Hydrological and Environmental Engineering

Hancocks Gully Development Services Scheme

Functional Design of Two Wetland/Retarding Basins and Two Vegetated Channels

Revision B

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1 Background

In 2013, Stormy Water Solutions produced a report entitled “Pakenham East Precinct Structure Plan, Proposed Drainage Strategy, Draft Report, 25 March 2013” (2013 PSP Report) for Cardinia Shire Council (Council).

This 2013 work detailed the overall requirements of the major drainage and the water sensitive urban design (WSUD) infrastructure required if the PSP was developed for urban purposes. Figure 1 details the preliminary drainage proposals. It should be noted that the 50 metre offset to Deep Creek was ultimately increased to 100 metres by Council and Melbourne Water Corporation (MWC) to allow for:

- Placement of all (or most) required frog ponds in this reserve area,
- Containment of the 100 Year ARI flood in the flood plain without increasing existing flood levels (due to filling of developable land) as per Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, “Stormy Water Solutions”, 5 October 2014

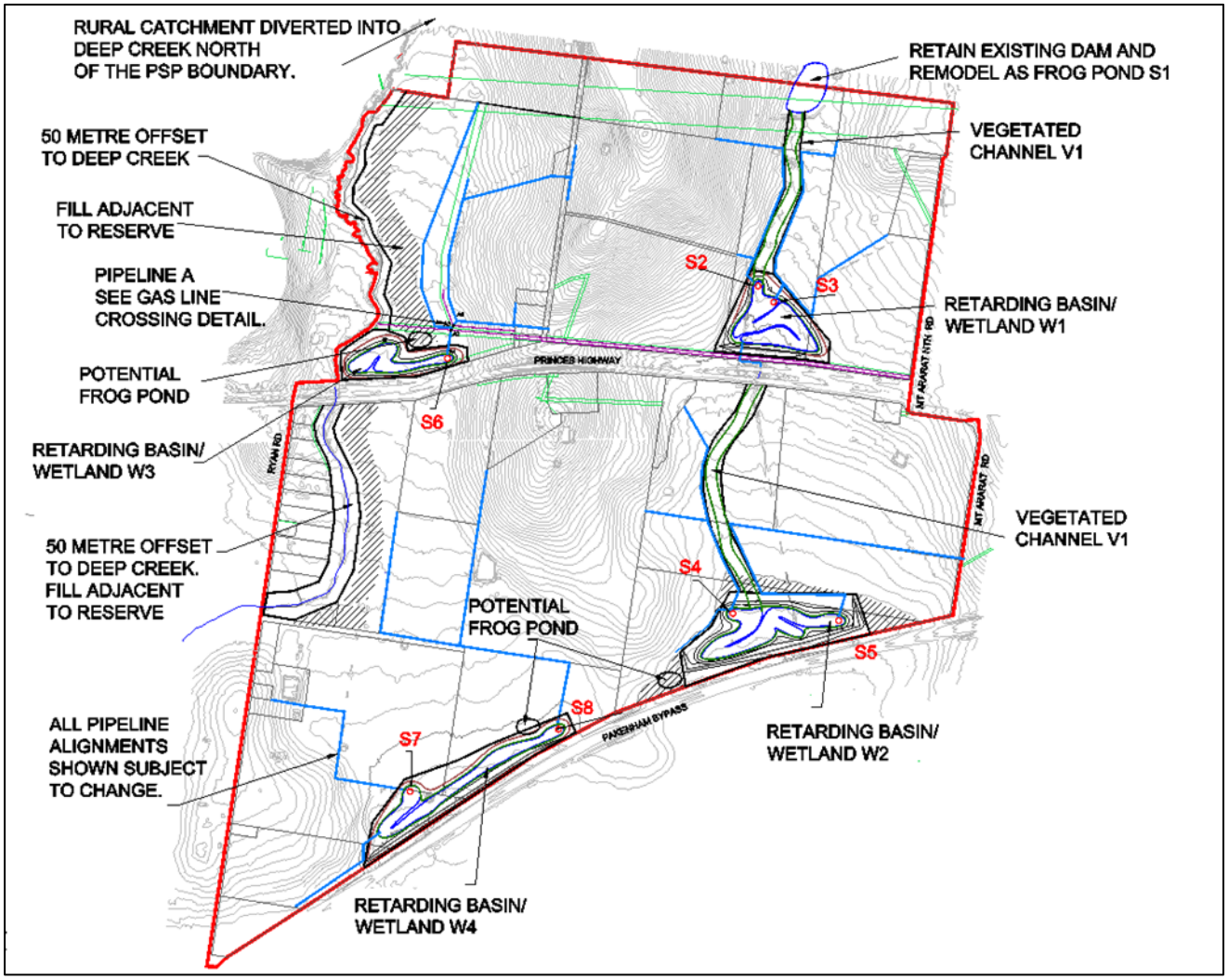


Figure 1 2013 Preliminary PSP Drainage Strategy

MWC has subsequently adopted the Hancocks Gully Development Services Scheme (DSS) which covers the eastern portion of the PSP area and the catchment contributing to W1 and W2. The adoption of the DSS is to guide orderly provision of main drainage services through the PSP area.

At this time Council and MWC require a functional design of Wetland/Retarding Basins W1 and W2 and vegetated channels V1 and V2 to be prepared to ensure future planning of the area can be met going forward.

Frog pond S1 from the original PSP proposal (Figure 1) has been omitted from the functional design as MWC have insisted that this feature provides no stormwater treatment and us such does no fall into the scope of this report.

W1 is referred to as the Hancocks Gully North Wetland/Retarding Basin in this report and W2 is referred to as the Hancocks Gully South Wetland/Retarding Basin in this Report. The remainder of this report considers the functional design requirements of these wetland systems in line with MWC's "2015 Constructed Wetland Design Manual" (2015 MWC Manual) and the design requirements of the vegetated channels in line with "Waterway Corridors, Guidelines for greenfield development areas within the Port Phillip and Westernport Region, MWC, October 2013".

2 Design Considerations

2.1 Outfall Considerations – Freeway Frog Ponds

The major constraint in regard to the Hancocks Gully South (W2) wetland/retarding basin design is the existing frog ponds located at Princess Freeway (i.e. at the catchment outfall point). Figure 2 below details this culvert system

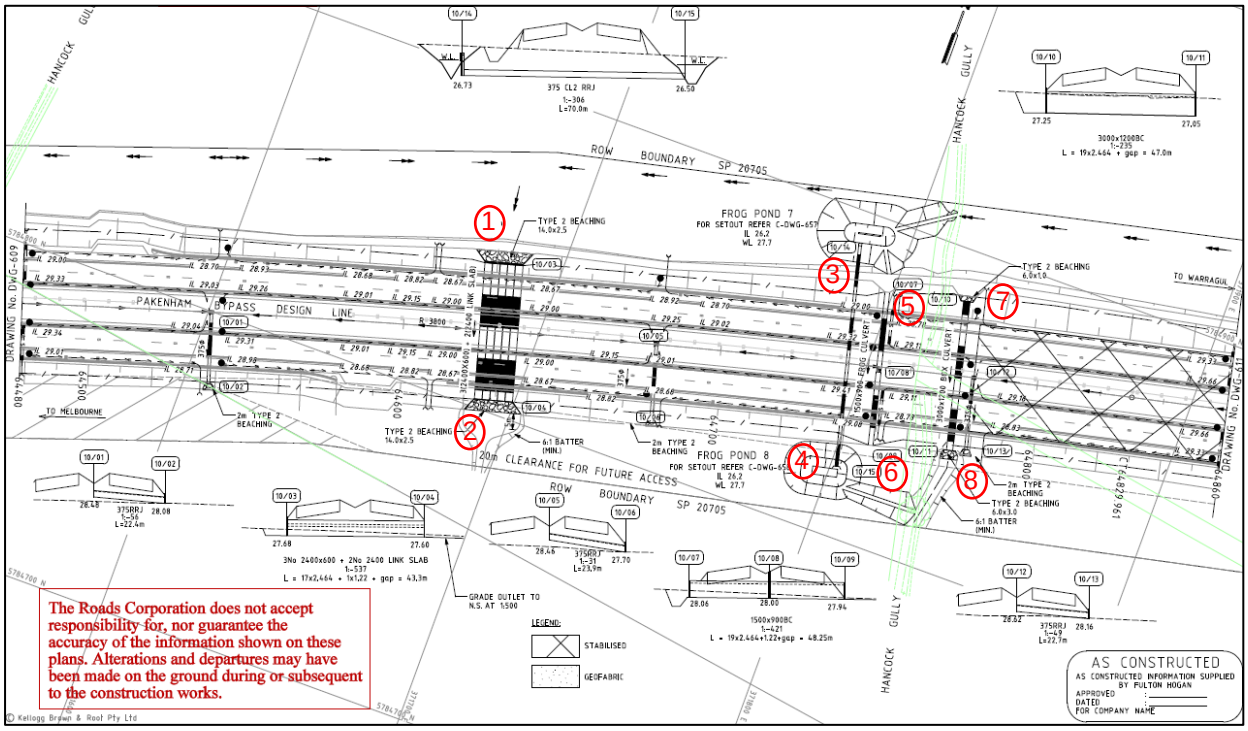


Figure 2 Vic Road Drawing 583249 detailing culvert configuration at the Freeway

The Freeway culvert system consists of:

- Culvert 1 – 2, three 2400 mm by 600 mm box culverts and two 2400 mm links slabs (IL 27.68 to 27.60 m AHD). This is the major flood conveyance structure at the freeway for the upstream catchment.
- Culvert 3 – 4, a 375 mmØ pipe (IL 26.73 to 26.50 m AHD) which connects the two freeway frog ponds,
- Culvert 5 – 6, a 1500 mm by 900 mm box frog culvert (IL 28.06 to 27.94 m AHD) which is assumed to primarily facilitate frog movements between flow events rather than provide an active flow conveyance outlet,
- Culvert 7 – 8, a 3000 mm by 1200 mm box culvert (IL 27.25 to 27.05 m AHD) which is in line with the existing farm cut alignment of Hancocks Gully.

It is assumed that low flows are required to feed the frog ponds as per Figure 2. The frog pond normal water level (NWL) at 27.70 m AHD is therefore a constraint in regard to the minimum upstream

Hancocks Gully South (W2) wetland NWL. The frog pond NWL is required to be proven in the next stage of the design process.

The total Outlet configuration has a total capacity of 20 m^3/s with 300 mm head loss over the system (Addendum B. 2)

2.2 Gas Line Considerations and Princess Highway Crossing

The major constraint in regard to the Hancocks Gully North (W1) wetland/retarding basin design is the two APA gas mains located in the Gas Easement South of the site and North of Princess Hwy.

MWC have obtained detailed survey of the Gas Mains as detailed in the September 2016 Cardno Report, “Utility Investigation Summary Report, Dore Road Pakenham”.

Previous discussions with APA Group indicate that the wetland outlet can extend over the pipelines provided:

- at least 1.2 meters of cover is provided over each gas main, and
- APA is able to “future proof” their assets before construction of the wetland occurs.

The findings from the Cardno Investigation within the vicinity of the two gas mains is shown below in Table 1. These levels are lower than those previously assumed for the gas mains and provide more cover to the top of the pipe than previously assumed. As such, there may be scope in the detailed design stage of the project to lower the five outlet pipes once the level at Princess HWY is proven.

Table 1 Summary of Gas Main Proving

Northern Line						
Location	CUE006	CUE007	CUE008	CUE009	CUE010	CUE011
NSL (m AHD)	38.902	38.628	38.407	38.454	38.819	38.932
Depth (m)	3.4	4.5	4	3.8	3.5	3.5
Top of Gas Main (m AHD)	35.502	34.128	34.407	34.654	35.319	35.432
Southern Line						
Location	CUE031	CUE030	CUE029	CUE028	CUE027	CUE026
NSL (m AHD)	38.45	38.222	38.379	38.452	38.777	38.951
Depth (m)	4	3.9	4.3	4	4	4
Top of Gas Main (m AHD)	34.45	34.322	34.079	34.452	34.777	34.951

Furthermore, the Princess highway culvert to the South of Hancocks Gully North (W1) wetland provides a constraint in regard to appropriate normal water level. The culvert configuration under Princess highway consists of twin 2400 mm by 2400 mm box culverts at an invert level of 36.28 m AHD on the upstream face with a capacity of approximately 20 m^3/s (Addendum A. 2).

2.3 Existing Surface Levels

The subject site of Hancocks Gully North (W1) exhibits natural surface levels between 38.50 and 42.50 m AHD over the wetland length of 700 m. The site is relatively steep, which has implications in regard to wetland placement.

The subject site of Hancocks Gully South (W2) exhibits natural surface levels between 28.50 and 30.50 m AHD over the wetland length of 700 m. The site is relatively flat and is ideal for wetland placement.

2.4 Water Quality Requirements

The Hancocks Gully DSS must ensure all stormwater is treated to at least current best practice prior to discharge from the PSP area.

Therefore, the wetland systems must ensure 80% retention of Total Suspended Solids (TSS), 45% retention of Total Phosphorus (TP) and 45% retention of Total Nitrogen (TN).

