the short term will result in drain and levee augmentation proposals in line with the recommendations in Report 5. Development of Berwick Waterways cannot occur until MWC and Council are satisfied in regard to the flood protection afforded by the HVCD remodelling works. Notwithstanding the above, preliminary flood protection and extreme flow considerations are as detailed in drawing 1411/3.

- Preliminary fill levels have been specified as detailed in SWS Drawing 1411/1 and 1411/3. Internal catchments can be kept relatively small to allow for small pipe runs which minimised pipe cover and development fill requirements. The intent of the catchment delineation should be adhered to throughout the design process.

- The preliminary fill levels detailed allow for:
  - Minimum Road levels surrounding W1 to be a minimum of 18.5 m AHD (the existing level minimum level of Homestead Road to the north is 18.8 m AHD and it is expected this road level will be maintained)
  - Minimum Road levels surrounding W2, W3 and W4 to be a minimum of 18.15 m AHD with at least part of the road adjacent to these systems at least 18.45 m AHD,
  - Minimum lot levels surrounding W1 to be a minimum of 19.1 m AHD (the existing level minimum lot level of existing lots to the east is 19.2 m AHD), and
  - Minimum lot levels surrounding W2, W3 and W4 to be a minimum of 18.75 m AHD.

- Fill levels are shown to meet the intent of any lot being accessible to an emergency exit route which raises away from the drainage low point (i.e. away from the wetland and balance pipe system).

- Four Sediment Ponds S1, S2, S3 and S4 are proposed be located prior to the outfall of any existing pipe to any wetland system. These are specifically proposed to address sediment originating from existing developed areas. Apart from S1, all NWL's are at the wetland NWL. Higher NWL's could not be accommodated given existing outfall invert levels (which are at or below 16.9 m AHD).

- Four Wetland sump vortex systems (at least) are proposed be located prior to the outfall to any wetland system. These are specifically proposed to address sediment originating from Berwick Waterways itself.

- Sediment Ponds S3 and S4 form the outfalls for the existing MWC DSS pipelines north of Centre Road. These outfalls incorporate relatively invert levels (16.3 m AHD and 16.6 m AHD respectively). As such, the pipe outfalls will be slightly drowned at these locations.

- Sediment Ponds S2 forms the outfall for the O'Sheas Road DSS diversion connection. As such, this sediment pond specifically addressed sediment originating from this external catchment. The low invert level of the existing pipeline upstream of the diversion point results in the diversion pipe requiring an invert level at the NWL of the wetland system = 16.9 m AHD.

- The 900 mm dia sewer main in the vicinity of Ward Road has an obvert level in the order of 15.8 m AHD in the vicinity of the proposed O'Sheas Road DSS low flow off take. This is well below proposed invert levels of this diversion pipe.

- The 900 mm dia sewer main in Centre Road has an obvert level in the order of 15.9 m AHD in the vicinity of the balance culvert crossing proposed. This is close to the proposed invert level of the balance culvert system at this location. Concrete capping of the sewer main may be required at this location.

- Within the Wetland 4 site all effort has been made to minimise areas where the wetland system covers the existing sewer alignment. Where this does occur, the wetland is proposed to be shallow marsh to ensure the base of the wetland well above the level of the sewer.

- The golf Links road outfall wetland outfall wetland (downstream of the syphon) has bunding on its northern edge to 17.4 m AHD to minimise impact of any sewer overflows on the wetland systems.

Unlike many wetland systems, it is proposed to incorporate a number of wetland edge styles in the development. "Soft" edges denote traditional vegetated edges with safety benches. "Built", "Urban" and "Park" edges denote a more structural style as detailed in SWS drawing set 1411/4. The two later styles incorporate walling around the wetland edge.

Typical longitudinal sections of the major pipelines should be formulated at the functional design stage of the project (given final fill proposals) to ensure adequate cover and invert level definitions going forward into the detailed design stage of the project.

MWC have confirmed that they have no objection to the drainage concept proposed with a number of water bodies connected by an underground drainage system provided:

- Hydrologic and hydraulic calculations are undertaken that satisfy Melbourne Water's concern for events greater than the 1 in 100 year event.
- Functional drawings are prepared showing that the water bodies adequately address the stormwater quality requirements and are readily maintainable with suitable sediment dry out areas and bypass facilities.

- Arrangements are made with Council for the maintenance of the walling around the water bodies as well as the open space areas proposed.

Once MWC approval for this SWMP is obtained, an acceptable staging plan should be developed for both the stages within the precinct as well as the drainage works proposed. This is a requirement of MWC.

In addition to the above, the SWMP detailed in this report disseminate the “usual” requirements for development of a SWMP in Melbourne (as usually required by MWC for (say) DSS development). However, due to the levee protection aspect of this site, consideration of extreme events in excess of the 100 Year ARI event magnitude (both within and external to Berwick Waterways) is required.

In this way emergency management procedures and risks associated with very extreme events can be accounted for both within the design process, and within the use of the area in the future.

5. Hydrological Modelling

Hydrological Modelling using the RORB model was developed for this study by SWS to estimate flood flows

5.1 RORB Model Description

Figure 2 details the RORB model setup for the post development catchment conditions. Tables 1 and 2 detail the tabulation of the post development RORB model subareas and reaches.

Table 1 RORB Model Sub Area Definition
(Berwick Waterways Development Denoted in red)

Sub Area	Area (ha)	Area (km^2)	Fraction Imperviousness (With Development)		
A	61.2	0.61	0.60		
B	54.3	0.54	0.60		
C	33.6	0.34	0.60		
D	45.8	0.46	0.60		
E	27.7	0.28	0.60		
F	24.2	0.24	0.60		
G	45.4	0.45	0.60		
H	31.3	0.31	0.60		
I	7.4	0.07	0.60		
J	4.5	0.05	0.60		
K	7.9	0.08	0.60		
L	9.0	0.09	0.60		
M	5.9	0.06	0.75		
N	5.1	0.05	0.60		
O	4.8	0.05	0.60		
P	4.6	0.05	0.65		
Q	6.0	0.06	0.65		
R	6.7	0.07	0.65		
S	3.0	0.03	0.65		
T	5.0	0.05	0.70		
U	5.6	0.06	0.65		
V	7.5	0.08	0.75		
W	7.4	0.07	0.75		
X	2.3	0.02	0.10		
Y	6.1	0.06	0.85		
Z	9.5	0.10	0.60		
AA	8.1	0.08	0.60		
AB	4.2	0.04	0.65		
AC	3.5	0.04	0.70		
AD	12.5	0.13	0.60		
AE	12.5	0.13	0.60		
AF	9.0	0.09	0.60		
AG	4.8	0.05	0.60		
AH	10.2	0.10	0.85		
AI	8.8	0.09	0.10		
Total	**505.4**	**5.05**	**0.61**		
O'Sheas Road DSS			323.5	ha	**0.60**
Homestead Road Catchment			181.9	ha	**0.62**

Table 2 RORB Model Reach Definition

Reach	Length (m)	Length (km)	Reach Type	Slope (%)
			(With site Development - 5 yr and 100 yr)	(Approx)
1	960	0.96	Piped	1.0%
2	605	0.61	Piped	1.0%
3	260	0.26	Piped	1.0%
4	350	0.35	Piped	1.0%
5	300	0.30	Piped	1.0%
6	740	0.74	Piped	1.0%
7	420	0.42	Piped	1.0%
8	300	0.30	Piped	1.0%
9	910	0.91	Piped	1.0%
10	680	0.68	Piped	1.0%
11	590	0.59	Piped	0.2%
12	270	0.27	Piped	0.2%
13	280	0.28	Piped	0.3%
14	200	0.20	Piped	0.3%
15	160	0.16	Piped	0.3%
16	160	0.16	Piped	0.3%
17	240	0.24	Piped	0.3%
18	130	0.13	Piped	0.3%
19	170	0.17	Piped	0.2%
20	170	0.17	Natural	
21	210	0.21	Piped	0.3%
22	250	0.25	Piped	0.3%
23	325	0.33	Piped	0.1%
24	270	0.27	Piped	0.2%
25	140	0.14	Piped	0.2%
26	160	0.16	Piped	0.2%
27	215	0.22	Natural	
28	145	0.15	Piped	0.1%
29	150	0.15	Piped	0.1%
30	260	0.26	Piped	0.2%
31	200	0.20	Piped	0.2%
32	130	0.13	Natural	
33	330	0.33	Piped	0.2%
34	230	0.23	Piped	0.2%
35	140	0.14	Natural	
36	200	0.20	Piped	0.1%
37	170	0.17	Piped	0.1%
38	300	0.30	Piped	0.2%
39	100	0.10	Piped	0.2%
40	150	0.15	Piped	0.2%
41	220	0.22	Piped	0.2%
42	90	0.09	Piped	0.2%
43	260	0.26	Piped	0.2%
44	180	0.18	Piped	0.2%
45	250	0.25	Natural	
46	520	0.52	Piped	1.0%
47	420	0.42	Piped	1.0%
48	150	0.15	Piped	1.0%
49	190	0.19	Piped	1.0%
50	200	0.20	Natural	
51	300	0.30	Piped	0.2%
52	140	0.14	Natural	