In addition to the above “best practice” water quality requirements, various authorities are advocating “over treatment” of Stormwater in the Westernport catchments given the RAMSAR wetland system located within Westernport Bay. The EPA have advised, given previous investigations in Westernport, that State Environment Protection Policy (SEPP) Schedule F8 could be interpreted as requiring 93% / 66% / 63% retention in regard to TSS, TP and TN respectively prior to Stormwater discharge to Westernport Bay. It is understood that currently MWC require, as a minimum, 93% retention of TSS should be adopted as a standard in this catchment, as sediment is the main treat to the Western Port Bay.

At this stage, as per previous investigations in this catchment, the functional design aims to retain stormwater pollutants to current best practice requirements.

It should be noted that future regional wetland systems located in the Cardinia Creek outfall downstream may be able to supplement and local treatment initiatives, although this regional analysis has not been undertaken at this stage. In addition, the Stormwater harvesting initiatives advocated within the “Pakenham East stormwater Harvesting Investigation, Project Report, Dalton Consulting Engineers, May 2015” will aid in meeting objectives in excess of burrent best practice.

2.5 Flood Storage Requirements

In line with current Koo Wee Rup Flood Protection District (KWRFPD) flood protection guidelines, the flood retarding basin objectives for the total catchment (W1 and W2) are to ensure:

- The peak 100 Year flow from the future development does not exceed the predevelopment flow rate at the catchment outfall point at Princess Freeway,

- The peak 24 hour 100 Year flow from the future development does not exceed the predevelopment flow rate for a storm of this duration at the catchment outfall point, and
- The two retarding basins in the catchment can store at least the difference between the expected post development and predevelopment 24 hour 100 Year flow volume to ensure no increased flood effect within the KWRFPD during a 24 hour 100 Year ARI flood event in the region.

In addition to the above, low flow regimes will need to be maintained post development in order to protect the existing ecology and channel morphology of the downstream drainage system. This is proposed to be achieved via ensuring the 1 Year ARI post development flood flow is reduced to the predevelopment rate.

2.6 Standards & Guidelines

2.6.1 MWC Constructed Wetlands Design Guidelines

Both W1 and W2 have been designed as close as possible to the current (2016) MWC constructed Wetlands Design Guidelines. Addendums A. 9 and B. 8 show how and where each wetland meet (or fail) the deemed to comply checklist for these guidelines.

2.6.2 MWC Waterway Corridors Guidelines

Both V1 and V2 have been designed to meet the main objectives of “Waterway Corridors, Guidelines for greenfield development areas within the Port Phillip and Westernport Region, MWC, October 2013”. Section 7 details the design of these elements.

3 Proposed Wetland Functional Designs

This section details the functional design of the two “on line” wetland systems required to meet the Hancocks Gully DSS objectives described in Section 2 and the Melbourne Water Constructed Wetlands Design Manual: Deemed to Comply Criteria.

3.1 Hancocks Gully North, Wetland/Retarding Basin W1

Addendum A details the functional design proposal. Stormy Water Solutions drawing set HCGN/SWS (Addendum A. 1) should be referred to for full details.

Hancocks Gully North, Wetland/Retarding Basin W1 NWL and TED have been set given the constraints described in Sections 2.2, 2.3 and the designed DSS pipeline system outfalls (as per the 2014 Master Plan work and MWC’s preliminary DSS proposals).

The NWL of Hancocks Gully North, Wetland/Retarding Basin W1 has been set to 39.00 m AHD and the TED has been set to 39.35 m AHD. It should be noted that there is very little room to move in regard to changing these levels as the design process moves forward.

The details of the wetland system are as detailed below. Note that the sediment ponds S2 and S3 have a NWL one metre above the wetland. A weir control will be used to transfer stormwater from the sediment ponds to the wetland. As such no extended detention is required in the sediment ponds.

The RORB modelling detailed in Section 4 predicts the design flows and retarding basin characteristics of the retarding basin/wetland system.

Addendum A. 9 provides the full MWC constructed wetlands deemed to comply checklist. A short summary of the criteria failed is provided below.

- MN10, LDS2 & LDS3: These criteria relate to landscaping considerations and cannot be met due to the nature of the project as it is being completed as part of the planning process compared to for a traditional functional design.
- MN9: The 15m offset to sediment de-watering area from the PSP boundary for S3 cannot be met in current alignment (4 m offset). Once a landscape architect is involved, they may be able to fit this better elsewhere where space is available.
- MZ3 & BY1: These are failed due to the wetland being online rather than offline as the deemed to comply checklist assumes. There is no room to make the wetland offline within the PSP boundary.
- IO4: Failed as S2 & S3 are online for the same reason as stated above for W1.

North Wetland (W1) & Sediment Ponds S2 & S3

Wetland Parameters

Normal Water Level (NWL) =				39	m AHD		
Top of Extended Detention (TED) =				39.35	m AHD		
Wetland Detention Time =				72	hours		
Shallow Marsh		38.85	to	39	m AHD		
Deep Marsh		38.65	to	38.85	m AHD		
Not Planted - deeper than			38.65	m AHD			

Macrophyte Zone Treatment Areas

Macrophyte Zone Area at NWL =						36169	m^2
Macrophyte Zone Area at TED =						39559	m^2
Volume of water stored for treatment over ED range =						**0.35**	**m**
						13252	**m^3**
Over an average ED treatment area of						**37864**	**m^2**
Detention time in macrophyte area =						**72.0**	**hours**

Wetland Macrophjyte Zone Permanent Pool Volume

Total wetland macropyhte zone:

Level		Area	Ave. Area	Delta H	Volume	Cumulative Volume	
(m AHD)		(m^2)	(m^2)	(m)	(m^3)	(m^3)	
38		2822.0				0.0	
38.4		4828.0	3825	0.40	1530	1530	
38.65		7198.0	6013	0.25	1503	3033	
38.85		25783.0	16491	0.20	3298	6331	
39		36169.0	30976	0.15	4646	10978	

Total Macrophyte Volume =	**10978**	m^3
Average Macrophyte Zone Depth =	**0.29**	m - OK
Area of Wetland below 350 mm deep =	7198.0	m^2
Area of Wetland (Excluding Sediment Ponds) =	36169	m^2
% Macrophyte Area Vegetated =	80%	OK (>80%)

Sediment Pond Parameters

Sediment Pond NWL = 40 m AHD

Sediment Pond	NWL Area	Permeant Pool Volume	Sump Volume	Detention Time
	(m^2)	(m^3)	(m^3)	(hours)
S2	1290	900	510	N/A
S3	1855	1380	810	N/A

3.2 Hancocks Gully South, Wetland/Retarding Basin W2

Addendum B details the functional design proposal. Stormy Water Solutions drawing set HCGS/SWS (Addendum B. 1) should be referred to for full details.

Hancocks Gully South, Wetland/Retarding Basin W2 NWL and TED have been set given the constraints described in Sections 2.1, 2.2 and the designed DSS pipeline system outfalls (as per the 2014 Master Plan work and MWC's preliminary DSS proposals).

The NWL in Hancocks Gully South, Wetland/Retarding Basin W2 is required to be higher than the 27.70 m AHD (frog ponds) and less than the minimum natural surface level of 28.50 m AHD.

Given the above, a NWL of 28.00 m AHD and Top of Extended Detention (TED) Level of 28.35 m AHD was set. It should be noted that there is very little room to move in regard to changing these levels as the design process moves forward.

The details of the wetland system are as detailed below. Note that Sediment Ponds S4 and S5 incorporate the same NWL as the Wetland. As such they are included in the extended detention calculations as shown.

The RORB modelling detailed in Section 4 predicts the design flows and retarding basin characteristics of the retarding basin/wetland system.

Addendum B. 8 provides the full MWC constructed wetlands deemed to comply checklist. A short summary of the criteria failed is provided below.

- MN10, LDS2 & LDS3: These criteria relate to landscaping considerations and cannot be met due to the nature of the project as it is being completed as part of the planning process compared to for a traditional functional design.
- MZ3 & BY1: These are failed due to the wetland being online rather than offline as the deemed to comply checklist assumes. There is no room to make the wetland offline within the PSP boundary.
- IO4: Failed as S2 & S3 are online and based on MWC standard drawing WG010 (B).

SOUTH WETLAND (W2) & Sediment Ponds S4 & S5

Wetland Parameters

Normal Water Level (NWL) =				28	m AHD			
Top of Extended Detention (TED) =				28.35	m AHD			
Wetland Detention Time =				72	hours	(total wetland and Sed Ponds)		
Shallow Marsh		27.85	to	28	m AHD			
Deep Marsh		27.65	to	27.85	m AHD			
Not Planted - deeper than			27.65	m AHD				

Treatment Areas (including Sediment Ponds)

Macrophyte Zone Area at NWL =						46548	m^2	
Macrophyte Zone Area at TED =						50422	m^2	
Volume of water stored for treatment over ED range =						**0.35**	**m**	
						16970	**m^3**	
Over an average ED treatment area of						**48485**	**m^2**	
Detention time in macrophyte area =						**66.5**	**hours**	

Wetland Permanent Pool Volume

Total wetland including sediment ponds

Level (m AHD)		Area (m^2)	Ave. Area (m^2)	Delta H (m)	Volume (m^3)	Cumulative Volume (m^3)		
27		3775				0		
27.4		7026	5401	0.40	2160	2160		
27.65		11172	9099	0.25	2275	4435		
27.85		31176	21174	0.20	4235	8670		
28		46548	38862	0.15	5829	14499		

Total wetland Volume including sediment ponds =						**14499**	m^3	
Total Macrophyte Volume (excluding sediment ponds)=						**11897**	m^3	
Average Macrophyte Zone Depth =						**0.28**	m - OK	
Area of Wetland below 350 mm deep =						8541	m^2	
Area of Wetland (Excluding Sediment Ponds) =						43000	m^2	
% Macrophyte Area Vegetated =						80%	OK (>80%)	

Sediment Pond Parameters

Sediment Pond	NWL Area (m^2)	Permanent Pool Volume (m^3)	Sump Volume (m^3)	ED (m)	Detention Time (hours)
S4	1910	1400	820	0.35	2.9
S5	1640	1200	700	0.35	2.5

4 Hydrologic Modelling

Hydrological Modelling using the RORB model was developed for the 2013 PSP Report. This modelling encompassed the whole PSP area including the Dore Road and Hancocks Gully catchments. To better understand the hydrology of the Hancocks Gully DSS area a new RORB model was created. The RORB Modelling is detailed below.

4.1 Pre-development RORB Model

Figure 3 details the RORB model setup for the pre-development scenario. Table 2 and Table 3 detail the tabulation of the RORB model inputs.

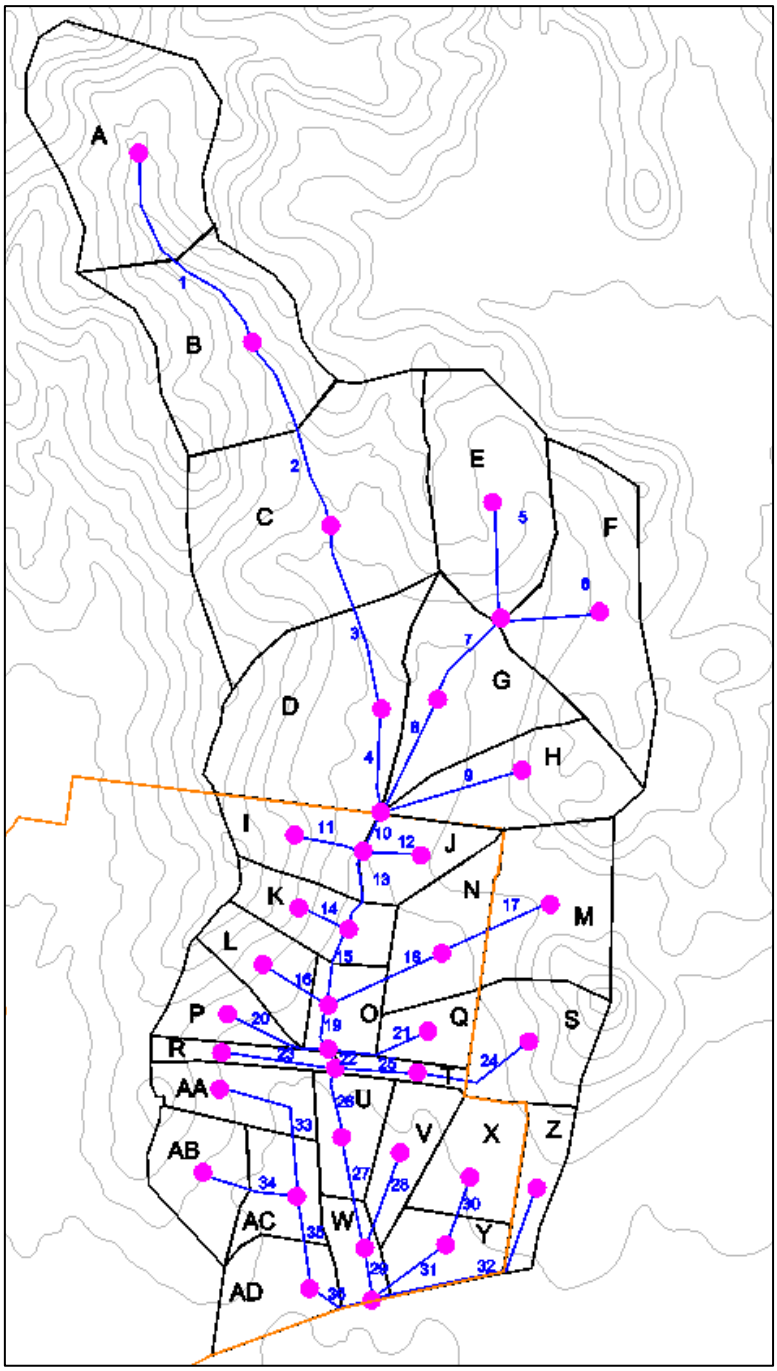


Figure 3 Pre-development RORB model

Table 2 RORB Pre-Development Model Sub Area Definition

Sub Area	Area (ha)	Area (km^2)	Fraction Imperviousness
A	68.1	0.68	0.05
B	58.4	0.58	0.05
C	99.1	0.99	0.05
D	63.9	0.64	0.05
E	49.6	0.50	0.05
F	56.0	0.56	0.05
G	40.8	0.41	0.05
H	33.9	0.34	0.05
I	20.1	0.20	0.05
J	15.5	0.15	0.05
K	19.1	0.19	0.05
L	16.2	0.16	0.05
M	37.6	0.38	0.05
N	24.3	0.24	0.05
O	12.7	0.13	0.05
P	15.3	0.15	0.05
Q	11.1	0.11	0.05
R	7.7	0.08	0.70
S	28.5	0.28	0.18
T	5.5	0.05	0.70
U	15.1	0.15	0.05
V	15.5	0.15	0.05
W	10.0	0.10	0.05
X	20.5	0.20	0.05
Y	16.3	0.16	0.05
Z	12.1	0.12	0.05
AA	18.2	0.18	0.05
AB	21.0	0.21	0.05
AC	16.3	0.16	0.05
AD	20.1	0.20	0.05
Total:	**848.2**	**8.48**	**0.06**