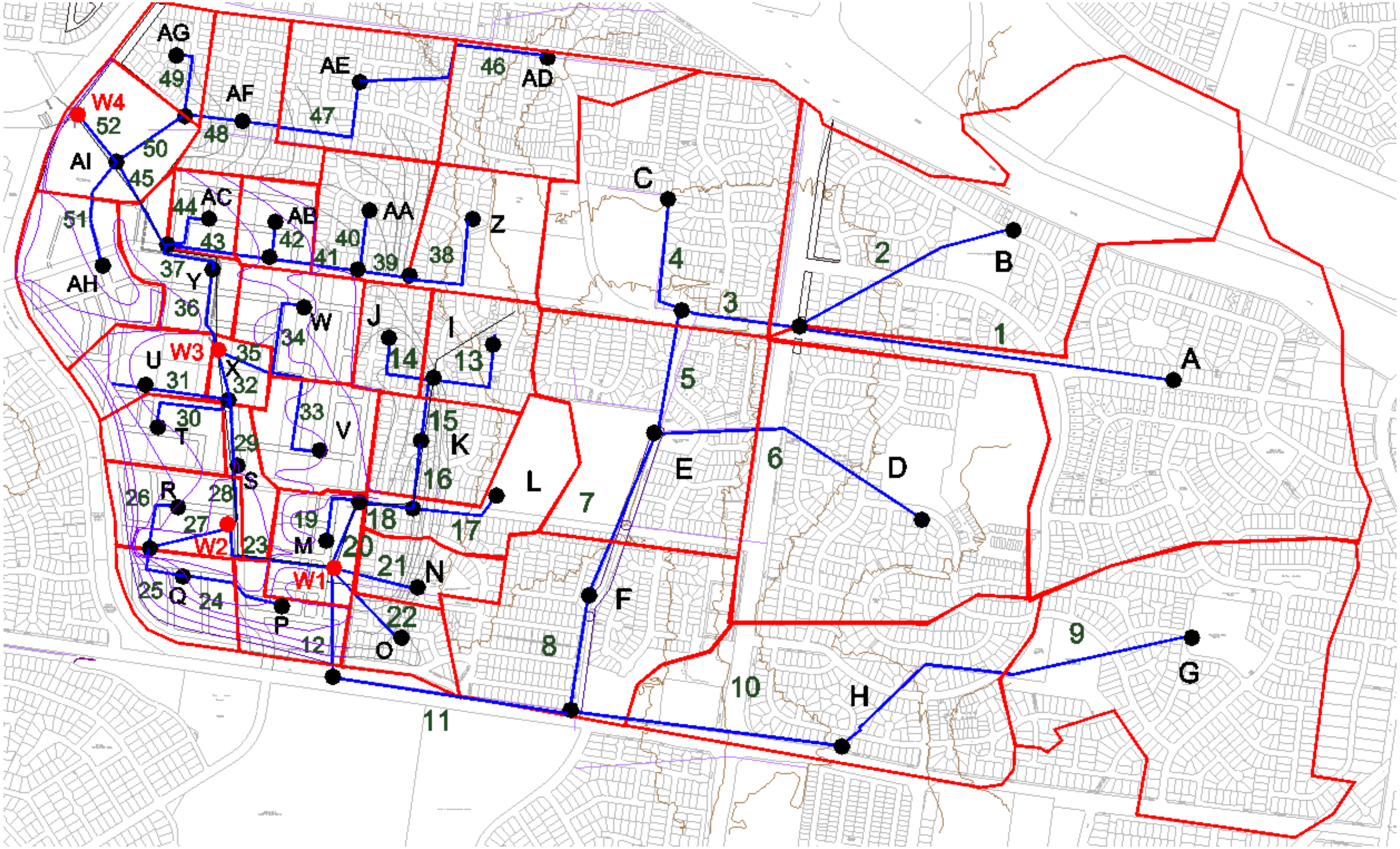


Figure 2 Berwick Waterways RORB Model – SWS 2014

MWC have developed a regional parameter set for the south east areas of Melbourne. This is the parameter set adopted for this study as detailed below.

- $K_c = 1.53 \times A^{0.55} = 2.1$ (K_c based on Homestead Road catchment area as the diversion from O'Sheas Road DSS is more about flood volume rather than peak flow . Only 0.3 m^3/s diverted from O'Sheas Road DSS)
- m = 0.8,
- IL = 10 mm (except for 5 mm in the 500 year ARI event)
- Pervious area runoff coefficient = 0.8 (500 Year), 0.6 (100 Year ARI), 0.4 (10 year ARI), and 0.2 (1 year ARI)
- Rainfall temporal pattern filtering – (ON, assumed),
- Uniform Temporal pattern – ON, and
- Models run with Siriwardena and Weinmann Area reduction factor – ON

The RORB model incorporating the MWC regional parameter set, with the areal reduction factor included, produced design flows which compared well with rational method estimates and the DEPI flood regression curves.

5.2 Berwick Waterways Stage/Storage/Discharge Relationship

The Stage/Discharge relationship at the syphon outlet from Berwick Waterways was determined given consideration of:

- 0.4 m^3/s (approx) maximum outflow through the existing 900 mm dia flood gated connection to the HVCD up to a HWL = 17.6 m AHD (as previously defined by Neil Craigie),
- The 600 mm and 1200 mm syphons operating as culvert under outlet control,
- Tailwater levels downstream of the syphon assuming full implementation of Golf Links Road Master Plan as defined in SWS Drawing 1411/2. A rating curve was developed using Hec Ras based on implementation of these works. This is a crucial aspect to the design. **If the downstream works do not occur flood levels within Berwick Waterways will be higher than predicted.** The downstream work optimise the effectiveness of the upstream flood storage in Berwick Waterways but also address existing flooding issues along Golf Links Road between the syphon and Narre Warren Cranbourne Road.

Table 3 details the Stage /Discharge Relationship developed.

Table 3 Berwick Waterways Stage/Discharge Relationship

Head Water Level (m AHD)	Flow in 1200 mm dia Syphon (m3/s)	Flow in 600 mm dia Syphon (m3/s)	Flow in 900 mm dia to HVCD (m3/s)	Total Outflow (m3/s)
16.9	0	0	0	0.00
17.1	0	0	0.063	0.063
17.15	0.4	0.08	0.4	0.90
17.23	0.8	0.165	0.35	1.35
17.47	1.3	0.25	0	1.50
17.67	1.7	0.33	0	2.00
18.13	2.2	0.43	0	2.60
18.44	2.5	0.5	0	3.00
18.90	2.9	0.58	0	3.50
19.37	3.3	0.665	0	4.00

The Stage/Storage relationship for the flood storage provision above W1, W2, W3 and W4 within Berwick Waterways was determined given consideration of the airspace above the proposed wetland systems as detailed in SWS Drawing 1411/1. Table 4 details the Stage /Storage Relationship developed.

Table 4 Berwick Waterways Stage/Storage Relationship

Water level (m AHD)	Area (m^2)	Average Area (m^2)	Delta h (m)	Volume (m^3)	Cumulative Volume (m^3)	Water level (m AHD)
16.9	82000				0	16.9
17.1	86700	84350	0.2	16870	16870	17.1
17.6	99000	92850	0.5	46425	63295	17.6
18.15	157000	128000	0.55	70400	133695	18.15
18.45	193300	175150	0.3	52545	186240	18.45
18.75	270400	231850	0.3	69555	255795	18.75
19	465000	367700	0.25	91925	347720	19
19.5	663000	564000	0.5	282000	629720	19.5
20	779000	721000	0.5	360500	990220	20

The Stage/Storage/Discharge relationship for the total system is detailed in Table 5 below.

Table 5 **Berwick Waterways Stage/Storage/Discharge Relationship**

Stage (m AHD)	Storage (m^3)	Outflow (m^3/s)
16.90	0	0.00
17.10	17000	0.063
17.15	20000	0.90
17.23	30000	1.35
17.47	50000	1.50
17.67	70000	2.00
18.13	130000	2.60
18.44	185000	3.00
18.90	300000	3.50
19.37	550000	4.00

5.3 Diversion Model

The RORB model assumes a maximum diversion of 0.3 m^3/s at the node upstream of Reach 12 into Berwick Waterways. All remaining flow above this value does not enter Berwick Waterways (from O'Sheas Road DSS) and continues west to the HVCD.

5.4 Design Flow Estimates

Table 6 details the RORB results.

Table 6 **RORB Results**

Location	500 Year ARI Design Flow	100 Year ARI Design Flow	10 Year ARI Design Flow	1 Year ARI Design Flow
Total Inflow to W1	13.0 m^3/s (15 min)	7.9 m^3/s (25 min)	3.9 m^3/s (2 hr)	2.0 m^3/s (2 hr)
Total Inflow to W2	16.2 m^3/s (20 min)	9.8 m^3/s (20 min)	5.0 m^3/s (2 hr)	2.4 m^3/s (2 hr)
Total Inflow to W3	20.1 m^3/s (25 min)	12.4 m^3/s (1 hr)	6.4 m^3/s (2 hr)	2.9 m^3/s (2 hr)
Inflow to W4 at Centre Road	20.1 m^3/s (25 min)	12.4 m^3/s (1 hr)	6.4 m^3/s (2 hr)	2.9 m^3/s (2 hr)
Inflow to W4 - Total	30.5 m^3/s (1 hr)	19.2 m^3/s (1 hr)	9.4 m^3/s (2 hr)	4.2 m^3/s (4.5 hr)
Outflow from W4	2.9 m^3/s (30 hr) WL = 18.33 m AHD*	2.4 m^3/s (30 hr) WL = 17.94 m AHD*	1.7 m^3/s (12 hr) WL = 17.54 m AHD*	1.4 m^3/s

** Note : Flood levels are accurate at W4, but increase upstream as detailed below.*

The peak inflow estimates have been used to assess wetland velocity criteria etc (See Section 9.5).

5.5 Flood Levels in Wetland Cells Given Balance Culvert Considerations

It should be noted that the water levels in Table 6 are applicable to W4 only. Levels rise in upstream wetlands due to balance culvert losses. Depending on the duration of the flood event, upstream water levels can be dependent on:

- the high tail water level in W4 dominating backwater effects when the total wetland system, (W1, W2, W3 and W4) is full, or
- The capacity of the balance culvert system producing high water levels in upstream wetlands due to the high flows expected though the culvert systems as the wetland system is filling.

As such, using the assumption that each balance culvert is acting under outlet control, various analysis were cried out to determine the maximum water level expected in each wetland. A summary of this analysis is detailed in Table 7 below. It should be noted that this is a conservative analysis, as the flood retardation effect in W1, W2 and W3 for all storage above the water level calculated in W1 has not been accounted for.

Table 7 Maximum Water Levels Expected in Each Wetland Cell

	500 Year ARI event		100 Year ARI event		10 Year ARI event	
Location	Peak WL (m AHD)	Critical duration of the Peak Water Level	Peak WL (m AHD)	Critical duration of the Peak Water Level	Peak WL (m AHD)	Critical duration of the Peak Water Level
WL4/S3/S4	18.33	30 hr	17.94	30 hr	17.54	12 hr
WL3	18.49	2 hr	17.96	30 hr	17.63	25 min
WL2	18.55*	2 hr	18.00	25 min	17.67	25 min
WL1/S1/S2	18.55*	2 hr	18.50	25 min	17.79	25 min

** WL's approximate only. Above 18.45 road drowned out and water level balances out as system acts as one retarding basin*

Minimum Road levels surrounding W1 are proposed to be a minimum of 18.5 m AHD and lot levels set at 19.1 m AHD (minimum). As such, the proposed fill levels around this asset allows for 100 year ARI protection for roads and this protection plus 600 mm freeboard for lots.

Minimum Road levels surrounding W2, W3 and W4 are proposed to be a minimum of 18.15 m AHD and lot levels set at 18.75 m AHD (minimum). As such, the proposed fill levels around these assets allows for 100 year ARI protection for roads and this protection plus 750 mm freeboard for lots. In the 500 Year ARI event in the order of 400 mm (max) may inundate part of the road system around these assets. However, there should be around 200 mm freeboard to the lots in the 500 Year ARI event.

The above analysis considers only the calculated RORB flood storage effects with the system acting as one unit and the effect of the balance culvert system losses between each wetland cell. At this concept design stage of the project, this is considered to give a reasonable estimation of expected flood levels within Berwick Waterways. Levels obtained can be used to ensure that the subdivisional layout proposed at this time can ensure adequate 100 Year flood protection to major assets and to ensure relative safety of the development population in the 500 Year ARI event. However, once the final form and levels of the development are set, it is recommended that a two dimensional, unsteady state model be used to confirm the flood levels calculated above.

5.6 Flood Levels in Wetland Cells Given Possible Syphon Blockage

The analysis in Section 6.5 assumes no blockage of the syphon outlet at the HVCD.

MWC inspection and maintenance regimes should ensure this system is routinely clear of blockage. However, it is considered prudent that this SWMP consider the possibility of system blockage. Tables 8 and 9 below summarise the results of this investigation.

Table 8 100 Year Water Levels Considering Blockage of the Syphon Outlet

	100 Year ARI event- 0% Blockage		100 Year ARI event - 50% blockage		100 Year ARI event - 90% blockage	
Location	Peak WL (m AHD)	Critical duration of the Peak Water Level	Peak WL (m AHD)	Critical duration of the Peak Water Level	Peak WL (m AHD)	Critical duration of the Peak Water Level
WL4/S3/S4	17.94	30 hr	18.25	36 hr	18.83	72 hr
WL3	17.96	30 hr	18.27	36 hr	18.84	72 hr
WL2	18.00	25 min	18.27	36 hr	18.85	72 hr
WL1/S1/S2	18.50	25 min	18.50	25 min	18.86	72 hr

Table 9 500 Year Water Levels Considering Blockage of the Syphon Outlet

	500 Year ARI event - 50% blockage		500 Year ARI event - 70% blockage		500 Year ARI event - 90% blockage	
Location	Peak WL (m AHD)	Critical duration of the Peak Water Level	Peak WL (m AHD)	Critical duration of the Peak Water Level	Peak WL (m AHD)	Critical duration of the Peak Water Level
WL4/S3/S4	18.62	36 hr	18.85	72 hr	19.10	72 hr
WL3	18.66	36 hr	18.89	72 hr	19.10	72 hr
WL2	18.7*	36 hr	18.90	72 hr	19.10	72 hr
WL1/S1/S2	18.7*	36 hr	18.9*	72 hr	19.10	72 hr

** WL's approximate only. Above 18.45 road drowned out and water level balances out as system acts as one retarding basin*

As detailed:

- If 50% blockage of the syphon occurs roads may by inundated in the order of 100 mm (max) in the 100 Year event and 550 mm in the 500 year event. Lots should still remain flood free in both events.
- If 70% blockage of the syphon occurs roads may by inundated in the order of 750 mm in the 500 year event. Lots may be inundated in the order of 150 mm.

- If 90% blockage of the syphon occurs roads may by inundated in the order of 710 mm (max) in the 100 Year event and 950 mm (max) in the 500 year event. Lots may be inundated in the order of 100 mm in the 100 Year ARI event and 350 mm in the 500 Year ARI event.

It is considered that 50% blockage of the syphon can be accommodated by the proposed development scenario without a significant increase in cost or inconvenience to the population (which would have otherwise occurred in a 100 year ARI or 500 year ARI event). However, if greater blockage occurs, inconvenience, and more particularly safety of the population does become a concern.

Given the above, Stormy Water Solutions recommends that provision be made at the syphon outlet to construct a balance pipe between the upstream end of the syphon and a new pump pit in the crest of the HVCD levee (SWs Drawing 1411/5). In this way, if required during a flood event, the MWC maintenance crew could place a pump in the new pit (accessible on the levee crest) and pump flows into the HVCD. The maximum pump rate should be a high as possible, but ideally over 1.2 m^3/s (50 % of the 100 Year outflow flow rate without blockage).

The above provision should be included in both Council's and MWC's emergency management plan for the area.

Any emergency management plan should also incorporate consideration of emergency evacuation routes. Preliminary routes, based on the proposed fill levels, are detailed in 1411/3.

6. Centre Road Drain Outfall System Augmentation

Neil Craigie (Report 1) indentified the 900 mm diameter flood gated outfall as a means for at least partially resolving the problem of impeded drainage from the Homestead Road DSS system. Particularly the attenuation of flows in the Hallam Valley Main Drain via the major wetlands and open water bodies in the Ti Tree Creek Retarding Basin system may result in significantly delay and attenuates peak flows from developments south of Greaves Road. However the 900 mm diameter pipeline will not be fully functional in times of significant flooding. The backflow device will prevent any discharge whenever water levels within the levee-banked main drain rise higher than those in the Homestead Road wetland pondage system. As such, relying on the 900 mm dia connection to HVCD was not seen as a complete solution to the existing outfall and flood dissipation issues above.

The 2012 SWS work (Report 6) recommended implementation flood mitigation works between the syphon outfall and Narre Warren Cranbourne Road. The works included:

- Extending the Golf Links Road wetland (and standing NWL) to the existing syphon pipes to ensure free outflow from the Berwick Waterways syhpon system when the water level in the upstream wetland is greater than NWL,
- Reducing the existing normal water level of the Golf Links Road wetland system from 16.9 m AHD to 16.7 m AHD to ensure independent operation of both the Berwick waterways wetland system and the Golf Links Road wetland system, and
- Construct a high flow outlet from the Golf Links Road wetland to the Narre Warren Cranbourne Road culverts designed to decrease the 100 Year ARI water level upstream of this road. This than allows the 100 Year tail water level at the syphon outlet to be substantially reduced and maximised outflow from Berwick Waterways in this event.

The primary aim of these mitigation works was to mitigate flood effects just upstream of Narre Warren Cranbourne Road. However an additional benefit would be to allow free outflow from Berwick Waterways through the syphon at any level above 17.1 m AHD and significant and outflows though the syphon in extreme events, without causing excessive water levels upstream of the syphon.

This proposal has been adopted as required by this SWMP as detailed in 1411/2.

It should be noted that not all of the works detailed may be required to obtain outfall from Berwick Waterways in the medium term. Berwick Waterways could develop if:

- The existing wetland extension to the BTD outlet pipes was extended slightly to the Syphon outlet,
- The outflow from the BTD was reduce to just a trickle feed only,
- The Golf links road wetland NWL was dropped to 16.7 m AHD via minor outlet works and cutting low flow connections though the ephemeral marsh and shallow marsh zones in the wetland, and

- The twin 900 by 2400 Golf Link Road high flow outlet was constructed as detailed in Drawing 1411/2.

These works could be completed by the developers of Berwick Waters in the short term.

The remainder of the wetland works (enlarging the existing outfall wetland channel, removing the northern levee as detailed) could be undertaken as deemed necessary by MWC or as part of the requirement on development for the land located north of the Golf Links Road Wetland extension.

7. O'Sheas Road DSS Diversion

The O'Sheas Road Drainage Scheme water (336 ha catchment) has been included in the SWMP. At present this catchment area receives virtually no water quality treatment before it is discharged to the Hallam Valley Contour Drain and then to Eumemmerring Creek and Port Phillip Bay. The drainage scheme outfall currently directs all flows up to about 1.5 m^3/s along the north side of Greaves Road by pipeline to the Hallam Valley Contour Drain. Higher flows pass south through culverts under Greaves Road and into the Ti Tree Creek Retarding Basin wetland and pondage system. The Homestead Road area presents the only viable opportunity for any stormwater treatment of this fully developed catchment.

The proposal provides for all flows up to 0.3 m^3/s to be directed from the Greaves Road pipeline into the Berwick Waterways floodplain wetland system. Sediment Pond S2 will be the primary collection point for course sediment from the O'Sheas Road DSS catchment. Preliminary calculations indicate that this diversion rate will have minimal implications with regard to increasing the cost of the Berwick Waterways wetland systems. However, will aid in maximising waterbody turnovers and hence minimise risk in regard to ongoing water quality issues.

The inflow structure from the O'Sheas Rd DS pipeline outfall in Greaves Road must:

- Allow a low flow inflow of 0.3 m^3/s before bypass of the O'Sheas Road flow east to HVCD, and
- Limit inflow to about 0.3 m^3/s in the 100 year ARI event in HVCD (MWC advice regarding the declared flood level in the Hallam Valley Contour Drain indicates that the 100 year ARI flood level on the north side of Greaves Road is 20.70 m)

The existing Greaves Road pipeline terminates as a 1500 mm diameter pipe at invert of 17.40 m as shown on the O'Sheas Rd DS Stage 13 as-constructed plans. The proposed diversion structure and 450 mm extension pipelines are to be constructed on the end of the existing 1500 mm dia pipeline and are shown in SWS Drawing 1411/5. The 450 mm control pipe will limit outflows to an average of 0.3 m^3/s over the water level range. The calculations in Appendix A detail the concept design of this diversion pipe.

MWC have indicated that the cost of the connection pipe for O'Sheas Road DSS to Wetland W1 will be met by MWC. MWC had also previously provided advice in regard to this issue in a meeting with the GAA on 15th December 2010. At this meeting MWC agreed in principal to the concept of apportioning the DSS (or drainage strategy) water quality funding between O'Sheas Road DSS, and the Homestead Road Extension DSS (Berwick Waterways). Apportioning regarding the existing Homestead Road DSS was not discussed, however it is assumed that this scheme has already contributed to this item via the existing Homestead Road retarding basin and land acquisition). If the scheme is a “drainage strategy” rather than a DSS, MWC indicated they would contribute the funds for water quality component of the O’Sheas Road DSS (and possibly some funds for the levee protection works).

It is recommended that the developer engage MWC in regard to discussing this fund sharing arrangement in more detail. The results of the MUSIC modelling detailed in Section 10 should be used to help facilitate this discussion.

It is recommended that no more than 300 l/s be allowed to divert to Berwick Waterways via a 450 mm diversion pipe. If more flow is allowed to divert, there could be adverse impacts in regard to the flood volume and flood level implications within the development as discussed.

8. Sediment Control

Four Sediment Ponds and three wet sump vortex systems are proposed as part of the SWMP to ensure course sediment collection prior to stormwater discharge to any wetland system.

8.1 Element Sizes

Essentially the sediment ponds treat existing external DSS catchments (which currently have little or no sediment pond collection facilities). The wet sump vortex systems are proposed to treat local Berwick waterways catchments only. Table 10 details the element sizes proposed.

Table 10 Sediment Control Elements Proposed

Element	Type	Normal Water Level (m AHD)	Top of Extended Detention (m AHD)	Area at NWL (m^2)	Detention Time (hrs)	Comments
S1	Sediment Pond - Existing Homestead Road Catchment	17.6 m AHD - 700 mm above NWL to isolate for maintenance	N/A - no ED required in isolated sediment pond	1150		1 m deep, 800 m^2 at average depth
S2	Sediment Pond - Existing O'Sheas Road and Homestead Road Catchment	16.9 m AHD - Same as wetland NWL to allow outfall from O'Sheas Road DSS	17.1 m AHD - 200 mm ED - same as wetland system	2200	1.9	2 m deep, 1600 m^2 at average depth
S3	Sediment Pond - Existing and Future Centre Road Catchment	16.9 m AHD - Same as wetland NWL to allow outfall from O'Sheas Road DSS	17.1 m AHD - 200 mm ED - same as wetland system	950	0.8	1 m deep, 650 m^2 at average depth
S4	Sediment Pond - Existing Golf Links Road Catchment	16.9 m AHD - Same as wetland NWL to allow outfall from Existing Golf Links Road Catchment	17.1 m AHD - 200 mm ED - same as wetland system	1300	1.1	1 m deep, 900 m^2 at average depth
CDS 1	Continuous Deflective Separation Structure or equivalent	N/A - primary treatment of local catchment flows				
CDS 2	Continuous Deflective Separation Structure or equivalent	N/A - primary treatment of local catchment flows				
CDS 3	Continuous Deflective Separation Structure or equivalent	N/A - primary treatment of local catchment flows				
CDS 4	Continuous Deflective Separation Structure or equivalent	N/A - primary treatment of local catchment flows				

Wet sump Vortex systems have been sized given the approximate catchment areas contributing to each inlet site. The inlet locations detailed are subject to change given the functional and detailed design of the wetland elements. However, all effort should be made to maximise inflow to these systems at the upstream end of each wetland cell affected.

The sediment ponds have been sized using the Fair and Geyer Equation as detailed in Appendix EX and are sized remove 95% of ≥ 125μm sediment in the 3 month ARI event.