Table 3 RORB Model Pre-Development Reach Definition

Reach	Length (km)	Slope (%)	Reach Type
1	1.051		NATURAL
2	0.871		NATURAL
3	0.842		NATURAL
4	0.456		NATURAL
5	0.540		NATURAL
6	0.434		NATURAL
7	0.436		NATURAL
8	0.553		NATURAL
9	0.642		NATURAL
10	0.191		NATURAL
11	0.308		NATURAL
12	0.253		NATURAL
13	0.355		NATURAL
14	0.240		NATURAL
15	0.351		NATURAL
16	0.329		NATURAL
17	0.524		NATURAL
18	0.549		NATURAL
19	0.207		NATURAL
20	0.482		NATURAL
21	0.460		NATURAL
22	0.081	1.23%	PIPED
23	0.510	1.27%	PIPED
24	0.572		NATURAL
25	0.357	0.56%	PIPED
26	0.306		NATURAL
27	0.509		NATURAL
28	0.449		NATURAL
29	0.224		NATURAL
30	0.315		NATURAL
31	0.400		NATURAL
32	0.995		NATURAL
33	0.702		NATURAL
34	0.429		NATURAL
35	0.403		NATURAL
36	0.301		NATURAL

RORB is based on the following equation relating storage (S) and discharge (Q) of a watercourse.

$S = k \times Q^m$ where $k = K_c \times K_r$

The values K_c and m are parameters that can be obtained by calibration of the model using corresponding sets of data on rainfall for selected historical flows. If historical flows are unknown, values can be estimated from regional analysis or by values suggested by Australian Rainfall and Runoff (AR&R). In this case, flow gauging information was not available. However, a regional parameter set has been developed by Melbourne Water for the South East Region of Melbourne. This relationship is detailed below.

- $K_c = 1.53 \times A^{0.55} = 4.96$
- m = 0.8
- Initial loss = 10 mm (100 yr and 5 yr)
- Pervious area runoff coefficients, $C_{100,perv}$= 0.6, $C_{10,perv}$= 0.4, $C_{5,per}$ = 0.3, $C_{1,perv}$= 0.2
- "Siriwardena & Weinmann" Areal Reduction factor (A = 0 km^2)
- Pakenham Intensities

The above model produced the following results:

- Hancocks Gully Northern Catchment at PSP boundary:
    - Pre-development 100 Year ARI Flow into PSP = 14.0 m^3/s (9-hour critical duration
    - Pre-development 24 hour 100Y Year ARI Flow into PSP = 11.8 m^3/s (Volume of the 24-hour Hydrograph = 390,000 m^3)
    - Pre-development 1 Year ARI Flow into PSP = 1.3 m^3/s (36-hour critical duration)
- Hancocks Gully Northern Wetland/Retarding Basin Site (W1):
    - Pre-development 100 Year ARI Flow at Outlet (Princess Hwy) = 17.7 m^3/s (9-hour critical duration
    - Pre-development 24 hour 100 Year ARI Flow at Outlet (Princess Hwy) = 15.1 m^3/s (Volume of the 24-hour Hydrograph = 532,000 m^3)
    - Pre-development 1 Year ARI Flow at Outlet (Princess Hwy) = 1.7 m^3/s (36-hour critical duration)
- Hancocks Gully Southern Wetland/Retarding Basin Site (W2):
    - Pre-development 100 Year ARI Flow at Outlet (Princess Fwy) = 21.4 m^3/s (9-hour critical duration
    - Pre-development 24 hour 100Y Year ARI Flow at Outlet (Princess Fwy) = 18.2 m^3/s (Volume of the 24-hour Hydrograph = 709,000 m^3)
    - Pre-development 1 Year ARI Flow at Outlet (Princess Fwy) = 2.2 m^3/s (36-hour critical duration)

The .cat file for the pre-development scenario is provided in Addendum D. 1.

Stormy Water Solutions checked estimated pre-development RORB flows (at Princess Fwy) against the DSE regional flow estimate graphs (23.9 m^3/s) and the rational method (21.0 m^3/s). The RORB flow is close to both estimates, and probably less than the DSE estimate due to the low value of fraction impervious used.

The RORB model was deemed appropriate to estimate the pre-development 100 Year flow. If anything adoption of a slightly low pre-development design flow will result in a slight oversizing of the post development retarding basins below.

4.2 Post-development RORB Model

4.2.1 Model Description

Figure 4, Figure 5 and Figure 6 detail the RORB model setup for the post-development scenario. This is one model, but three figures have been used for clarity. Table 4 and Table 5 detail the tabulation of the RORB model inputs.

The post-development RORB model layout has been based on the 2016 updated PSP information provided by Council and Hancocks Gully DSS information provided by MWC. This has aided in determination of future flow paths, pipe locations and appropriate fraction impervious values etc.

The RORB parameters are as detailed in Section 4.1 above.