8.2 General Requirements

The final site layouts have not been determined yet as this is the SWMP concept design stage of the project. However, preliminary batter slopes, and siteing of the sediment ponds have been determined to ensure enough space is available for the following requirements to be met during the design process:

- Sediment pond batter slopes and access provisions will be as currently required by MWC,
- Sediment ponds will be at least 1 metre deep,
- Sediment ponds will incorporate rock or concrete bases,
- 1 in 8 batters will be incorporated from the cut line to the NWL,
- 1 in 8 batters from NWL to 500 mm below NWL will be incorporated,
- One side of the sediment ponds will incorporate 1 in 20 batters (min) to allow provision for future access tracks etc,
- Enough site area has been allocated as free off WSUD features to ensure enough area adjacent to the sediment ponds for sediment dry out during maintenance. These areas will be required to be clearly defined and raised at least 500 mm above the 10 year ARI flood level in the functional design stage of the project).

8.3 Maintenance Bypass Provisions

In addition to the above, it is proposed to ensure that each sediment pond is able to be isolated and bypassed during times of maintenance. Appendix B details the concept design of this arrangement. In summary:

- The bypass system allows bypass of flows which occur almost all the time as predicted by the MUSIC model.
- For S1, S3 and S4:
    - The headwall of the inlet pipe into the sediment pond will be able to be blocked off via bolting a temporary steel weir to the facing edge of the headwall (maximum crest height = 17.5 m AHD.
    - A 450 mm dia pipe (IL at inlet pipe headwall – 16.9 m AHD) will be constructed connecting the sediment pond headwall with a downstream wetland balance pipe headwall. These pipes

will be blocked off during normal wetland operation and opened during maintenance periods to allow bypass of the sediment pond areas during maintenance.

- For S2 (the connection to O'Sheas Road DSS):
    - The headwall of the inlet pipe into the sediment pond S2 will be able to be blocked off via bolting a temporary steel weir to the facing edge of the headwall (maximum crest height = 17.5 m AHD.
    - Twin 450 mm dia pipes (IL at inlet pipe headwall – 16.9 m AHD) will be constructed connecting the sediment pond headwall with a downstream wetland balance pipe headwall. These pipes will be blocked off during normal wetland operation and opened during maintenance period to allow bypass of the sediment pond areas during maintenance.

This maintenance isolation system can also be expanded to the wetland cells during maintenance periods. A simular headwall blocking arrangement on all upstream and downstream balance culvert headwall and end walls can be designed. When this occurs, maintenance flows can bypass all wetland cells via twin 450 mm dia maintenance pipes.

The detailed design of the sediment pond systems and maintenance bypass systems (given final development catchment and road configurations) will be required to incorporate full consideration to MWC cleanout regimes and requirements.

9. Wetland System Design

The Berwick Waterways wetland system is a on line system. That is, there is no bypass provision for high flows (although a maintenance bypass provision is proposed). In fact, there is no provision to allow this wetland system to be an off line system due to the very flat nature of the land. Given existing DSS pipes have invert level below the proposed NWL (already minimised given downstream constraints as discussed above), any bypass system would in fact have to make water go uphill. This "on line" aspect of the design is completely consistent with the original 2005 proposal adopted by MWC as an appropriate strategy for development of Berwick Waterways.

9.1 Element Sizes

Table 11 details the wetland element sizes proposed. Table 10 details the sediment pond sizes required.

Table 11 Wetland System Sizes Proposed

Element	Type	Normal Water Level (m AHD)	Top of Extended Detention (m AHD)	Area at NWL (m^2)	Detention Time (hrs)	Comments
W1	Wetland W1 - Part combined retarding basin system	16.9 m AHD	17.1 m AHD	4040	3.5	Systems all act as one wetland - joined by balance pipes
W2	Wetland W2 - Part combined retarding basin system	16.9 m AHD	17.1 m AHD	11340	9.9	Systems all act as one wetland - joined by balance pipes
W3	Wetland W3 - Part combined retarding basin system	16.9 m AHD	17.1 m AHD	16870	14.8	Systems all act as one wetland - joined by balance pipes
P1	Pond north of Centre Road where best practice is met above this point	16.9 m AHD	17.1 m AHD	2900	2.5	Treatment to best practice to be achieved prior to discharge to P1
W4	Wetland W4 - Part combined retarding basin system	16.9 m AHD	17.1 m AHD	32600	28.6	Systems all act as one wetland - joined by balance pipes
P2	Pond at downstream end of system where best practice is met above this point	16.9 m AHD	17.1 m AHD	10000	8.8	Treatment to best practice to be achieved prior to discharge to P2
			Total ED area	82200	72.0	hours

NB: Total ED area includes sediment ponds. P1 and P2 are at the upstream and downstream ends of W4.

9.2 General Requirements

The “on line” wetland system detailed in SWS Drawing Set 1411/1 incorporates the following characteristics:

- Four wetlands W1, W2, W3 and W4 to the sizes specified in Table 11 (NWL = 16.9 m AHD),
- Four Sediment Basins S1, S2, S3 and S4 to the sizes specified in Section 8,
- A detention time for the TOTAL wetland = 72 hours (the system acts as one during frequent events),
- A extended detention depth for all wetlands = 200 mm (TED = 17.1 m AHD, Neil Craigie originally suggested an extended detention storage level of 17.3 m AHD. SWS proposes to reduce this to 17.1 m AHD (as MUSIC runs indicate additional extended detention will not afford significant water quality benefits),

9.3 Maintenance Bypass Provisions

Provision will be required to ensure that each wetland cell is able to be isolated and pumped dry during maintenance periods.

It is proposed to incorporate a maintenance isolation system similar to that proposed for S2 for all wetland cells. A simular headwall blocking arrangement (as detailed in Section 8) on all upstream and downstream balance culvert headwall and end walls can be designed. When this occurs, maintenance flows can bypass all wetland cells via twin 450 mm dia maintenance pipes.

Each wetland will incorporate smaller internal balance pipe systems connected to local maintenance pumping pits. In this way, one a wetland cell is isolated (via the mechanism above) it can be pumped dry.

9.4 Wetland Edge Design

Unlike many wetland systems, it is proposed to incorporate a number of wetland edge styles in the development. “Soft” edges denote traditional vegetated edges with safety benches. “Built”, “Urban” and “Park” edges denote a more structural style as detailed in SWS drawing set 1411/4. The three later styles incorporate walling around the wetland edge.

9.5 Flood Flow Velocity Checks

MWC require on line wetland systems to meet the following velocity requirements:

- The 100 Year ARI velocity over the wetland areas are less than 0.5 m/s,
- The 3 month ARI velocity over the wetland areas are less than 0.05 m/s,

Appendix C details the results of the analysis “Manual Velocity Calculation” as recommended by MWC for preliminary calculations in regard to this requirement. A detailed the 100 Year ARI and 3 month ARI velocity requirements can be met (provided at the thinnest part of the wetlands proposed the wetland is at least 300 m deep).

Ass this is a concept design at this stage, and the velocity requirements met, it is assumed a more detailed Hec Ras analysis and/or slight design changes during the functional design stage of the project will show the all wetland velocity requirements to be met.

9.6 Vegetation Inundation Frequency Checks

MWC require on line wetland systems to meet water depth frequency requirements to ensure vegetation health and establishment. The two checks required to be undertaken are:

- CHECK 1: Regular inundation check, Water level 80% of the time (or more) < 50% Critical Plant Height
- CHECK 2: Excessive inundation duration, Water level cannot exceed 85% of Critical Plant Height for more than 5 days

Appendix D details the results of the water depth frequency analysis which shows that on line wetland vegetation inundation requirements can be met.

Critical Plant Height is defined as the Plant Height relative to NWL (m). In all wetlands the analysis to date assumes that the shortest allowable average plant heights are 1.0 metres in Shallow Marsh zones and 1.5 m in the Deep Marsh zones. This results in

- Common Spike Rush being excluded from the specified in plant list (MWC 2014) in shallow marsh zones, and
- Water Ribbons being excluded from the specified in plant list (MWC 2014) in deep marsh zone

The analysis assumes the MUSIC model detailed in Section 10 of this report run for 10 Years of Melbourne Airport Data at 3 hourly increments (1981 – 1991, 522 mm/hr). Unlike the stormwater pollutant model, this analysis models the total wetland system as one system, with the retarding basin outlet characteristics input as into the custom outflow relationship. This model should not be used for the stormwater pollutant retention assessment. It is only applicable for the inundation depth frequency analysis.

The final form and location and bathymetry of the wetland system will be determined at the functional design stage of the project.

9.7 Wetland Outlet System

The control for wetland low flow regimes will be a formal connection to the Hallam Valley Contour drain via the existing 900 mm diameter outlet.

The wetland outlet is proposed to be a shielded weir system (crest width of 0.5 m at 16.90 m). This approach would be preferred because enhanced discharge capacity is available at low heads. With such a large wetland area this attribute is considered desirable as it minimises problems with water levels remaining above NWL for excessive periods of time between rain events.

SWS Drawing 1411/5 provide concept sketches for a suitable structure. The calculations below confirm the weir design.

Total ED area	82200	m^2			72	hours
Detention time =	72	hours				
Detention depth =			0.2	m		
ED Volume =			16440	m^3		
Wetland Outflow required =					0.063	m^3/s
Weir Design	$Q = K_w L H^{1.5}$					
Maximum Head = ED =			0.2	m		
K_w =			1.45			
L =			0.5	m		
Q =			0.065	m^3/s		OK

10. Stormwater Pollutant Modelling

The performance of wet sump vortex systems, sediment ponds and wetland system detailed in SWS Drawing Set 1411/1 and described in detail above was analysed using the MUSIC model, Version 5. Subareas and fraction imperviousness are as detailed in the RORB Model (Section 5).

Sub areas are subject to change given the final development layout, however, provided the criteria of directing as much catchment as possible to (or close to) the defined inlet locations is adhered to, the final MUSIC results are not expected to change significantly.

Bureau of Meteorology rainfall and evaporation data available in Koo Wee Rup (2004) at 6 minute intervals was utilised. This is the reference gauge defined by MWC for this area of Melbourne.

Figure 3 details the model layout developed for the SWMP.

Table 12 details the MUSIC results for the wetland system. The MUSIC model assumes no bypass of the systems in extreme events (i.e. all flow enters the sediment ponds and wetland systems). The sediment ponds and wetland systems are modelled as separate nodes in this model as they incorporate varying contributing catchment areas. Modelling them as one node will not represent what is physically happening. A nominal sediment pond size of $10m^3$ in input for each wetland node to ensure the MUSIC model runs smoothly.

The current best practice requirements of 80% TSS, 45% TP and 45% TN retention can be met upstream of both proposed “pond” systems P1 and P2. This includes allowance for the pollutants contained within the 300l/s flow diverted from the O’Sheas Road DSS. As such, the SWMP:

- Meeting best practice in regard to retention of stormwater pollutants,
- Can allow for two ecological/landscape ponds P1 and P2 as detailed while ensuring treatment to best practice upstream of these elements.

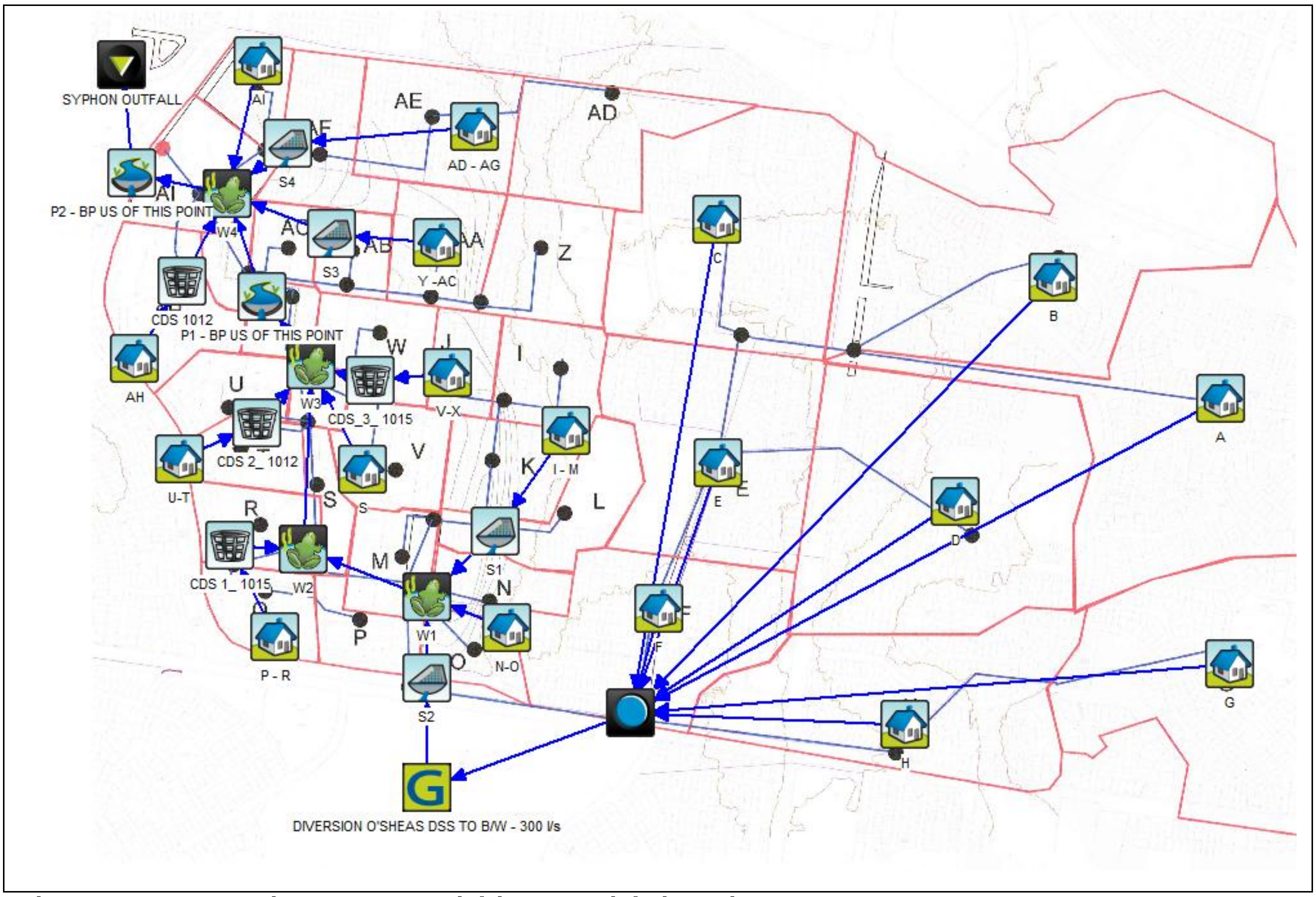


Figure 3 On Line MUSIC Model (Part model shown)

Table 12 **MUSIC Results**

O'Sheas Road DSS	Into to Diversion	Diverted to Berwick Waterways
Flow (ML/yr)	1,610	365
Total Suspended Solids (kg/yr)	306,000	51,400
Total Phosphorus (kg/yr)	639	122
Total Nitrogen (kg/yr)	4,560	968

Treatment to End W3	Pollutants Produced in total Catchment including O'Sheas Road Diversion Flow	Total Out of W3	Treatment of Water Column to W3
Flow (ML/yr)	847	805	
Total Suspended Solids (kg/yr)	145,600	11,000	92%
Total Phosphorus (kg/yr)	316	67	79%
Total Nitrogen (kg/yr)	2,328	1,190	49%

Treatment to End W4	Pollutants Produced in total Catchment including O'Sheas Road Diversion Flow	Total Out of W4	Treatment of Water Column to W4
Flow (ML/yr)	1,297	1,210	
Total Suspended Solids (kg/yr)	233,500	16,500	93%
Total Phosphorus (kg/yr)	496	99	80%
Total Nitrogen (kg/yr)	3,608	1,660	54%

11. Waterbody Long Term Sustainability

Two issues are required to be assessed to ensure a low risk of water quality and aesthetic problems in the wetlands and ponds as proposed in the SWMP. These are

- Waterbody Residence Time and
- Waterbody drawdown.

These are discussed below.

11.1 Waterbody Residence Time

Water quality issues are common problems encountered by urban water bodies. Problems can typically arise if the water body receives insufficient stormwater inflows to circulate and/or displace the water stored in the water body.

This can be a problem because high residence times between system flushing causes a reduction in dissolved oxygen levels in the water body. Under low oxygen conditions toxic algal blooms, such as blue green algae (cyanobacteria), can occur. It is therefore important to consider the residence time of the water body (or “turnover period”) assessing water body systems.

The current WSUD Engineering Procedures 2005 Manual recommendation is that water bodies located south of the Great Dividing Range should exhibit turnover periods of less than about 20 - 30 days at least 80% of the time depending on the amount of system mixing and water temperature. From this it can be inferred that when turnover periods exceed 20-30 days there is a higher chance of experiencing a blue green algal bloom, given all other factors being favourable.

Using the MUSIC model described above, a quantitative water balance analyses was undertaken to determine the water body turnover periods. As this aspect of the design is primarily looking at how the wetland operates in low flow regimes, the total wetland and pond system was modelled as one combined system (Area = 82,200 m^2, waterbody volume = 25,000 m^3). The model was run for 10 years of Koo Wee Rup data (1981 – 1991). Figure 4 details the results of the analysis.

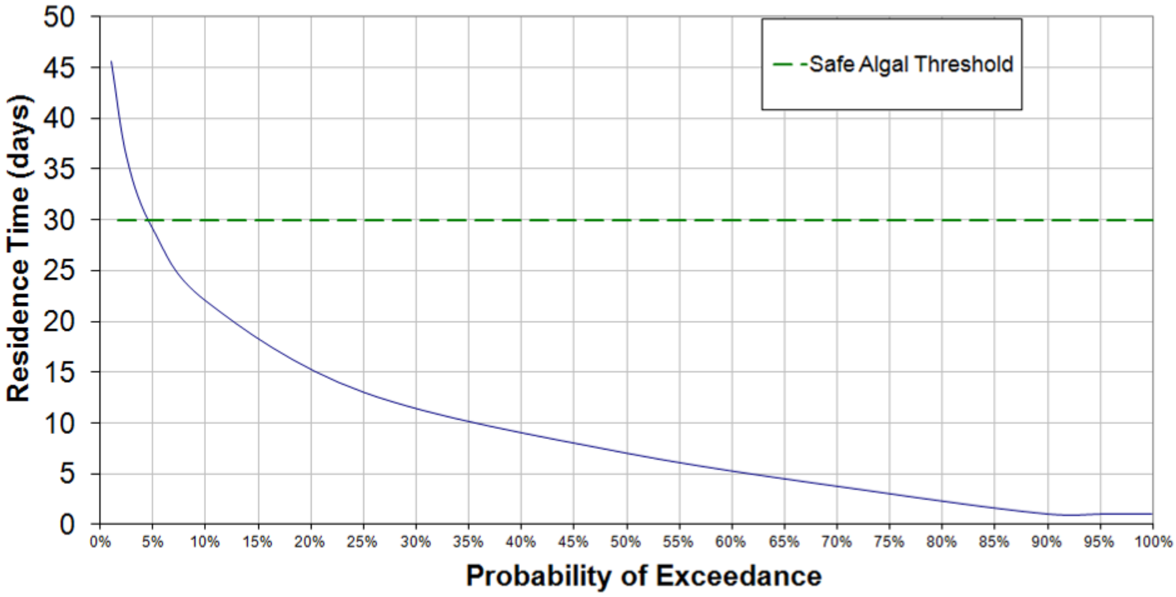


Figure 4 **Waterbody Turnover Analysis Results**

As detailed, a residence time of less than 20 days is expected to occur 875% of the time over the period analysis. As such, the waterbody proposed is deemed to have a low risk of water quality issues over time.

This risk is further reduced via the following aspects of the design:

1. Water body evaporation (i.e. surface area) is minimised via the use of formal edges in many areas,
2. The majority of the water body is a wetland system which ensures that any algae in the system has competition for food via the significant wetland vegetation present, and therefore has less chance of expanding onto a bloom at any stage.

11.2 Water Body Drawdown Analysis

Water body drawdown issues are common problems encountered by urban water bodies. Problems can typically arise if water body receives insufficient stormwater inflows resulting in significant water level drawdown in-between storm events. To assess the level of drawdown expected, the water balance model described above was used to assess drawdown levels over the time series detailed.

Table 13 details the result of the drawdown analysis.

Table 13 **Expected Drawdowns – Total Wetland and Pond system**

Percentile	Drawdown Total Wetland System (mm)
0.5%	-93.5
1.0%	-78.7
5.0%	-29.5
15.0%	-4.2
20.0%	0.6
50.0%	46.7
70.0%	109.0
95.0%	208.0
97.0%	220.0
99.5%	263.0

The above indicates that 70% of the time, drawdowns will be higher than 110 mm below NWL. 3% of the time they could be 220 mm below NWL.

It should be noted that draw down is primarily an aesthetic issue. Wetland vegetation will largely "hide" drawdown effects in very dry times. In "new" water bodys vegetation is often used to hide drawdown in the order of 500 mm below normal water level. As such, the drawdowns estimated in Table 14 should not cause undue aesthetic issues in the long term.

12. Flooding Due to Hallam Valley Contour Drain

12.1 Flood Protection Required due to Extreme Floods in the Hallam Valley Contour Drain

Report 5 (Section 1) recommended that MWC address the issue of flood protection in regard to HVCD levee overtopping via levee and drain augmentation works Rather than raise the levee system, and local flood levels west of HVCD, it was proposed to use the existing UFZ drainage reserve located to the west of the HVCD to supplement capacity. The proposal is as detailed in Figures 5.

Report 5 should be referred to in detail for the full recommendations. The SWMP detailed below assumes that the intent of this previous concept design is met.