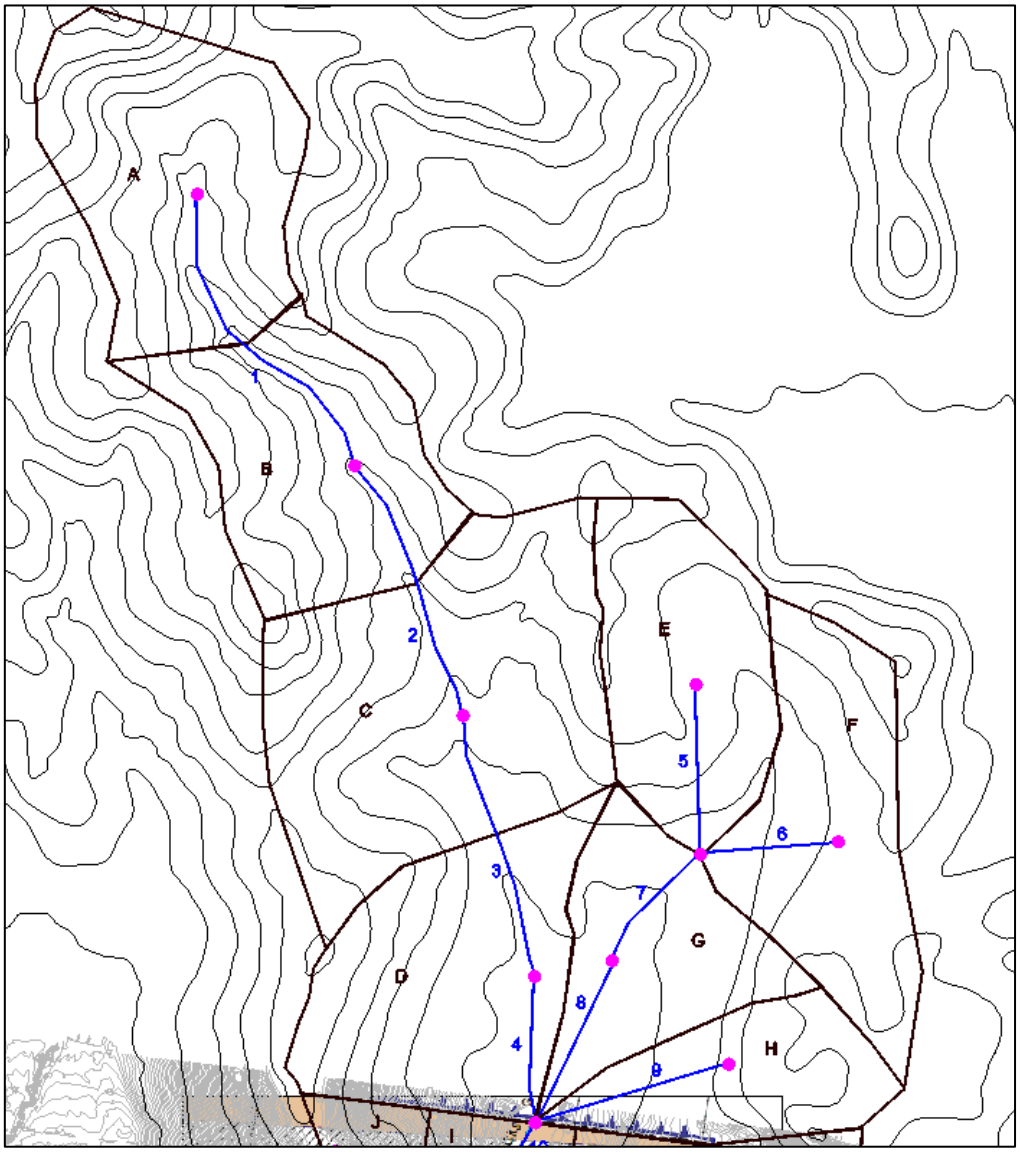


Figure 4 Upstream Catchment Post-Development RORB Model

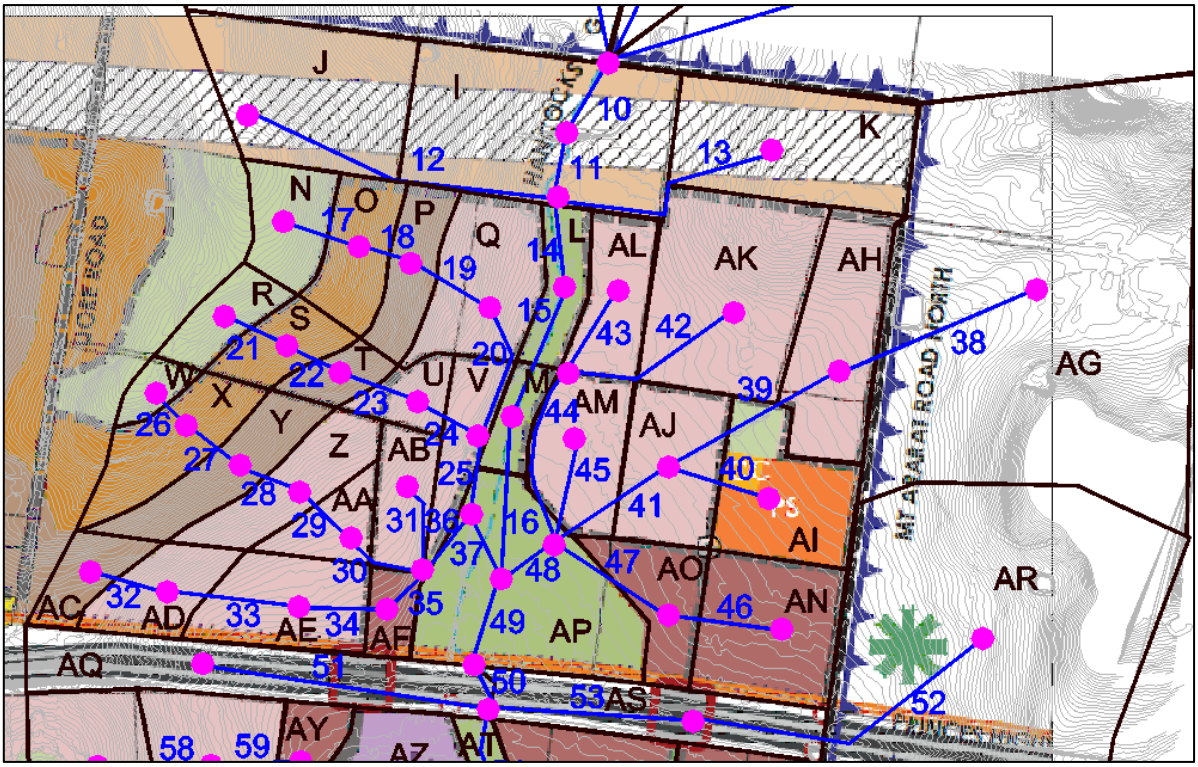


Figure 5 **Hancocks Gully North (W1) Post-Development RORB Model**

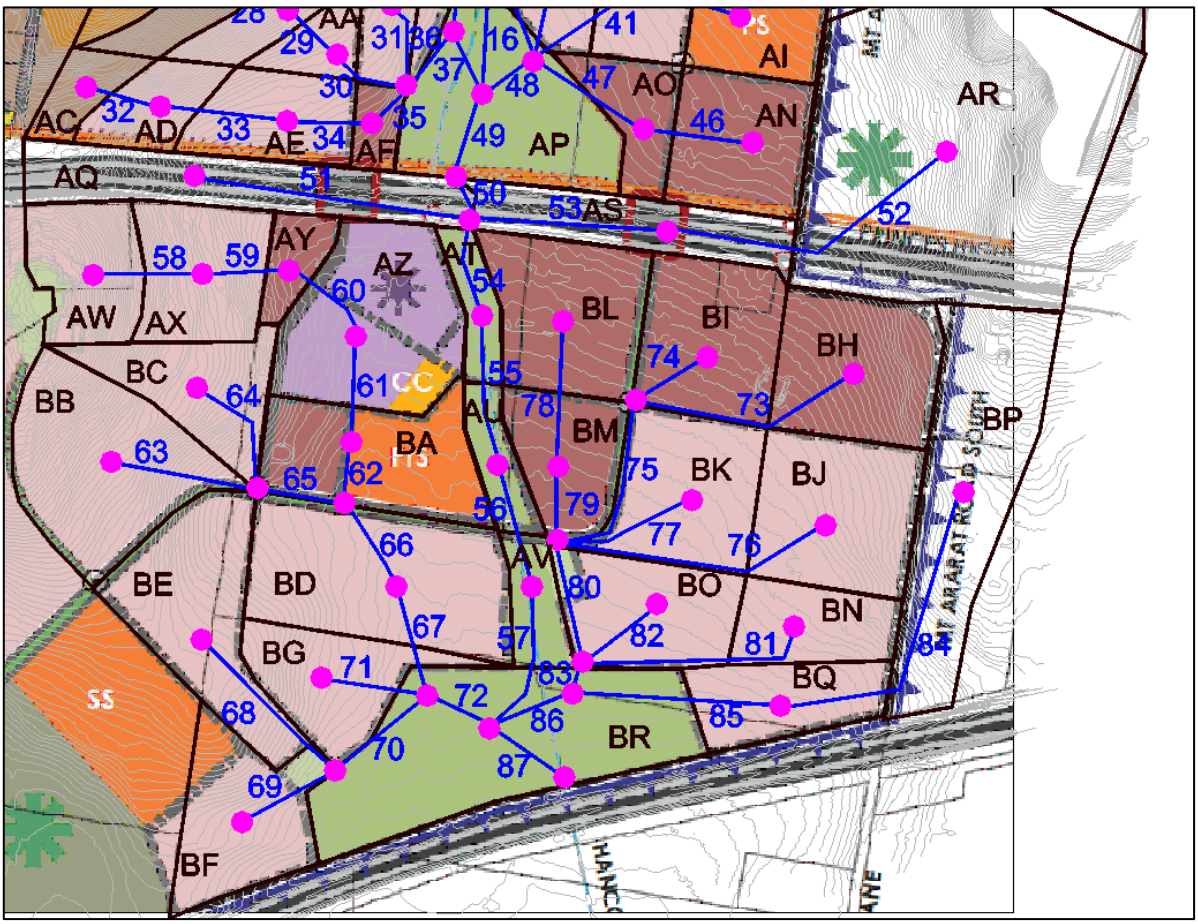


Figure 6 **Hancocks Gully South (W2) Post-Development RORB Model**

Table 4 RORB Post-Development Model Sub Area Definition

Sub Area	Area (ha)	Area (km^2)	Fraction Imperviousness
A	68.1	0.68	0.05
B	58.4	0.58	0.05
C	99.1	0.99	0.05
D	63.9	0.64	0.05
E	49.6	0.50	0.05
F	56.0	0.56	0.05
G	40.8	0.41	0.05
H	33.9	0.34	0.05
I	11.8	0.12	0.10
J	8.5	0.08	0.10
K	8.3	0.08	0.10
L	2.2	0.02	0.10
M	1.4	0.01	0.10
N	2.8	0.03	0.10
O	3.0	0.03	0.40
P	2.6	0.03	0.50
Q	5.1	0.05	0.75
R	2.0	0.02	0.10
S	2.2	0.02	0.40
T	1.3	0.01	0.50
U	1.6	0.02	0.75
V	2.5	0.02	0.75
W	0.9	0.01	0.10
X	3.7	0.04	0.40
Y	3.5	0.04	0.50
Z	3.4	0.03	0.75
AA	1.9	0.02	0.75
AB	2.4	0.02	0.75
AC	1.8	0.02	0.50
AD	2.8	0.03	0.75
AE	4.4	0.04	0.75
AF	1.3	0.01	0.80
AG	37.6	0.38	0.05
AH	6.1	0.06	0.75
AI	5.4	0.05	0.59
AJ	4.7	0.05	0.75
AK	9.4	0.09	0.75
AL	3.3	0.03	0.75
AM	4.2	0.04	0.75
AN	6.0	0.06	0.80
AO	3.1	0.03	0.80
AP	10.0	0.10	0.50
AQ	7.6	0.08	0.70
AR	28.5	0.28	0.05
AS	5.4	0.05	0.70
AT	1.9	0.02	0.10
AU	2.1	0.02	0.10
AV	2.2	0.02	0.10
AW	4.8	0.05	0.75
AX	5.6	0.06	0.75
AY	1.8	0.02	0.80
AZ	9.5	0.10	0.90
BA	7.5	0.07	0.74
BB	9.8	0.10	0.75
BC	5.2	0.05	0.75
BD	11.1	0.11	0.75
BE	8.3	0.08	0.75
BF	6.4	0.06	0.69
BG	5.3	0.05	0.75
BH	7.6	0.08	0.80
BI	7.0	0.07	0.80
BJ	7.9	0.08	0.75
BK	7.2	0.07	0.75
BL	7.4	0.07	0.80
BM	4.7	0.05	0.80
BN	4.0	0.04	0.75
BO	6.1	0.06	0.75
BP	12.1	0.12	0.05
BQ	4.5	0.05	0.75
BR	14.9	0.15	0.50
Total:	**847.2**	**8.47**	**0.25**

Fraction impervious above based on: Standard Residential = 0.75, Medium Density Residential = 0.8, Low Density Residential = 0.2, School = 0.7, Major Road = 0.7, Minor Road = 0.6, Commercial = 0.9, Open Space = 0.1, Wetland = 0.5

Table 5 RORB Model Post-Development Reach Definition

Reach	Length (km)	Slope (%)	Reach Type
1	1.051		NATURAL
2	0.871		NATURAL
3	0.842		NATURAL
4	0.456		NATURAL
5	0.540		NATURAL
6	0.434		NATURAL
7	0.436		NATURAL
8	0.553		NATURAL
9	0.642		NATURAL
10	0.144		NATURAL
11	0.110		NATURAL
12	0.551		NATURAL
13	0.436		NATURAL
14	0.156	0.96%	NATURAL
15	0.245	0.82%	NATURAL
16	0.279	0.90%	NATURAL
17	0.139	12.27%	PIPED
18	0.095	12.05%	PIPED
19	0.158	9.17%	PIPED
20	0.244	3.08%	PIPED
21	0.124	10.91%	PIPED
22	0.096	11.99%	PIPED
23	0.140	8.60%	PIPED
24	0.118	8.08%	PIPED
25	0.136	1.11%	PIPED
26	0.076	13.76%	PIPED
27	0.121	11.96%	PIPED
28	0.115	8.26%	PIPED
29	0.120	4.99%	PIPED
30	0.141	4.62%	PIPED
31	0.160	3.76%	PIPED
32	0.137	6.95%	PIPED
33	0.230	2.83%	PIPED
34	0.149	2.01%	PIPED
35	0.094	1.06%	PIPED
36	0.124	0.40%	PIPED
37	0.127	0.79%	EX/UNLINED
38	0.375	2.40%	EX/UNLINED
39	0.334	1.95%	PIPED
40	0.179	1.12%	PIPED
41	0.237	1.68%	PIPED
42	0.333	1.65%	PIPED

Reach	Length (km)	Slope (%)	Reach Type
43	0.165	0.91%	PIPED
44	0.323	1.08%	PIPED
45	0.184	1.09%	PIPED
46	0.200	0.75%	PIPED
47	0.232	0.86%	PIPED
48	0.111	0.90%	EX/UNLINED
49	0.146	1.37%	DROWNED
50	0.081	0.12%	PIPED
51	0.510	1.37%	PIPED
52	0.572	2.53%	EX/UNLINED
53	0.357	0.56%	PIPED
54	0.185	1.62%	NATURAL
55	0.274	0.91%	NATURAL
56	0.232	0.86%	NATURAL
57	0.276	0.91%	NATURAL
58	0.191	9.17%	PIPED
59	0.147	6.46%	PIPED
60	0.169	1.48%	PIPED
61	0.182	1.10%	PIPED
62	0.115	0.65%	PIPED
63	0.256	5.66%	PIPED
64	0.233	5.58%	PIPED
65	0.159	1.73%	PIPED
66	0.175	1.00%	PIPED
67	0.210	0.71%	PIPED
68	0.337	3.42%	PIPED
69	0.192	1.04%	PIPED
70	0.213	0.24%	PIPED
71	0.188	2.12%	PIPED
72	0.123	0.41%	EX/UNLINED
73	0.437	1.26%	PIPED
74	0.148	1.02%	PIPED
75	0.340	1.03%	PIPED
76	0.515	0.78%	PIPED
77	0.268	0.75%	PIPED
78	0.263	1.14%	PIPED
79	0.136	0.74%	PIPED
80	0.228	0.66%	PIPED
81	0.428	0.70%	PIPED
82	0.175	0.71%	PIPED
83	0.058	0.86%	PIPED
84	0.597	2.68%	EX/UNLINED
85	0.367	0.14%	PIPED
86	0.147	0.68%	EX/UNLINED
87	0.163	0.31%	DROWNED

4.2.2 Model Verification

Stormy Water Solutions checked estimated post-development RORB flows (at Princes Fwy) against the DSE regional flow estimate graphs and the rational method. The results are detailed in Table 6 below.

Table 6 Post-Development Model Verification

Method	Q_{100} (m^3/s)
RORB (Reaches 14, 15, 16, 54, 55, 56 & 57 Natural)	24.1
RORB (Reaches 14, 15, 16, 54, 55, 56 & 57 Ex/Unlined)	32.1
Rational	44.8
Regression	46.9

NB – Flow check only – not to be used for design purposes (no storages modelled).

At first glance, the RORB model does not seem to be verified by the two comparison methods. However, SWS feel that the results shown in Table 6 do verify the RORB model due to the lack of timing present in both comparison methods. RORB is able to time the peak flows better due to the model setup (Section 4.2.1). The nature of the site (and hence the model), being long and skinny, mean that the southern peak flow is through the system before the peak from the northern development reaches the outlet. As such there is an extended peak (>20 m^3/s) over 2.5 hours in the 9-hour critical storm duration rather than a distinct peak expected from the verification methods.

For comparison SWS has compared the difference between having the “trunk” reaches (14, 15, 16, 54, 55, 56 & 57) modelled as both Natural and Ex/Unlined. With these reaches as Ex/Unlined, the model produced flows closer to the comparison methods. However, as the vegetated channels are going to be heavily vegetated (n = 0.15) as discussed in Section 7, the natural reach type is a more accurate representation of what will physically be built in the channel. As such the Natural reach type was selected for further modelling.

SWS feel if the RORB model conveyed the flows so that the timing of the northern and southern peaks matched, the RORB model would be close to the verification methods.

4.2.3 Wetland/Retarding Basin Retardation

Both wetland/retarding basins have been designed to work in unison so that all flow conditions are met at the outlet of the PSP. The following sections provide a summary of what each wetland/retarding basin has been designed to achieve within RORB.

4.2.3.1 Hancocks Gully North (W1)

Hancocks Gully North (W1) wetland/retarding basin system provides flood storage and treatment for flows from the external northern catchment and all catchments north of Princess Highway. The outflow relationship of this system is constrained by the two gas mains located north of Princess

Highway. Using the initial information provided on the gas mains (levels to be proven) SWS has developed a possible longitudinal section for the high flow culverts as shown in drawing HCGN/SWS/ 4. The system requires the constructed conveyance system to incorporate an internal depth no greater than 1.05 m. Due to this constraint and the ease at which Hancocks Gully South (W2) (Section 4.2.3.2) can attenuate the outflow to the 1 Year ARI, SWS, has not designed this wetland/retarding basin to attenuate the 1 Year ARI post-development flow to the 1 Year ARI pre-development flow.