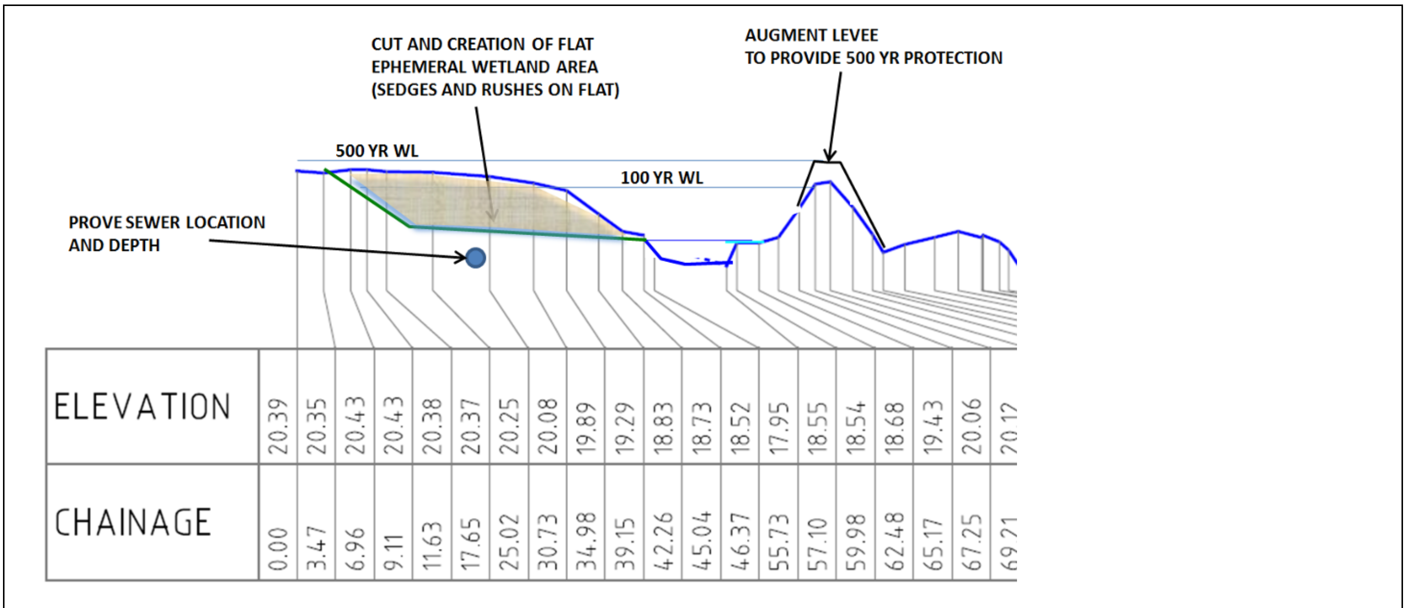


Figure 5 **Proposed HVCD Flood Mitigation Works – Typical Cross Section 2011 Report 5**

In 2013 Aurecon provided MWC with updated hydraulic modelling along the Hallam Valley Contour Drain (HVCD) at Berwick Waterways. Aurecon confirmed that currently,

- between the confluence with BTD and Centre Road, the existing levee is expected to provide 100 year ARI protection,
- at Centre Road and upstream of Centre Road to Greaves Road, the levee provides between 20 year ARI – 50 year ARI protection, and
- adjacent to Greaves Road, the HVCD is likely to be overtopped in events between 20 year ARI - 50 year ARI.

It is not known at this time if information was given to MWC in relation to the levee adjacent to Berwick Town Drain.

Aurecon found that if 100 ARI protection is sought for land to the east of the HVCD levee through levee bank increases only, the levee will need to be raised up to 600mm in some locations. The increase in 100 year flood level that this introduces would lead to extensive inundation of Montebello Boulevard to the west. However dwellings are thought to be located sufficiently high above the flood level. Surveyed property levels adjacent to Parkwood Avenue indicate that approximately 300mm freeboard is available between existing floor levels and 100 year ARI Flood Level. Other locations would require additional survey to confirm floor levels relative to flood levels.

It is assumed that undertaking levee bank works only would be an interim measure. No freeboard would be provided to properties on the east side of the HVCD as this would increase the risk of flooding for events greater than 1 in 100 year ARI on the west side. Prior to undertaking these works, Aurecon suggested that additional full width cross-sections be surveyed to provide additional resolution to the hydraulic modelling.

Aurecon reiterated that drain augmentation in conjunction with levee works is required ultimately. This can provide 100 year ARI protection and freeboard to land on both sides of the HVCD and can provide 200 year ARI and 500 year ARI flood protection if required. The general adoption of the Aurecon and MWC plan as detailed above has been adopted as part of the SWMP as detailed below.

At the present time it is assumed that, if Berwick Waterways is to be rezoned to allow development then the HVCD levee (or equivalent system) our area must ultimately provide 100 Year ARI flood protection with 600 mm freeboard or 500 yr protection whichever is the highest. It is proposed to address the flood protection aspect of the HVCD levee in stages in line with the current advice above. SWS drawing 1411/3 details this proposal and the various stages of levee/drain augmentation required.

Stage 1 will be increasing the height of the existing levee to provide 100 yr protection to Berwick Waterways. The work will be undertaken by the developers of the land.

Stage 2 will be to remove existing levee within the drainage reserve and construct new levee to merge with the development fill. This will be done stage by stage as abutting stage works are developed. The new road will be located on much of the relocated bund. It is proposed to incorporate roads levels to at least the ultimate 100 Year ARI flood level plus 300 mm freeboard. New lots abutting this perimeter road will be at a level that provides 100 Year ARI flood protection with 600 mm freeboard or 500 yr protection whichever is the highest. Where there is no road on top of bund, that land will be at 100 Year ARI flood protection with 600 mm freeboard or 500 yr protection whichever is the highest.

It should be noted that the fill/levee height will largely be 100 mm above the requirements of the previous paragraph. The lowest point in the system will be adjacent to W2 (100 mm below the fill/levee height) to ensure, if an event greater than a 500 year magnitude occurs, this overflow will be directed to the extreme flow path provision as detailed in SWS Drawing 1411/3.

Stage 3 will be completed by MWC. At some time in the future MWC are required to undertake the ultimate remediation, augmentation works of the contour drain so as to manage those catchments external to our site that contribute to it.

It should be noted that all fill and levee levels detailed are based on the preliminary 2011 SWS analysis in Report 5. As such, all levels and proposals are subject to change during the design process. However, at this stage in the project, it is considered that the levels detailed will give a reasonable representation of the ultimate fill levels required in the development.

It is recommended that, once Council and MWC have approved this approach, that the detailed design of Stages 1, 2 and 3 be completed. This should be done for both the Berwick Town Drain and the Hallam Valley Contour Drain interface to a point upstream where no inflow can occur into Berwick Waterways. As part of the design process, detailed hydraulic modelling must be performed to set 100 Year and 500 year ARI flood levels at each stage of levee, fill and drain augmentation. Once this is complete, final fill levels and extreme flow provisions within Berwick Waterways must be set. All modelling must show no impact of the proposals on existing subdivision to the west.

12.2 Extreme Flood Emergency Provisions

Any SWMP for Berwick Waterways must recognise that, whatever the flood protection provided adjacent to the HVCD, there is always the very small risk that an event may occur of a magnitude in excess of the flood protection provided.

This SWMP addresses this issue by:

1. Defining an extreme overland flow path provisions though the subdivision as described above (See 1411/3),
2. Ensuring a emergency supplementary pumping system is constructed and available to be accessed in events greater than the 500 Year ARI event (See SWS Drawing 1411/5),
3. That adequate emergency access/evacuation routes are indentified and designed into the subdivisional layout of Berwick waterways as shown in SWS Drawing 1411/3,
4. Ensuring that an emergency management plan be prepared by council and MWC with specific reference to ensuring Points 3 and 4 are incorporated within this document and procedures.

13. Discussion

This SWMP proposes an online wetland, flood storage system and HVCD flood protection works to enable the development Berwick Waterways.

The sediment ponds, wetlands and flood storage provisions detailed in this report have been sited and placed given consideration of many constraints and objectives, including the landscape and social objectives of the developers of the land. System location, size, shape and orientation may vary during the design process. However, the sizes and shapes of drainage elements detailed are realistic given:

- The sediment pond and wetland inlet pipe system outfall invert levels (existing and future),
- The probable road alignments associated with future development,
- The catchment characteristics,
- The very flat site topography,
- The outlet invert level constraints and required outfall works,
- Stormwater pollutant retention requirements,
- WSUD asset maintenance requirements,
- Flood storage requirements,
- WSUD asset integrity requirements, and
- Extreme flood provision requirements.

Given the above it is considered that the SWMP, as detailed at this time, is deemed to provide realistic sizes and shapes for drainage assets.

This report is provided to MWC to show that all issues of importance have been considered in regard to ensuring MWC can proceed in confidence in regard to ultimate adoption and construction of all drainage assets.

It is noted, that once this approval in principle to the SWMP is agreed to, future work is required to further develop the drainage proposals. This includes:

- Determining the soil and groundwater constraints with regard to the nature of the soil, any acid sulphate issues, and groundwater level impacts,

- Undertaking detailed site service proving (overhead and underground),

- Undertaking a two dimensional, unsteady state hydrological model to confirm the flood levels calculated in Section 5,

- Undertaking the detailed design of Stages 1, 2 and 3 of the HVCD augmentation and flood protection works as detailed in SWS Drawing 1411/3 (this work must include using detailed hydraulic modelling to define 100 Year and 500 year ARI flood levels at each stage of levee, fill and drain augmentation),

- Ensuring detailed design and construction of the outfall system as defined in SWS Drawing 1411/2,

- Finalising an extreme overland flow path provision though the subdivision (See 1411/3),

- Finalising the design on the maintenance/emergency supplementary pumping system (See SWS Drawing 1411/5),

- Consulting with MWC and council in regard to adequate emergency access/evacuation routes and ensuring these are captured in any future emergency management plan prepared by these two authorities (SWS Drawing 1411/3),

- Ensuring a full design of all sediment pond systems (given final development catchment configurations) to ensure full consideration of MWC cleanout regimes and safety requirements,

- Finalising road layout and subdivision levels to meet all drainage requirements (catchment delineation, flood protection, emergency access provisions etc) as detailed in this report,

- Formulating longitudinal sections of the major pipelines at the functional design stage of the project (given final fill proposals) to ensure adequate cover and invert level definitions going forward into the detailed design stage of the project,

- Developing an acceptable staging plan for both the stages within the precinct as well as the drainage works proposed, and

In addition it is recommended that the developer engage MWC in regard to discussing a fund sharing arrangement given the significant stormwater treatment provided to the external O'Sheas Road DSS in this SWMP.

14. Abbreviations and Definitions

The following table lists some common abbreviations and drainage system descriptions and their definitions which are referred to in this report.

Abbreviation Descriptions	Definition
AHD - Australian Height Datum	Common base for all survey levels in Australia. Height in metres above mean sea level.
ARI - Average Recurrence Interval.	The average length of time in years between two floods of a given size or larger
BoM	Bureau of Meteorology
BTD	Berwick Town Drain
Evapotranspiration	The loss of water to the atmosphere by means of evaporation from free water surfaces (e.g. wetlands) or by transpiration by plants
Groundwater Level	The level of groundwater below the surface level at a particular point of interest (usually given in AHD or relative to surface level)
Inlet Pond	See Sediment Pond
Hectare (ha)	10,000 square metres
Hec Ras	A one dimensional, steady state hydraulic model which uses the Standard Step Method to calculate flood levels and flood extents
HVCD	Hallam Valley Contour Drain
Kilometre (km)	1000 metres
m^3/s -cubic metre/second	Unit of discharge usually referring to a design flood flow along a stormwater conveyance system
Megalitre (ML) (1000 cubic metres)	1,000,000 litres = 1000 cubic metres Often a unit of water body (eg pond) size
MUSIC	Hydrologic computer program used to calculate stormwater pollutant generation in a catchment and the amount of treatment which can be attributed to the WSUD elements placed in that catchment. Can also be used to calculate water body turnover period and wetland draw downs etc
NWL	Normal Water Level – invert level of lowest outflow control from a wetland or pond.
PET	Potential Evapotranspiration – potential loss of water to the atmosphere by means of evaporation or transpiration from wetland or pond systems.
Retarding Basin	Drainage element used to retard flood flows to limit flood impacts downstream of a development. Can include complementary WSUD and ecological site benefits if wetland incorporated within the site.
RORB	Hydrologic computer program used to calculate flood flows and retarding basin storage requirements.
Sedimentation basin (Sediment pond)	A pond that is used to remove coarse sediments from inflowing water mainly by settlement processes.
SWS	Stormy Water Solutions (Report author)
TED	Top of Extended Detention – Level to which stormwater is temporarily stored for treatment in a wetland or pond (above NWL).
TSS	Total Suspended Solids – a term for a particular stormwater pollutant parameter
TP	Total Phosphorus – a term for a particular stormwater pollutant parameter
TN	Total Nitrogen – a term for a particular stormwater pollutant parameter
Wetland	WSUD elements which is used to collect TSS, TP and TN. Either permanently or periodically inundated with shallow water and either permanently or periodically supports the growth of aquatic macrophytes

APPENDIX A Design of O’Sheas Road Diversion Pipe

O'Sheas Road Connection Culvert						
Upstream IL =	17.3	m AHD				
Downstream IL at S1 =		16.9	m AHD			
Culvert length =		185	m			
Diversion weir crest =		18.5	m AHD			
Driving head low flow diversion =			1.6	m		
Low Flow diversion required =			0.3	m^3/s		
Culvert acting under outlet control - low flow off take						
head loss = (Ke+Kex)×V^2/2g+ S_f×L						
S_f = $Q^2n^2/A^2R^{4/3}$						
culvert flowing full						
pipe dia =		0.457	m	actual 450 dia		
RCP pipe radius =		0.2285	m			
Design flow =		0.3	m^3/s			
Wetted perimeter =		1.44	m			
Area =		0.16	m^2			
Hyd radius =		0.11425	m			
V =		1.83	m/s			
Ke =		0.5				
Kex =		1				
n =		0.011				
L=		185				
S_f =		0.0073				
Head loss =		1.6	m			
450 mm connection pipe						
Assessment of maximum flow when flooding in HVCD						
HVCD 100 Year level		20.7	m AHD			
100 yr WL in S1 =		18.5	m AHD			
Driving head in 10 yr event =			2.2	m		
pipe dia =		0.457	m			
RCP pipe radius =		0.2285	m			
Design flow =		0.35	m^3/s			
Wetted perimeter =		1.44	m			
Area =		0.16	m^2			
Hyd radius =		0.11425	m			
V =		2.13	m/s			
Ke =		0.5				
Kex =		1				
n =		0.011				
L=		185				
S_f =		0.0099				
Head loss =		2.2	m			
450 mm connection pipe						
Therefore 450 limits inflow to S1 to 0.3 m^3/s over range of HWL considerations						

APPENDIX B Maintenance Bypass Pipe Calculations

All Flow Frequency analysis detailed below are based on the 3 hourly output from the appropriate MUSIC node for a MUSIC run undertaken using 10 years of 1981 – 1991 Koo Wee Rup Data.

The following details the bypass provisions for S1, S3 and S4.

Worst case in terms of flow and pipe length assessed - S4

Flow Frequency Analysis

Maximum flow calculated = 1.7 m^3/s

in this year , an event about 2 year occurred

Percentile	Inflow (m^3/s)
0.5%	0.000
1.0%	0.000
5.0%	0.000
15.0%	0.000
20.0%	0.000
50.0%	0.000
70.0%	0.001
80.0%	0.002
98.0%	0.075
99.5%	0.187

Assume sediment pond to be isolated for maintenance.

Bypass flow for maintenance activity > flow which occurs 98% of the time = 0.075 m^3/s

Upstream IL at Sediment pond inlet pipe Headwall =	16.9	m AHD
Water allowed to head up to top of temporary weir =	17.5	m AHD
Downstream IL at W4 outlet =	16.8	m AHD
Obvert at downstream end =	17.25	m AHD
Maximum Maintenance bypass pipe length =	330	m - S4
Maximum Head Loss =	0.25	m

culvert flowing full

pipe dia =	0.45	m
RCP pipe radius =	0.225	m
Design flow =	0.075	m^3/s
Wetted perimeter =	1.41	m
Area =	0.16	m^2
Hyd radius =	0.1125	m
V =	0.47	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	330	
S_f =	0.0007	
Head loss =	0.2	m

OK - 450 mm maintenance sediment pond bypass pipe can pass the flow which occurs 98% of the time.

The following details the bypass provisions for S2 and for maintenance on wetland cells.

Wetlands and Sediment Pond S2

Flow Frequency Analysis — Maximum flow calculated = 0.3 m^3/s

O'Sheas Road Maximum inflow

Percentile	Inflow (m^3/s)
0.5%	0.000
1.0%	0.000
5.0%	0.000
15.0%	0.000
20.0%	0.000
50.0%	0.003
70.0%	0.009
80.0%	0.014
96.0%	0.173
99.5%	0.300

Assume sediment pond to be isolated for maintenance.

Bypass flow for maintenance activity > flow which occurs 96% of the time =

0.173 **m^3/s**

Upstream IL at Sediment pond inlet pipe Headwall =	16.9	m AHD
Water allowed to head up to top of temporary weir =	17.5	m AHD
Downstream IL at W1-W2 balance pipe US end =	16.8	m AHD
Obvert at downstream end =	17.25	m AHD
Maximum Maintenance bypass pipe length =	150	m

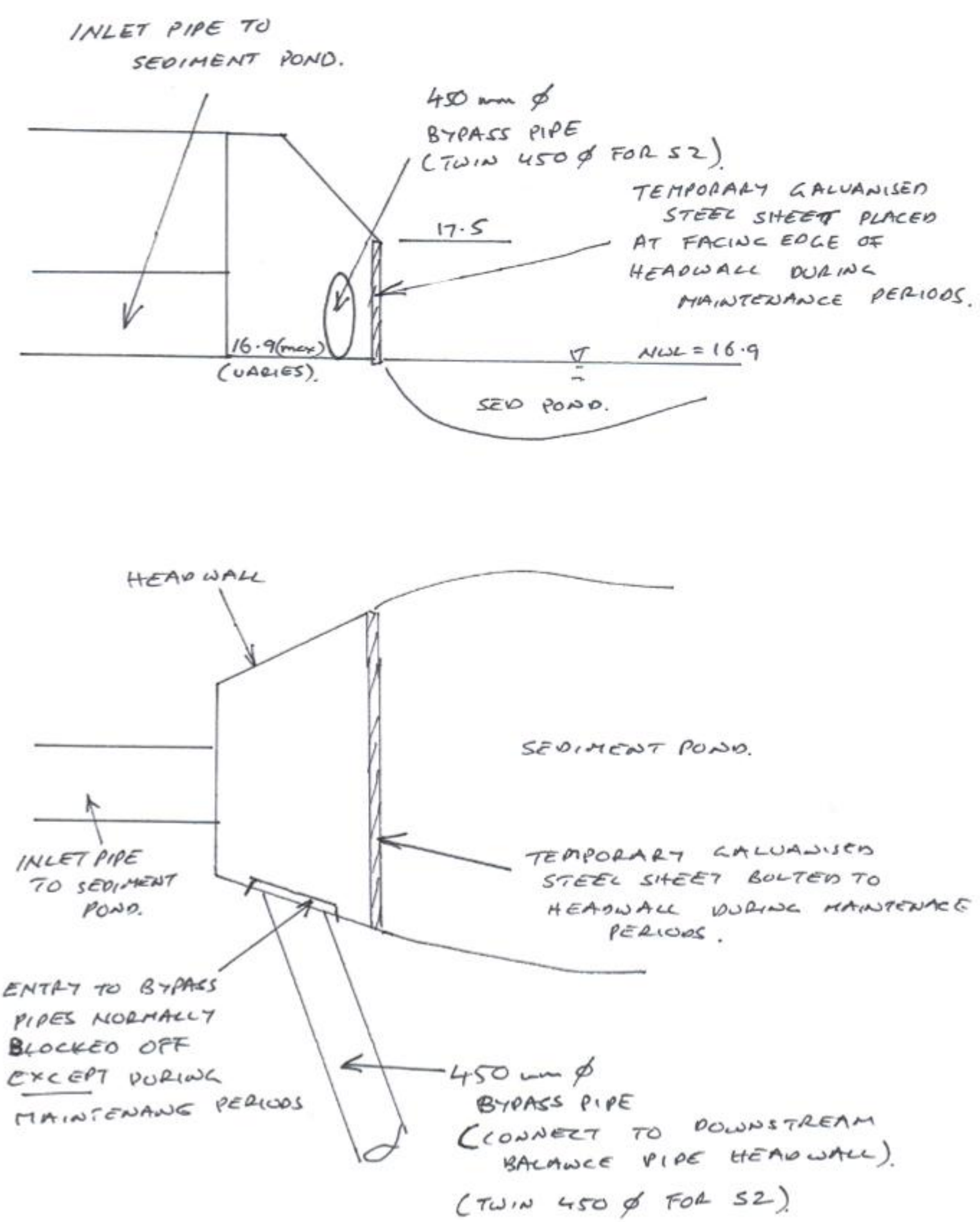
Maximum Head Loss = 0.25 m

culvert flowing full

pipe dia =	0.45	m
RCP pipe radius =	0.225	m
Design flow =	0.086	m^3/s
Wetted perimeter =	1.41	m
Area =	0.16	m^2
Hyd radius =	0.1125	m
V =	0.54	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	150	
S_f =	0.0009	
Head loss =	0.2	m

OK - Twin 450 mm maintenance bypass pipe can pass the flow which occurs 96% of the time around S2.