The wetland/retarding basin outlet has been configured to:

- Detain Stormwater for treatment for 72 hours between the levels of 39.00 and 39.35m AHD via the use of weir controls in the wetland outlet pit,
- Retard 100 Year ARI flows to less than the capacity of the princess freeway culverts (20 m^3/s) via the use of five 1.05 m Ø outlet pipes (US IL = 38.3 m AHD, DS IL = 37.4 m AHD), and
- For extreme flows in excess of the 100 Year ARI event, provide a 40 m spillway between the 100 Year flood level (40.5 m AHD) and the embankment crest (41.0 m AHD).

The outlet configuration is as depicted in drawing HCGN/SWS/ 3. Weir, orifice and culvert calculations were used to size the outlet system. The retarding basin storage characteristics were derived using the design contours as detailed in Addendum A. 1. The resulting Stage/Storage/Discharge (SSD) relationship is as detailed in Table 7 below.

Table 7 Hancocks Gully North (W1) Wetland/Retarding Basin SSD Relationship

Level (m)	Storage (m^3)	Flow (m^3/s)	Notes:
39.00	0	0.00	NWL
39.35	13252	0.05	Wetland ED control
39.50	19680	1.52	Outlet pit acting as a as weir
39.60	24736	3.20	
39.70	29792	5.27	
39.80	34848	7.67	
39.90	39904	10.34	
40.00	44960	11.98	Five 1050 mmØ outlet culverts acting under outlet culvert control
40.10	50681	12.56	
40.20	56402	13.12	
40.30	62123	13.65	
40.40	67844	14.16	
40.50	73566	14.66	
40.60	79744	17.67	Spillway flow
40.80	92100	28.73	
41.00	104456	44.27	

4.2.3.2 Hancocks Gully South (W2)

Hancocks Gully South (W2) is an online wetland/retarding basin system treating the catchments South of Princes Hwy. The outlet configuration is constrained by the existing frog ponds (Section 2.1).

The wetland/retarding basin outlet has been configured to:

- Detain Stormwater for treatment for 72 hours between the levels of 28.00m AHD and 28.35m AHD via the use of weir controls in the wetland outlet pit,
- Retard the total catchment 1 Year ARI flow via triple 600mmØ outlet culverts between the levels of 28.35m AHD and 29.30m AHD,
- Allow flows in excess of the 1 Year ARI flow overtop a 30m long spillway at 29.30m AHD, and
- Allows for 300mm freeboard from the 100 Year ARI Water level of 29.75m AHD to the embankment crest at 30.05m AHD.

Due to the nature of the developed catchment, the timing of the peaks for the North and South catchments do not align for the developed conditions. As such given the retardation from the North Wetland/Retarding Basin (Section 4.2.3.1), no retardation of the 100 Year ARI flow is required in the Southern Wetland/Retarding Basin.

The outlet configuration is as depicted in Addendum B. 1 and drawing HCGS/SWS/ 2. Weir, orifice and culvert calculations were used to size the outlet system. The retarding basin storage characteristics were derived using the design contours as detailed in Addendum B. 1. The resulting Stage/Storage/Discharge (SSD) relationship is as detailed in Table 8 below.

Table 8 Hancocks Gully South (W2) Wetland/Retarding Basin SSD Relationship

Level (m)	Storage (m^3)	Flow (m^3/s)	Notes:
28.0	0	0.00	NWL
28.35	16942	0.06	Wetland ED Control
28.7	36914	0.38	1 Year ARI Outler Control
29.0	54863	0.69	
29.1	61728	1.06	
29.2	68593	1.24	
29.3	75458	1.40	
29.4	82324	3.43	Spillway to convey flows in excess of 1 Year ARI flow
29.5	89189	7.03	
29.6	97419	11.62	
29.7	105648	17.04	
29.8	113878	23.15	
29.9	122108	29.87	
30.0	130338	37.17	

4.2.4 Post-development RORB Results

Table 9 details the post-development RORB results upstream of Princess Hwy (Hancocks Gully North, W1). Table 10 details the post-development RORB flows upstream of Princess Fwy (Hancocks Gully South, W2). The resultant flows were used to assess wetland velocity implications, vegetated channel flood levels etc.

It should be noted that this system configuration is essentially using W1 and the heavily vegetated nature of the vegetated channels V1 and V2 to control the 100 Year ARI flow at the catchment outlet. W2 is not required to control the 100 Year ARI flow as RORB indicates that the condition on the 100 Year ARI flow will already be met via the provision of W1 and the upstream vegetated channels V1 and V2. Therefor the 1 Year ARI flow at the catchment outlet is met using only W2.

The .cat file for the post-development scenario is provided in Addendum

Table 9 Post Development RORB Results Hancocks Gully North

Location	100 Year ARI	10 Year ARI	5 Year ARI	1 Year ARI
Into Northern PSP Boundary (Reach 14)	15.0 m^3/s (9 hr)	5.7 m^3/s (9 hr)	3.6 m^3/s (9 hr)	1.4 m^3/s (36 hr)
Vegetated Waterway, Northern Hancocks Gully (Reach 16)	14.6 m^3/s (9 hr)	5.5 m^3/s (9 hr)	3.5 m^3/s (9 hr)	1.4 m^3/s (36 hr)
North West Sediment Pond (S2) Inlet (Reach 37)	7.6 m^3/s (20 min)	3.7 m^3/s (2 hr)	2.9 m^3/s (2 hr)	1.5 m^3/s (2 hr)
North East Sediment Pond (S3) Inlet (Reach 48)	9.8 m^3/s (2 hr)	4.4 m^3/s (2 hr)	3.4 m^3/s (2 hr)	1.8 m^3/s (2 hr)
Total Retarding Basin Inflow (Critical Event)	17.9 m^3/s (9 hr), Volume of Inflow HG = 384,000 m^3	8.5 m^3/s (2 hr)	6.6 m^3/s (2 hr)	3.4 m^3/s (2 hr)
Total Retarding Basin Outflow (Critical event)	14.6 m^3/s (9 hr)	6.9 m^3/s (12 hr)	4.8 m^3/s (12 hr)	2.0 m^3/s (36 hr)
Retarding Basin Storage (Critical event)	72,700 m^3	33,200 m^3	28,600 m^3	21,200 m^3
Retarding Basin Flood Level (Critical event)	40.49 m AHD	39.77 m AHD	39.68 m AHD	39.53 m AHD
Total Retarding Basin Outflow (24 hr Event)	13.2 m^3/s (24 hr), Volume of Inflow HG = 561,000 m^3			
Retarding Basin storage (24 hr event)	57,000 m^3			
Retarding Basin Flood Level (24 hr event)	40.21 m AHD			
File: 1603_Han_Gul_PostDev_V5N_V3S_SSD_15Aug16.cat				

Table 10 Post Development RORB Results Hancocks Gully South

Location	100 Year	10 Year	5 Year	1 Year
South of Princes Hwy (Reach 54)	15.3 m^3/s (12 hr)	7.4 m^3/s (12 hr)	5.1 m^3/s (12 hr)	2.2 m^3/s (36 hr)
Vegetated Waterway, Southern Hancocks Gully (Reach 57)	15.2 m^3/s (12 hr)	7.1 m^3/s (12 hr)	4.9 m^3/s (12 hr)	2.1 m^3/s (36 hr)
South West Sediment Pond (S4) Inlet (Reach 72)	12.9 m^3/s (25 min)	6.7 m^3/s (25 min)	5.4 m^3/s (2 hr)	3.2 m^3/s (2 hr)
South East Sediment Pond (S5) Inlet (Reach 86)	8.5 m^3/s (25 min)	4.4 m^3/s (2 hr)	3.6 m^3/s (2 hr)	2.1 m^3/s (2 hr)
Total Retarding Basin inflow (Critical Event)	22.6 m^3/s (25 min), Volume of Inflow HG = 90,300 m^3	11.5 m^3/s (1 hr)	9.3 m^3/s (2 hr)	5.4 m^3/s (2 hr)
Total Retarding Basin Outflow (Critical event)	20.0 m^3/s (12 hr)	8.5 m^3/s (12 hr)	5.8 m^3/s (30 hr)	1.7 m^3/s (48 hr)
Retarding Basin Storage (Critical event)	110,000 m^3	91,900 m^3	86,800 m^3	76,300 m^3
Retarding Basin Flood Level (Critical event)	29.75 m AHD	29.53 m AHD	29.46 m AHD	29.31 m AHD
Total Retarding Basin Outflow (24 hr Event)	17.1 m^3/s (24 hr), Volume of Inflow HG = 775,000 m^3			
Retarding Basin storage (24 hr event)	106,000 m^3			
Retarding Basin Flood Level (24 hr event)	29.70 m AHD			
File: 1603_Han_Gul_PostDev_V5N_V3S_SSD_15Aug16.cat				

The above indicates the following:

- The peak 100 Year ARI flow from the future development (19.9 m^3/s) does not exceed the pre-development flow rate (21.4 m^3/s) at the PSP outfall point.

- The peak 24-hour 100 Year ARI flow from the future development (17.1 m^3/s) does not exceed the pre-development flow rate for a storm of this duration (18.2 m^3/s) at the PSP outfall point (KWRFPD requirement)

- The total storage of both wetland/retarding basins (57,000 m^3 + 106,000 m^3 = 163,000 m^3) in the 24-hour 100 Year ARI event which is greater than the difference between the expected post-development and pre-development 24-hour 100 Year ARI flow volume at the PSP outfall point (775,000 m^3 – 709,000 m^3 = 66,000 m^3). This will ensure no increased flood effect within the KWRFPD during a 24-hour 100 Year ARI event.

- The peak 1 Year ARI flow from the future development (1.7 m^3/s) does not exceed the pre-development flow rate (2.2 m^3/s) at the PSP outfall point.

The above shows all conditions from Section 2.5 have been met.

4.2.5 Fill Requirements

4.2.5.1 Hancocks Gully North (W1)

At this stage it is assumed that all future roads adjacent to the retarding basin must be filled to at least 40.50m AHD and all future lots to 41.10m AHD.

4.2.5.2 Hancocks Gully South (W2)

At this stage it is assumed that all future roads adjacent to the retarding basin must be filled to at least 29.75m AHD and all future lots to 30.35m AHD.

4.2.6 Extreme Event Preliminary Assessment

At this stage, extreme flows in Hancocks Gully North (W1) wetland/retarding basin are proposed to be conveyed over a 40m spillway (spillway crest = 40.50m AHD). Extreme flows in Hancocks Gully South (W2) wetland/retarding basin are proposed to be conveyed over a 30m spillway (spillway crest = 29.30m AHD).

To provide a preliminary assessment of the spillway and embankment design, the 500 Year ARI events have been simulated using the RORB model.

To reflect the nature of the 500 Year ARI events, the IL parameter has been set to 5mm and the pervious area runoff coefficient has been set to 0.8.

The analysis found that the 9-hour 500 Year ARI event was critical for both wetland/retarding basins.

Hancocks Gully North (W1) does not overtop in the critical 500 Year ARI event with the modelling indicating the peak outflow 31.6 m^3/s at a level of 40.84m AHD. This level is approximately 150mm below the proposed embankment crest at 41.00m AHD.

Hancocks Gully South (W2) does not overtop in the critical 500 Year ARI event with the modelling indicating the peak outflow 36.7 m^3/s at a level of 29.90m AHD. This level is approximately 150mm below the proposed embankment crest at 30.05m AHD.

An ANCOLD assessment is required to determine the appropriate extreme flow provision for the basin. If this ‘design’ event is required to be greater in magnitude than the 500 Year ARI event, the spillway could be widened and/or the embankment raised slightly.

5 Sediment Pond Design

The wetland designs aim to provide primary treatment to flow conveyed in the incoming pipes via the use of two large sediment ponds at each wetland system.

Primary treatment is concerned with the collection of course sediment greater in size than 125 micrometres (0.125 mm). Particles smaller in size than this pass through the sediment ponds areas and are treated downstream in the wetland system. Sediment ponds typically are required to be cleaned out every 5 years. As such, an important part of the functional design process is ensuring there is enough space for maintenance access and dewatering activities. This has been accounted for as per the calculations in Addendums A. 3 and B.3 and the functional design drawing sets (Addendums A. 1 and B. 1).

In regard to the design of all sediment ponds (S2, S3, S4 and S5) it should be noted that, as described in the functional design drawings HCGN/SWS/ 2, HCGS/SWS/ 2, Addendum A. 3 and Addendum B.3, they all:

- Have been designed to ensure the sediment build up height in 5 years is lower than 350 mm below the sediment pond NWL,
- Are located online to the DSS pipelines which they are treating water from,
- Capture greater than 95% of course particles ≥ 125 µm diameter for the peak three month ARI flow,
- Provide adequate sediment storage volume to store 5 years of sediment (volume between the base level and 'NWL – 350 mm'),
- Ensure that the velocity through the sediment pond during the peak 100 Year ARI event is ≤ 0.5 m/s. (The flow area has been calculated using the 100 Year ARI flood levels estimated in the functional design calculations and the narrowest width of the sediment pond, at NWL, between the inlet and overflow outlet as these basins do not form part of the 'active' flood storage in the basin).
- Incorporate provision in regard to space allocation to ensure that the sediment ponds can incorporate 4 m access tracks (1 in 15 battering (max) above NWL, 1 in 5 (max) below NWL),
- Incorporate concrete bases to MWC requirements, and
- Incorporate provision in regard to providing dedicated sediment dewatering areas which:
    - Are accessible from the maintenance ramp,
    - Are able to contain all sediment removed from the sediment pond accumulation zone every 5 years, spread out at a depth of 500mm, and
    - Are above the estimated peak 10 Year ARI water level.