Note: This arrangement can also be used to isolate the main wetland systems



Concept Design of typical maintenance pipe bypass provision for Sediment Ponds

Subject to change given detailed design

APPENDIX C Flood Flow Velocity Checks

Initial Check - Wetland W1						
Maximum Flow Rate						
Q_{100} =	7.9	m^3/s (RORB)				
Q_{10} =	3.9	m^3/s (RORB)				
Q_{3mth} =	0.6	m^3/s (MWC Rule of Thumb given 1 Year Flow)				
Wetland Normal Water Level (NWL) =				16.9	m AHD	
Wetland Top of Extended Detention (TED				17.1	m AHD	
Base level at wetland narrwoest width =				16.6	m AHD	
10 Year TWL =		17.78	m AHD (RORB)			
Narrowest NWL width =			**34**	m		
Narrwowest Base width =			29.2	m		
Narrwowest TED width =			37.2	m		
Narrowest 10 yr width =			48.08	m		
3 month Cross Sectional area =			16.6	m^2		
10 year Cross Sectional area =			45.6	m^2		
3 month Velocity =			0.04	m/s		
100 year Velocity =			0.17	m/s		

Initial Check - Wetland W3

Maximum Flow Rate

Q_{100} =	12.4	m^3/s (RORB)
Q_{10} =	6.4	m^3/s (RORB)
Q_{3mth} =	0.9	m^3/s (MWC Rule of Thumb given 1 Year Flow)
Wetland Normal Water Level (NWL) =	16.9	m AHD
Wetland Top of Extended Detention (TED) =	17.1	m AHD
Base level at wetland narrwoest width =	16.6	m AHD
10 Year TWL =	17.63	m AHD (RORB)
Narrowest NWL width =	**35**	m
Narrwowest Base width =	35	m formal
Narrwowest TED width =	35	m edges
Narrowest 10 yr width =	35	m
3 month Cross Sectional area =	17.5	m^2
10 year Cross Sectional area =	36.1	m^2
3 month Velocity =	0.05	m/s
100 year Velocity =	0.34	m/s

Initial Check - Wetland W2

Maximum Flow Rate

Q_{100} =	9.8	m^3/s (RORB)
Q_{10} =	5.0	m^3/s (RORB)
Q_{3mth} =	0.7	m^3/s (MWC Rule of Thumb given 1 Year Flow)
Wetland Normal Water Level (NWL) =	16.9	m AHD
Wetland Top of Extended Detention (TED	17.1	m AHD
Base level at wetland narrwoest width =	16.6	m AHD
10 Year TWL =	17.66	m AHD (RORB)
Narrowest NWL width =	**38**	m
Narrwowest Base width =	38	m formal
Narrwowest TED width =	38	m edges
Narrowest 10 yr width =	38	m
3 month Cross Sectional area =	19.0	m^2
10 year Cross Sectional area =	40.3	m^2
3 month Velocity =	0.04	m/s
100 year Velocity =	0.24	m/s

Initial Check - Wetland W4 downstream of Centre Road

Maximum Flow Rate

Q_{100} =	12.4	m^3/s (RORB)
Q_{10} =	6.4	m^3/s (RORB)
Q_{3mth} =	0.9	m^3/s (MWC Rule of Thumb given 1 Year Flow)
Wetland Normal Water Level (NWL) =	16.9	m AHD
Wetland Top of Extended Detention (TED) =	17.1	m AHD
Base level at wetland narrwoest width =	16.6	m AHD
10 Year TWL =	17.54	m AHD (RORB)
Narrowest NWL width =	**50**	m
Narrwowest Base width =	45.2	m
Narrwowest TED width =	53.2	m
Narrowest 10 yr width =	60.24	m
3 month Cross Sectional area =	24.6	m^2
10 year Cross Sectional area =	49.6	m^2
3 month Velocity =	0.03	m/s
100 year Velocity =	0.25	m/s

Initial Check - Wetland W4 downstream of Centre Road - when total catchment contributing

Maximum Flow Rate

Q_{100} =	19.2	m^3/s (RORB)
Q_{10} =	9.4	m^3/s (RORB)
Q_{3mth} =	1.3	m^3/s (MWC Rule of Thumb given 1 Year Flow)
Wetland Normal Water Level (NWL) =	16.9	m AHD
Wetland Top of Extended Detention (TED	17.1	m AHD
Base level at wetland narrwoest width =	16.6	m AHD
10 Year TWL =	17.54	m AHD (RORB)
Narrowest NWL width =	**50**	m
Narrwowest Base width =	45.2	m
Narrwowest TED width =	53.2	m
Narrowest 10 yr width =	60.24	m
3 month Cross Sectional area =	24.6	m^2
10 year Cross Sectional area =	49.6	m^2
3 month Velocity =	0.05	m/s
100 year Velocity =	0.39	m/s

Appendix D Wetland Inundation Checks

The analysis detailed below documents the regular inundation checks and excessive inundation checks required by MWC to ensure the long term wetland health of systems of this type. The two checks required to be undertaken are

CHECK 1: Regular inundation check

Water level 80% of the time (or more) < 50% Critical Plant Height

CHECK 2: Excessive inundation duration

Water level cannot exceed 85% of Critical Plant Height for more than 5 days

Critical Plant Height is defined as the Plant Height relative to NWL (m). It is assumed that the shortest allowable average plant heights are 1.0 metres in Shallow Marsh and 1.5 m high in Deep Marsh zones. This results in

- Common Spike Rush being excluded from the specified in plant list in shallow marsh zones, and
- Water Ribbons being excluded from the specified in plant list in deep marsh zone

The analysis assumes the MUSIC model detailed in Section 10 of this report is run Koo Wee Rup Data at 3 hourly increments (1981 – 1991).

As the extended detention orifice systems and the weir systems can pond water back over the wetland systems. The MUSIC model incorporates a custom outflow relationship to model this effect based on the RORB stage/storage relationship.

Wetland Plant Health Requirements Check - total wetland

Critical Plant Depth = Plant Height relative to NWL (m) — 10 Year KWR data 1981 - 1991 - 794 mm/yr

Extended Detention = 0.2 m

Shortest Allowable Average Plant Heights in Wetland 1 =	1.0	m	Shallow Marsh
	1.5	m	Deep Marsh

Plants to be excluded from plant list for shallow marsh and/or deep marsh

Note: ***Common Spike Rush not to be specified in plant list in shallow marsh zones***

Water Ribbons not to be specified in plant list for W1 in deep marsh zone

Wetland specifications

	Max Permanent Pool Depth *(as advised Design Flow 11/2/14)*	Average Height of Plant in wetland plant list given any exclusions	50% Average Plant Height	Check 1 - Critical Plant Height Relative to NWL (Average plant height/2)-PP	Check 2 - Critical Plant Height Relative to NWL (Average plant height -PP)*0.85)
Shallow Marsh	0.15	1.0	0.50	0.35	0.7
Deep Marsh	0.35	1.5	0.75	0.40	1.0

CHECK 1: Regular inundation check

Water level 80% of the time (or more) < 50% Critical Plant Height (CHECK 1)

W1 Inundation Frequency Analysis — Water level (relative to NWL) expected 80% of the time or more = 150.0 mm

Percentile	WL Relative to NWL - (mm)
0.5%	-93
1.0%	-79
5.0%	-29
15.0%	-4
20.0%	1
50.0%	47
70.0%	109
80.0%	150
97.0%	220
99.5%	263

Water level (relative to NWL) expected 80% of the time or more < CHECK (1) Shallow Marsh?

YES

Check1 met

Water level (relative to NWL) expected 80% of the time or more < CHECK (1) Deep Marsh?

YES

Check1 met

CHECK 2: Excessive inundation duration

Water level cannot exceed 85% of Critical Plant Height for more than 5 days

Critical plant Height to meet this condition =	0.7	m	
Number of times exceeded in a 3 HOUR time step =	0	0	
		days total over 10 years	
Longest duration height exceeded (by examination of printed graph of depth charts	0	time increments starting at X	
.=	0	days	

Is the water level NEVER more than 85% of Critical Plant Height for more than 5 days **YES** **Check2 met**

APPENDIX E Sediment Pond Sizing

The following defines the sediment pond for Sediment Ponds S1, S2, S3 and S4 based on the Fair and Geyer Equation. These sizes are specified in Section 8 and in the MUSIC model in Section 10 of this report.

Sediment Pond S1				
Vs =	0.011	m/s		
d_e =	0.0	m		
d_p =	1.0	m		
d* =	1.0	m		
(d_e+d_p) = / (d_e+d^*)	1.0			
$Q_{1\ year}$ =	0.6	m/s	(RULE OF THUMB GIVEN 1 YEAR FLOW)	
A =	800	m^2		
V_s = / Q/A	14.67			
λ =	0.26	pond shape assumption		
n =	1.35			
Fraction of Initial Solids Removed				
R =	96%			
Requirement: Melbourne Water Usually Require R = 95% for a 125 micrometer particle for 3 month event				
Cleanout Frequency				
Catchment Area =	45	ha	Just urban catchment considered	
Sediment load =	1.6	m^3/ha/yr	(Willing and Partners 1992 urban load)	
Gross Pollutant Load =	0.4	m^3/ha/yr	(Alison et al 1998)	
Actual basin depth =	1	m		
Actual Basin area =	800	m^2		
Therefore, cleanout frequency required =	$\frac{R(1.6+0.4)A_{catchment}}{0.67d_{basin}*A_{basin}}$ =	0.16	per year	
	.= every	6.1	years	
Assumes cleanout when basin 2/3 full				

Sediment Pond S2				
Vs =	0.011	m/s		
d_e =	0.2	m		
d_p =	2.0	m		
d* =	2.0	m		
(d_e+d_p) = / (d_e+d^*)	1.0			
$Q_{1\ year}$ =	0.3	m/s	(O'Sheas Road diversion)	
A =	2200	m^2		
V_s = / Q/A	80.67			
λ =	0.26	pond shape assumption		
n =	1.35			
Fraction of Initial Solids Removed				
R =	100%			
Requirement: Melbourne Water Usually Require R = 95% for a 125 micrometer parti for 3 month event				
Cleanout Frequency				
Catchment Area =	323	ha	Just urban catchment considered	
Sediment load =	1.6	m^3/ha/yr	(Willing and Partners 1992 urba	
Gross Pollutant Load =	0	m^3/ha/yr	(GP picked up in diversion pit)	
Actual basin depth =	2	m		
Actual Basin area =	2200	m^2		
Therefore, cleanout frequency required =	$\frac{R(1.6+0.4)A_{catchment}}{0.67d_{basin}*A_{basin}}$ =	0.18	per year	
	.= every	5.6	years	
Assumes cleanout when basin 2/3 full				

Sediment Pond S3

Vs =	0.011	m/s	
d_e =	0.2	m	
d_p =	1.0	m	
d^* =	1.0	m	
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.0		
$Q_{1\,year}$ =	0.15	m/s	(RULE OF THUMB GIVEN 1 YEAR FLOW)
A =	650	m^2	
$\frac{V_s}{Q/A}$ =	47.67		
λ =	0.26	pond shape assumption	
n =	1.35		

Fraction of Initial Solids Removed

R = 99%

Requirement: Melbourne Water Usually Require R = 95% for a 125 micrometer particle for 3 month event

Cleanout Frequency

Catchment Area =	25	ha	Just urban catchment considered
Sediment load =	1.6	m^3/ha/yr	(Willing and Partners 1992 urban load)
Gross Pollutant Load =	0.4	m^3/ha/yr	(Alison et al 1998)
Actual basin depth =	1	m	
Actual Basin area =	650	m^2	

Therefore, cleanout frequency required = $\frac{R(1.6+0.4)A_{catchment}}{0.67d_{basin}*A_{basin}}$ = 0.12 per year

.= every 8.6 years

Assumes cleanout when basin 2/3 full

Sediment Pond S4

Vs =	0.011	m/s	
d_e =	0.2	m	
d_p =	1.0	m	
d^* =	1.0	m	
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.0		
$Q_{1\,year}$ =	0.2	m/s	(RULE OF THUMB GIVEN 1 YEAR FLOW)
A =	900	m^2	
$\frac{V_s}{Q/A}$ =	49.50		
λ =	0.26	pond shape assumption	
n =	1.35		

Fraction of Initial Solids Removed

R = 99%

Requirement: Melbourne Water Usually Require R = 95% for a 125 micrometer particle for 3 month event

Cleanout Frequency

Catchment Area =	39	ha	Just urban catchment considered
Sediment load =	1.6	m^3/ha/yr	(Willing and Partners 1992 urban load)
Gross Pollutant Load =	0.4	m^3/ha/yr	(Alison et al 1998)
Actual basin depth =	1	m	
Actual Basin area =	900	m^2	

Therefore, cleanout frequency required = $\frac{R(1.6+0.4)A_{catchment}}{0.67d_{basin}*A_{basin}}$ = 0.13 per year

.= every 7.7 years

Assumes cleanout when basin 2/3 full