Isolation of wetland and sediment ponds is possible by blocking off inlet headwalls and pits. Drawings HCGN/SWS/ 6 and HCGS/SWS/ 4 detail how this can occur.

6 Stormwater Pollutant Modelling

The performance in regard to stormwater pollutant retention of the PSP wetland systems was analysed using the MUSIC model, Version 6. Subareas and fraction imperviousness are as detailed in the RORB model (Section 4.2).

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 for Narre Warren North (1984 - 1993) at 6 minute intervals was utilised. This is the reference gauge defined by MWC for this area of Melbourne. Figure 7 details the model layout developed.

The modelled element characteristics are as detailed in Section 3 and Section 5. As required by MWC for similar systems (e.g. Deep Creek South Wetland, December 2015), all wetland as sediment ponds have been modelled separately with details as shown in Section 5. Furthermore, the heavily vegetated, Hancocks Gully waterway has been assumed to provide no treatment to the system.

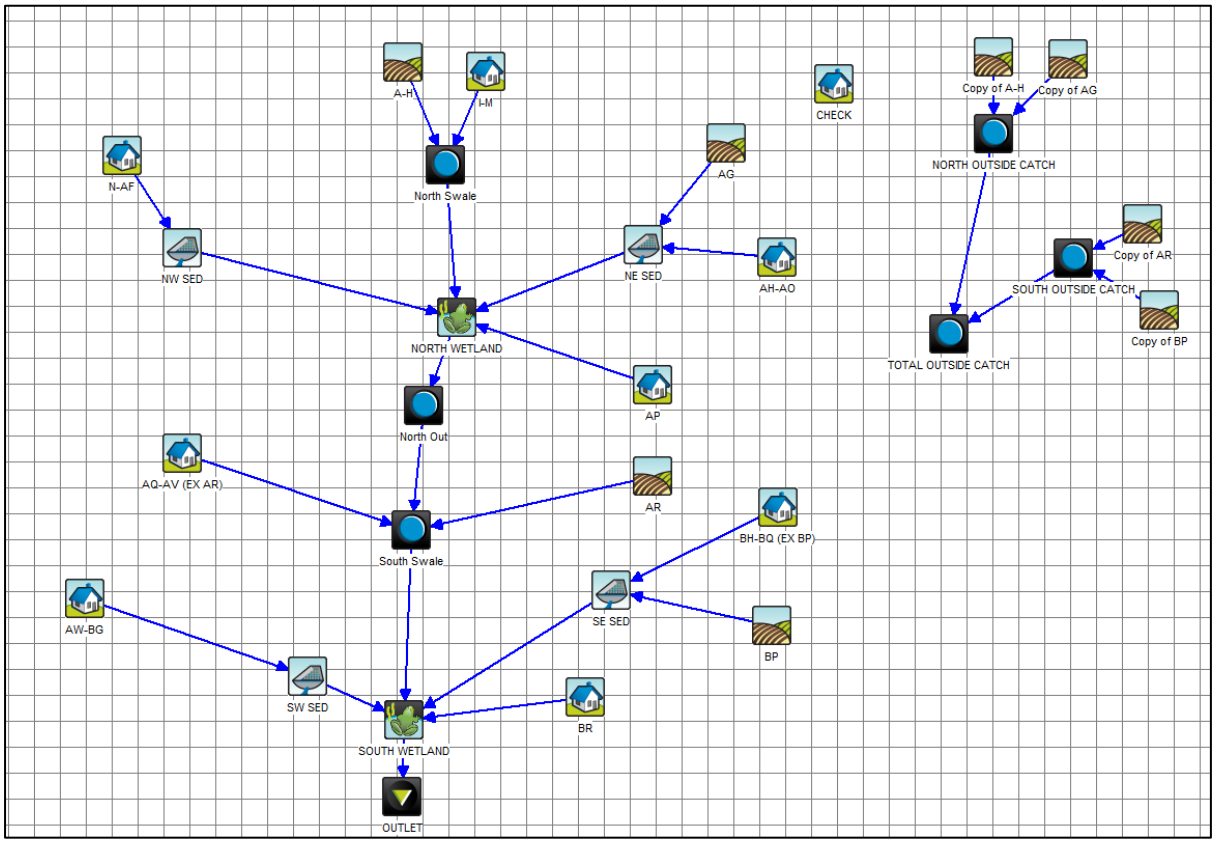


Figure 7 MUSIC Model

Table 11 MUSIC Results at DSS Outlet

Flow (ML/yr)	6%
Total Suspended Solids (kg/yr)	100%
Total Phosphorus (kg/yr)	93%
Total Nitrogen (kg/yr)	52%
Gross Pollutants (kg/yr)	100%

The current best practice requirements of 80% TSS, 45% TP and 45% TN retention can be met by proposed wetland system. As such, the functional design of the elements meets the requirements of the current Hancocks Gully DSS. It should be noted that the results shown in Table 11 have been derived by subtracting the mean annual loads from the contributing external catchments from the treatment train effectiveness at the outlet as shown in Addendum B. 5.

In addition to the above “best practice” water quality requirements, various authorities are advocating “over treatment” of Stormwater in the Westernport catchments given the RAMSAR wetland system located within Westernport Bay. MWC have advised, that to meet the intent of the State Environment Protection Policy (SEPP) Schedule F8, 93% retention of TSS is required in this catchment.

Table 11 shows that the current proposal as outlined in Section 3 meet the SEPP, Schedule F8 requirement of 93% TSS retention can be met with the proposed design. Furthermore, the Stormwater harvesting initiatives advocated within the “Pakenham East stormwater Harvesting Investigation, Project Report, Dalton Consulting Engineers, May 2015” will aid in further meeting objectives in excess of current best practice once issues are adressed by MWC. Certainly, stormwater harvesting (if occuring) should increase TSS retention. This is required to be confirmed during the detailed design stage of the project.

6.1 Current Stormwater Harvesting Proposal

Dalton Consulting Engineers (DCE) has prepared a report on possible Stormwater harvesting initiatives in Pakenham East titled “Pakenham East Stormwater Harvesting Investigation, Project Report, Dalton Consulting Engineers, May 2015”.

The DCE report recommends options 3-G and 3-P. The diference in both options is in how they convey the stormwater that is harvested from Wetland 2 (Southern Hancocks Gully) and Wetland 4 (Ryans Road) to Bald Hill Reservoir for re-use. Option 3-G gas a gravity fed pipeline at a considerable depth (approximatly 4m when cross the gass easement at the Ryan Road Wetland (Chainage 2400)) between Wetland 2 and Bald Hill Reservoir. Option 3-P has a primed line between Wetland 2 and Bald Hill Reservoir to reduce depth to the pipe along the line to help with construction (approximatly 1.7m when cross the gass easement at the Ryan Road Wetland (Chainage 2400)).

SWS has identified issues with the proposed options in the DCE report as follows:

- The general arrangement of the outlet structures appears to be drawing water down to well below NWL in both W2 and W4.
- No consideration has been given to the Origin Gas Easement (that intersects the Ryan Road Wetland/Retarding Basin, W4)) and how each option will cross the easement. No costing has been provided for the crossing in either option. SWS feel that the gravity option (3-G) will work (in principle) with adequate cover provided underneath the gas main (however APA advised they prefer going over the top of the gas main). The primed option (3-P) will need to be look at as the current pipe location is very close to the gas main. However, as the system is primed it should not be too hard to raise the pipe and still provide adequate cover to the gas main and adequate cover to the surface of the harvesting pipe if the harvesting pipe is run underneath the proposed embankment as shown in RYANRD/SWS/1.

Future harvesting offtake design <u>MUST</u> take into account the design levels off the pump pit to ensure <u>NONE</u> of the drainage/WSUD initiatives of the wetland are compromised.

7 Hancocks Gully Vegetated Waterway Design

Two vegetated waterways have been designed, one upstream of Hancocks Gully North (W1) wetland/retarding basin (between the northern PSP boundary and W1) and one upstream of Hancocks Gully South (W2) wetland/retarding basin (between Princess Highway and W2).

Both V1 and V2 have been designed to meet the main objectives for waterways corridors in green field development areas as described by “Waterway Corridors, Guidelines for greenfield development areas within the Port Phillip and Westernport Region, MWC, October 2013”. The main objectives are:

- Objective 1: To protect, enhance or restore river health and biodiversity
- Objective 2: To enable some complementary use of waterways for recreational purposes and infrastructure (if appropriate) while maintaining primary river health, flood protection and biodiversity functions, and
- Objective 3: To provide effective flood protection

Currently, both V1 and V2 are the simple drain the farmer cut with very little fauna as shown in Figure 8 and Figure 9. SWS has taken this as an opportunity to increase the amount of fauna within the region and have such designed both channels as heavily vegetated. The other advantage of having a heavily vegetated channel (beds and banks) is that the system becomes maintenance free. As a maintenance free system, species such as swamp scrub etc may invade the channel over time. In fact, propagation of dense swamp scrub (or similar) should be encouraged to maximise the reach flood storage effect of the channels as advocated in Section 4.2. Utilising a Mannings n of 0.15 for this means that, should the system become totally vegetated with dense species, there will be <u>no</u> flood impact. With this heavily vegetated channel arrangement, SWS has assumed that Objective 1 has been met.

Figure 8 Existing Condition of Hancocks Gully North Vegetated Waterway V1 (Source: Google Maps)

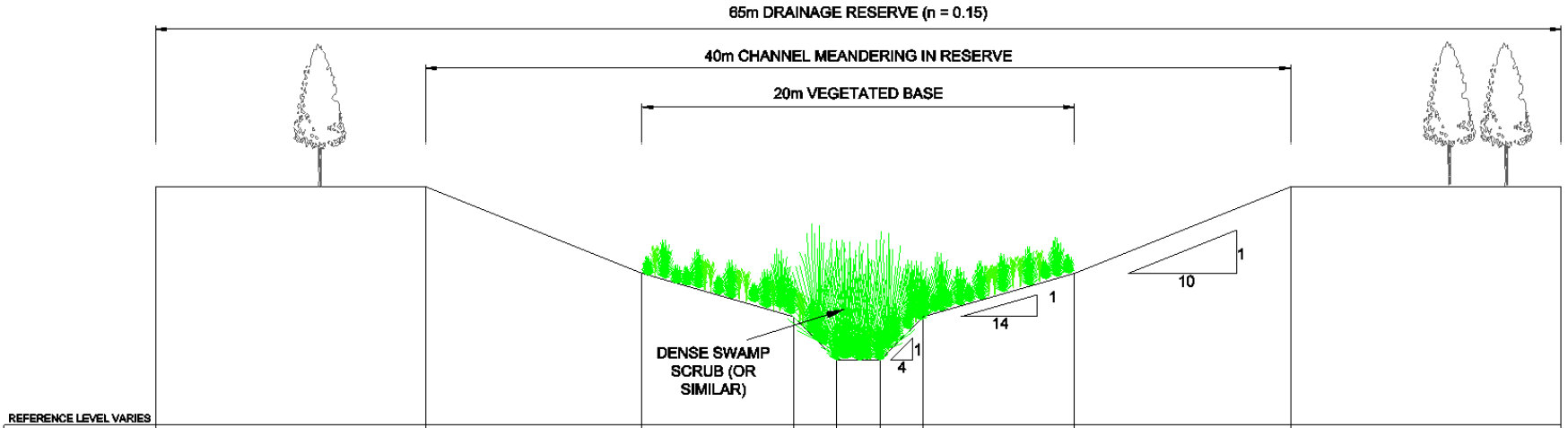
Figure 9 Existing Condition of Hancocks Gully South Vegetated Waterway V2 (Source: Google Maps)

Both systems have been designed to incorporate the same general cross section of a 40m channel meandering in the 65m drainage reserve as shown in Figure 10.

Each system has been modelled separately as described in Addendum C. 1 and C. 2.

Both designs meet the sliding scale for corridor width (Table 4, Waterway Corridors, Guidelines for greenfield development areas within the Port Phillip and Westernport Region, MWC, October 2013) with both designs having a hydraulic width (100 Year ARI width) of approximately 35 m. The guidelines recommend having a 55 m corridor for this hydraulic width. As shown in Figure 10, both channels have been designed in a 65 m reserve. As the SWS design is in a 65 m drainage corridor, it has been assumed to meet Objective 2 as more space has been given for possible uses around the waterway.

The proposed design is able to convey the 100 Year ARI flow within the 40m meandering channel for the total length of both channel sections as detailed in Addendum C. 1 and C. 2. Therefore, Objective 3 has been met by the proposed channel design for both V1 and V2.



CHAINAGE (m)	0	x	x+10	x+17	x+19	x+21	x+23	x+30	x+40	65
DESIGN SURFACE (m AHD)	VARIES	y	y-1	y-1.5	y-2	y-2	y-1.5	y-1	y	VARIES

Figure 10 General Vegetated Channel Cross Section

8 Further Work Required in Detailed Design Stage

The following further work is required as part of the design process going forward.

- Review of the service proving is required at the detailed design stage of the project.,
- Detailed survey of freeway frog ponds must occur to ensure that the assumed normal water level in the northern frog pond is valid,
- Ecological and archaeological studies are required to ensure no adverse impacts to existing site values,
- MWC should conduct an ANCOLD assessment on this design to ensure the embankment and spillway provisions are adequate.
- Soil tests should be performed to confirm the assumption that the existing clayey soil (once compacted) will be suitable without an additional liner.
- Review of the ""Pakenham East Stormwater Harvesting Investigation, Project Report, Dalton Consulting Engineers, May 2015" report as detailed in Section 6.1.

Council and MWC may wish to consider reducing the retarding basin site space in W2 slightly via reducing the eastern site boundary by 90 meters as detailed in HCGS/SWS/ 2.

9 Abbreviations, Descriptions 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. A 100 Year ARI event has a 1 in 100 chance of occurring in any one year.
DSS	Development Services Scheme – catchment drainage strategies developed, implemented and run by MWC.
Grassed Swale	A small shallow grassed drainage line designed to convey stormwater discharge. A complementary function to the flood conveyance task is its WSUD role (where the vegetation in the base acts as a treatment swale).
Hectare (ha)	10,000 square metres
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 (e.g. 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
MWC	Melbourne Water Corporation
Retarding basin	A flood storage dam which is normally empty. May contain a lake or wetland in its base
Normal Water Level	Water level of a wetland or pond defined by the lowest invert level of the outlet structure
RORB	Hydrologic computer program used to calculate the design flood flow (in m^3/s) along a stormwater conveyance system (e.g. waterway)
Sedimentation basin (Sediment pond)	A pond that is used to remove coarse sediments from inflowing water mainly by Settlement processes.
SWMP	Stormwater Management Plan
Surface water	All water stored or flowing above the ground surface level
Total Catchment Management	A best practice catchment management convention which recognises that waterways and catchments do not stop at site boundaries and decisions relating to surface water management should consider the catchment as a whole
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
Extended Detention	Range of water level rise above normal water level where stormwater is temporarily stored for treatment for a certain detention period (usually 48 – 72 hours in a wetland system)
WSUD - Water Sensitive Urban Design	Term used to describe the design of drainage systems used to o Convey stormwater safely o Retain stormwater pollutants o Enhance local ecology o Enhance the local landscape and social amenity of built areas
Wetland	WSUD elements which are used to collect TSS, TP and TN. Usually incorporated at normal water level (NWL) below which the system is designed as shallow marsh, marsh, deep marsh and open water areas.

Addendum A – Hancocks Gully North Wetland/Retarding Basin (W1) Design

The following section details the design and calculations for Hancocks Gully North Wetland/Retarding Basin (W1) functional design.

Addendum A Contents:

A. 1 Functional Design Drawings 38
A. 2 Functional Design Calculations 45
A. 3 Sediment Pond Design 49
A. 4 Wetland Velocity Checks 53
A. 5 North Wetland MUSIC results 56
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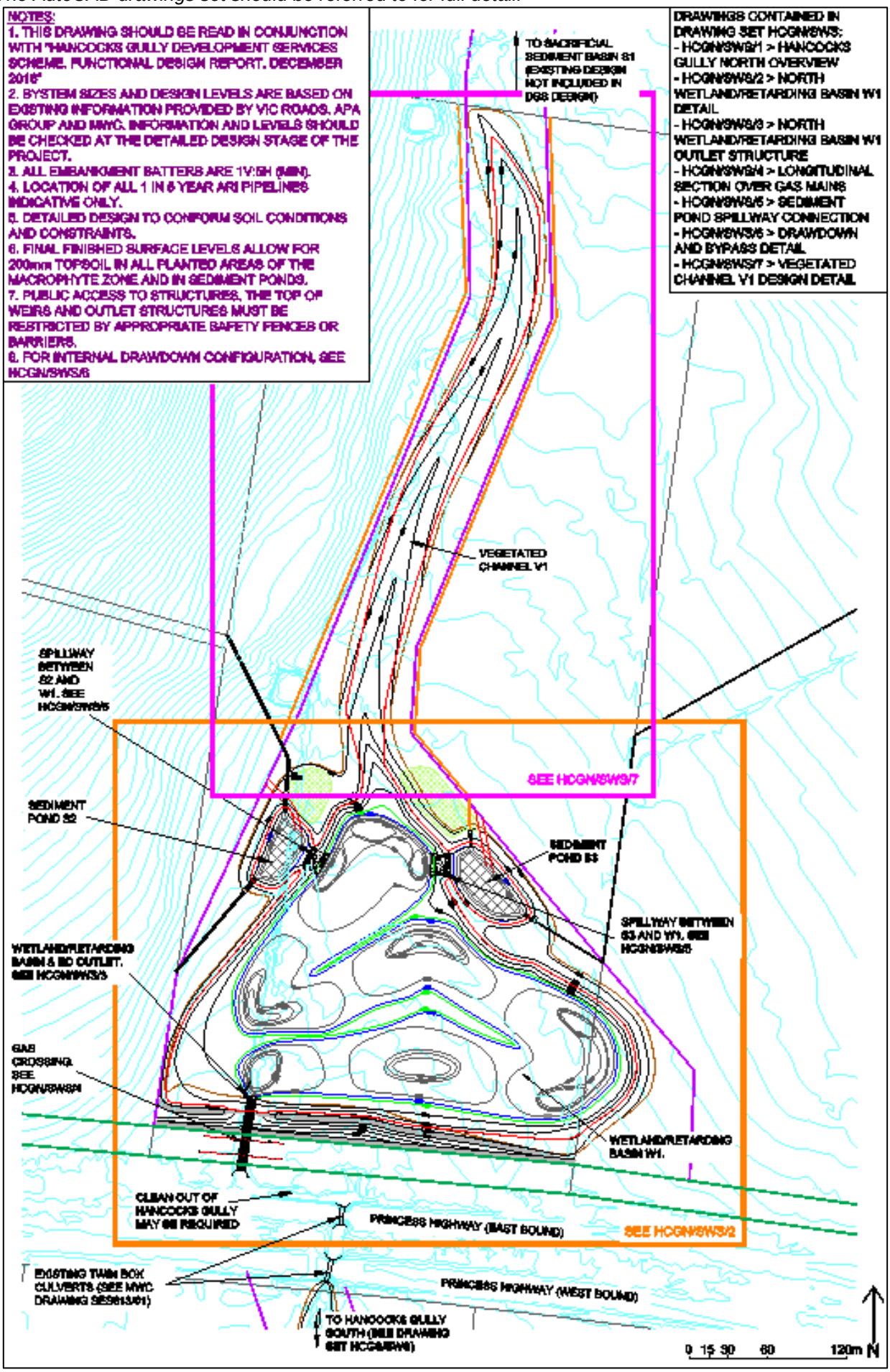
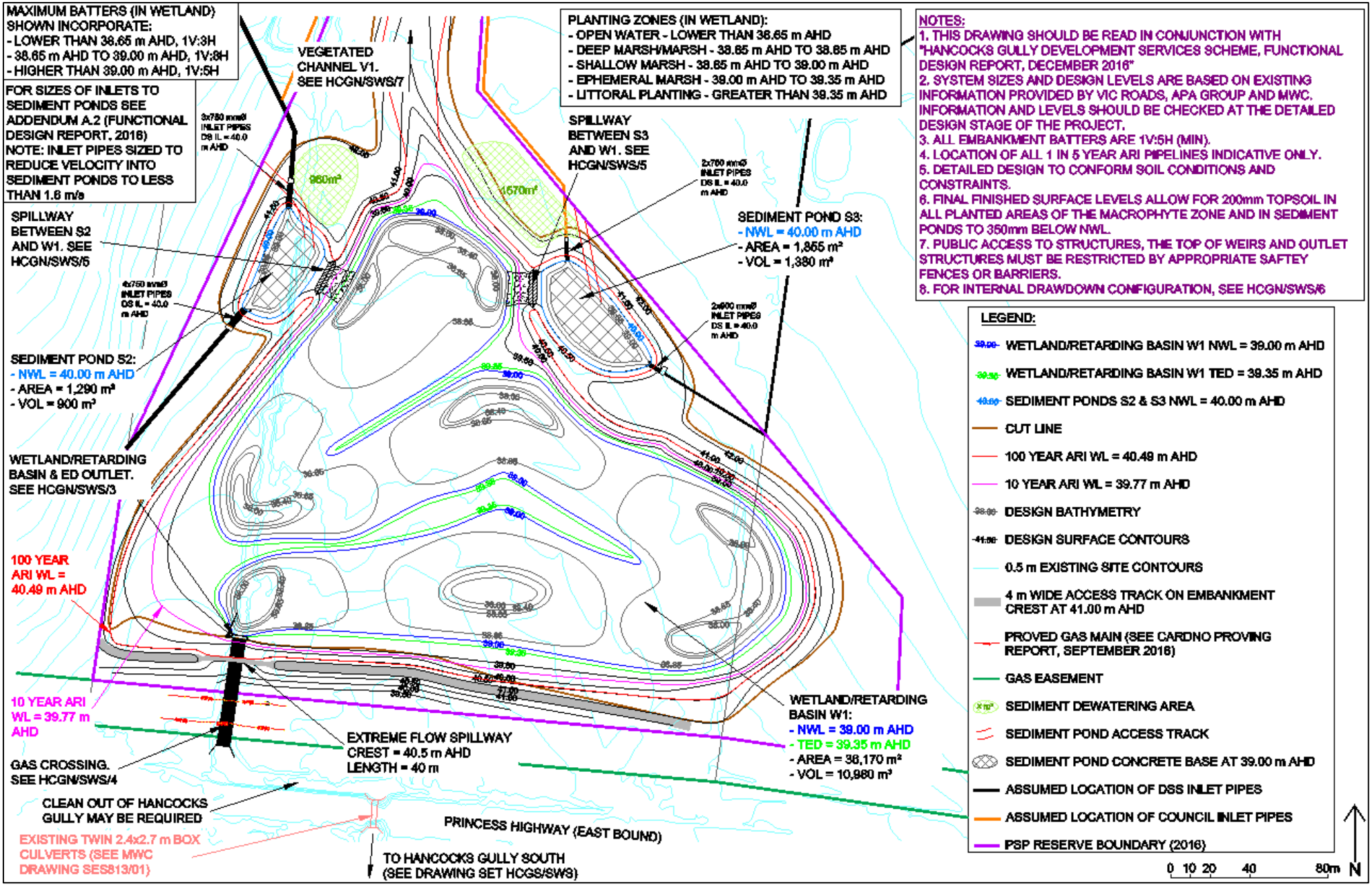
A. 7 Sediment Pond Isolation Pipe Design 60
A. 8 Liner and Topsoil 64
A. 9 Constructed Wetlands Design Manual, Part A.2: Checklist 65

A. 1 Functional Design Drawings

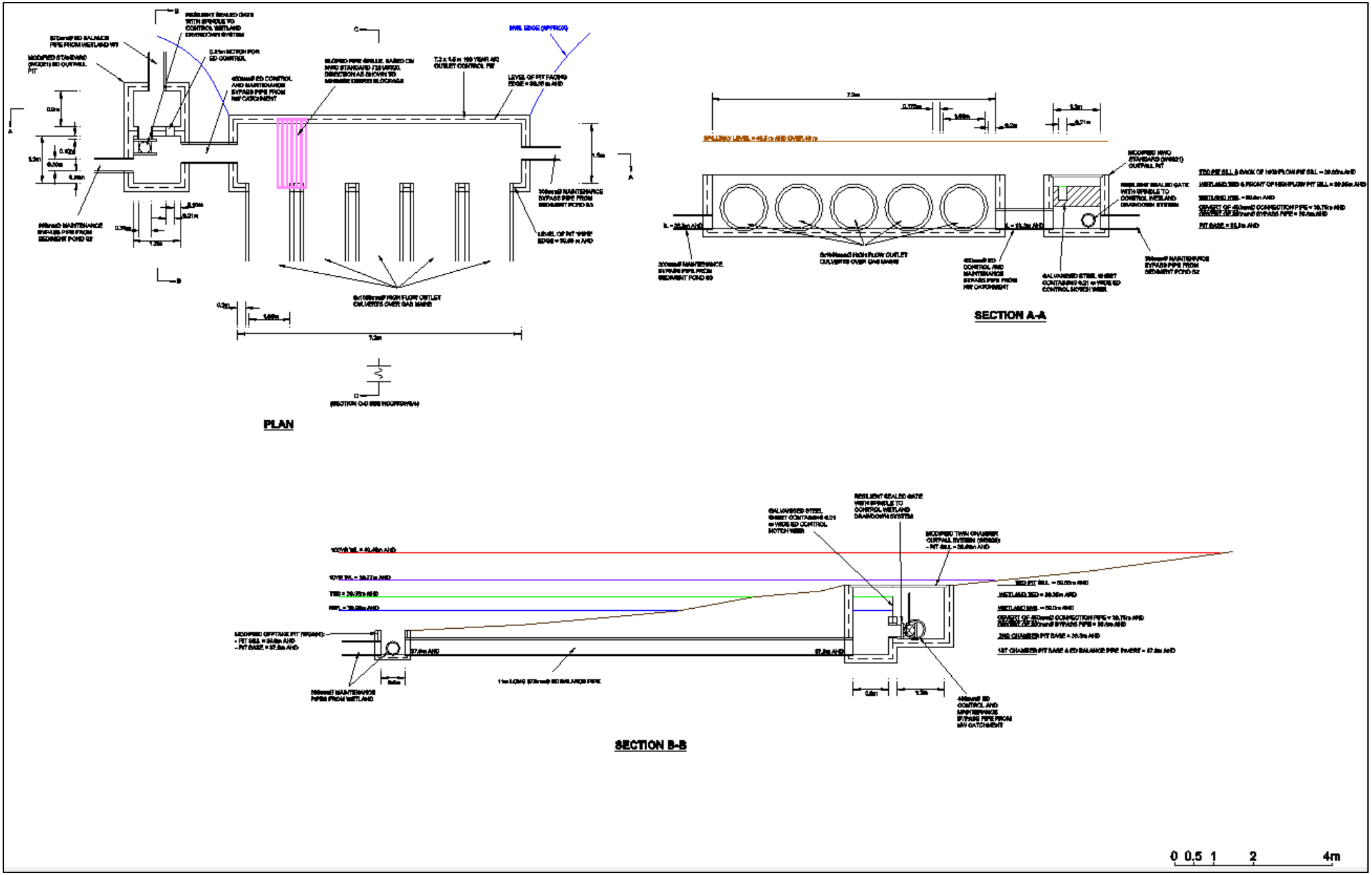
Note: The AutoCAD drawings set should be referred to for full detail.



HCGN/SWS/ 1 Hancocks Gully North Overview



HCGN/SWS/ 2 North Wetland Retarding Basin W1 Detail

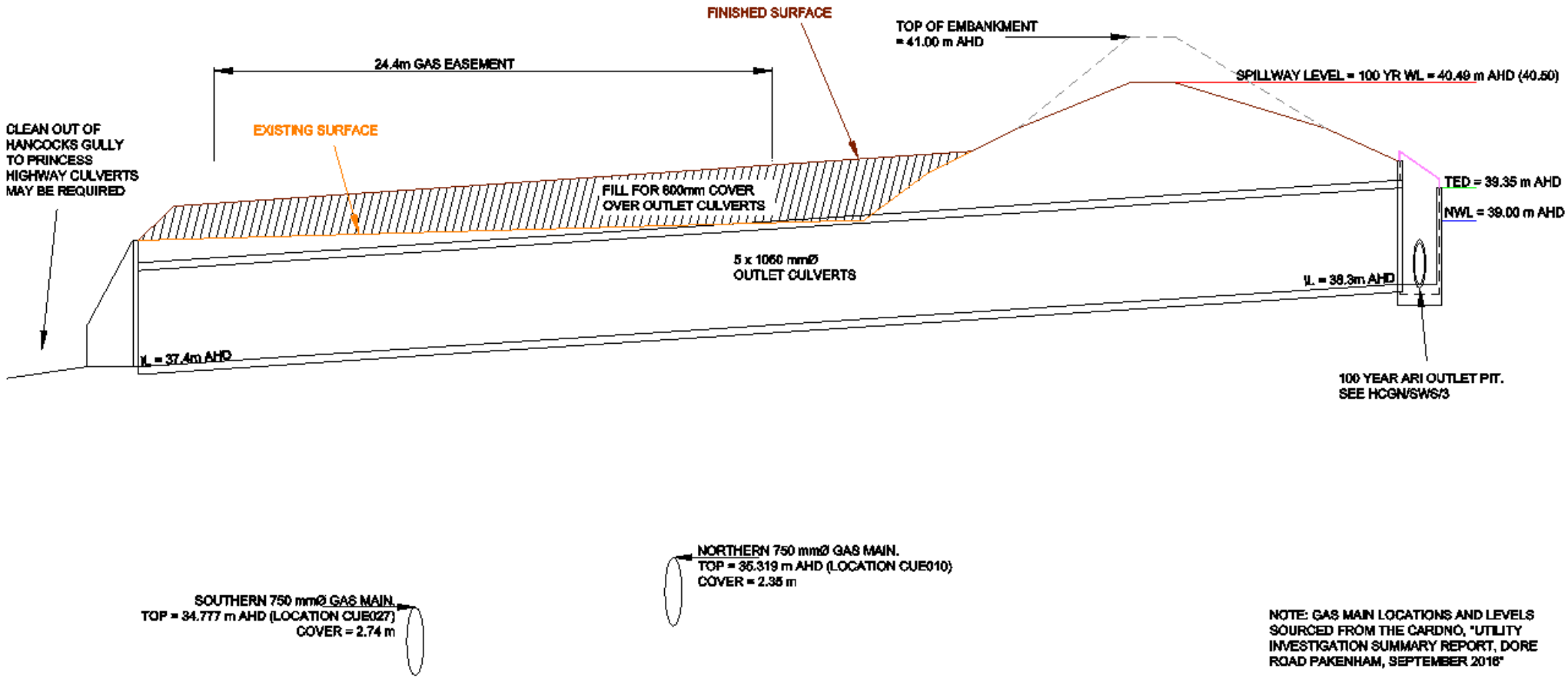


HCGN/SWS/ 3 North Wetland Retarding Basin W1 Outlet Structure

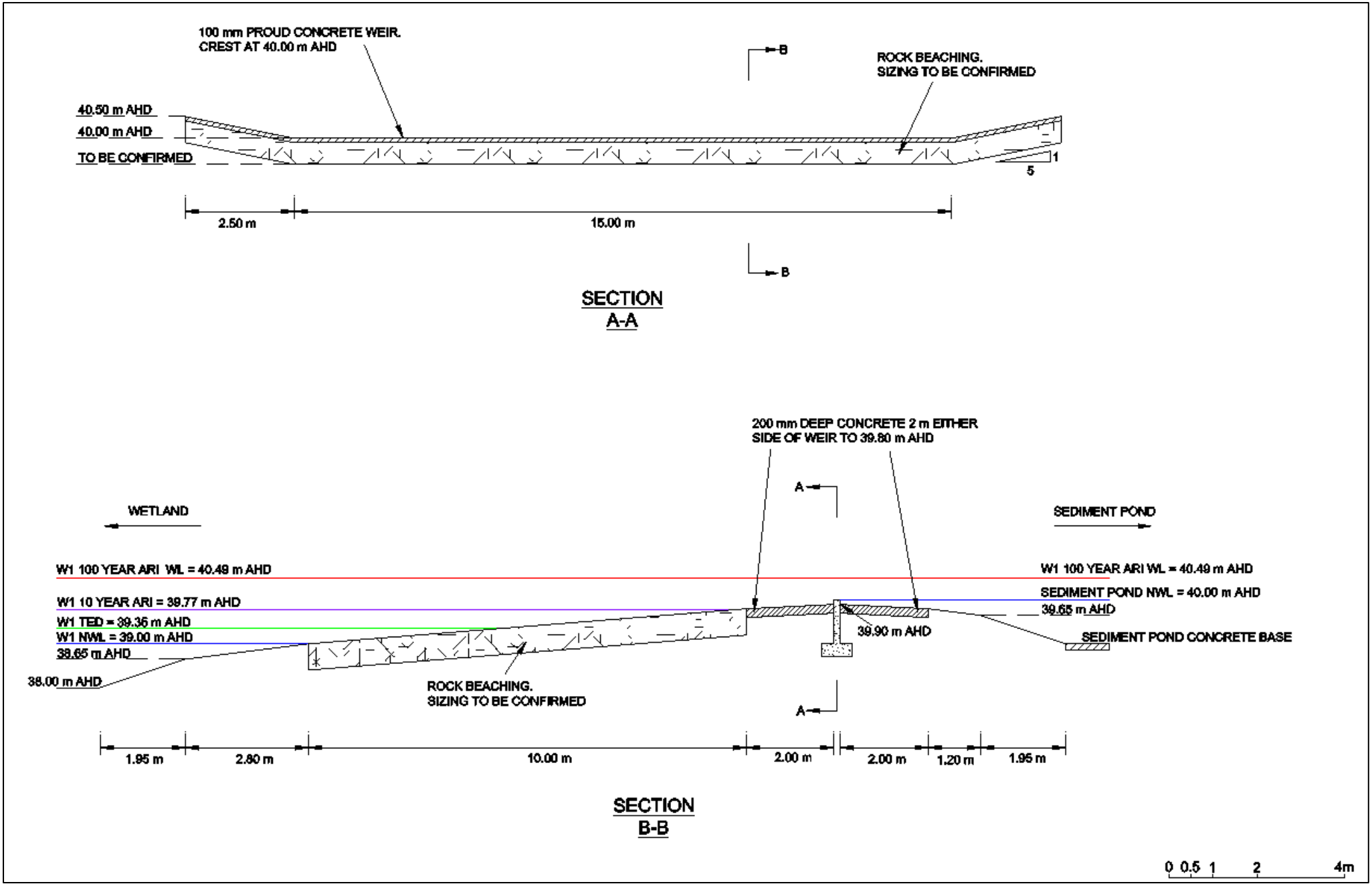
THE TABLES BELOW PROVIDE A SUMMARY OF THE CARDNO SURVEY.
- CUE007 TO CUE010 BOUND THE NORTHERN LINE
- CUE030 TO CUE027 BOUND THE SOUTHERN LINE

Northern Line						
Location	CUE030	CUE029	CUE028	CUE027	CUE026	CUE011
RL (m AHD)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Depth (m)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Top of Gas Main (m AHD)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

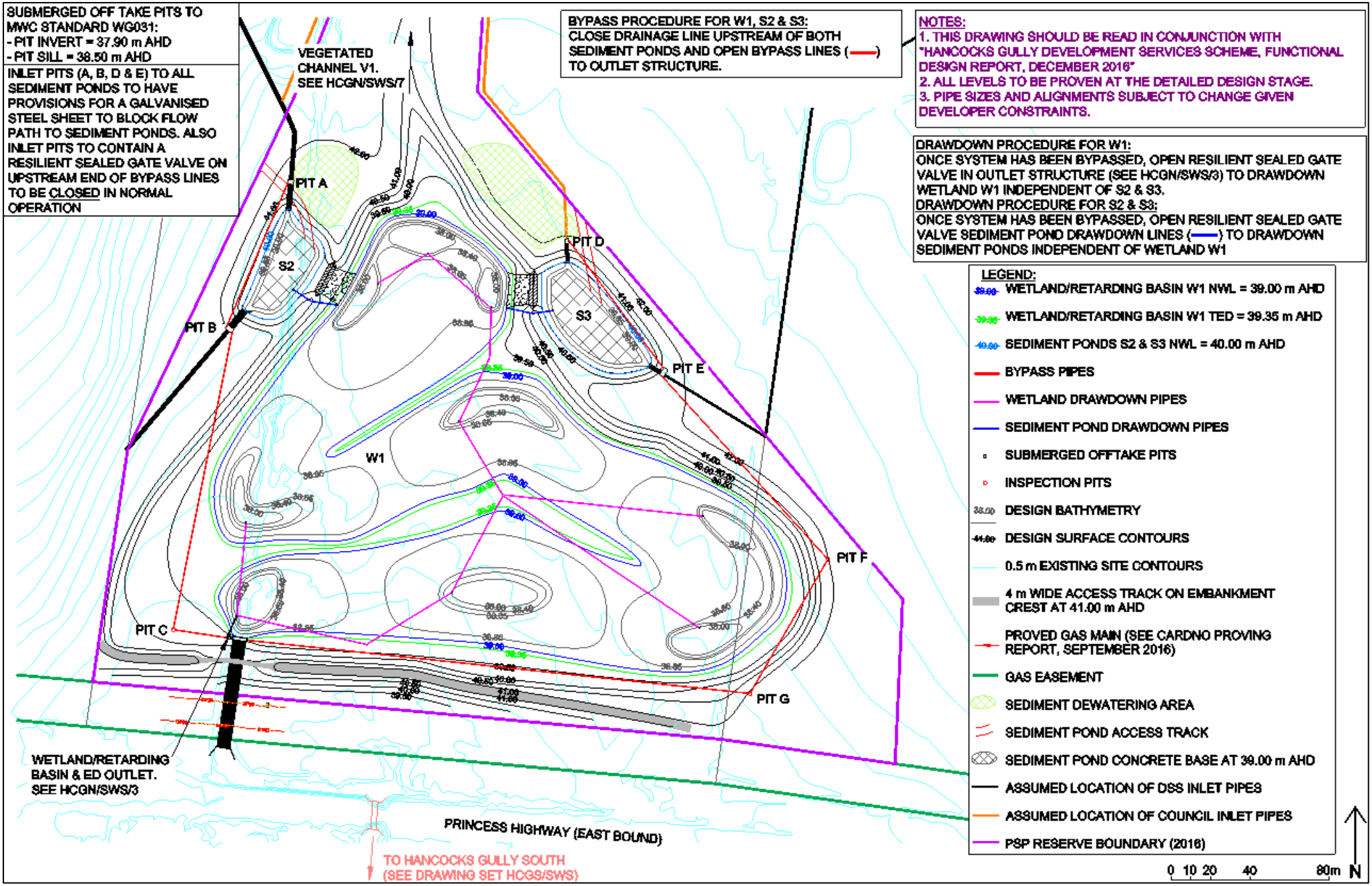
Southern Line						
Location	CUE008	CUE009	CUE010	CUE011	CUE012	CUE013
RL (m AHD)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Depth (m)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Top of Gas Main (m AHD)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]



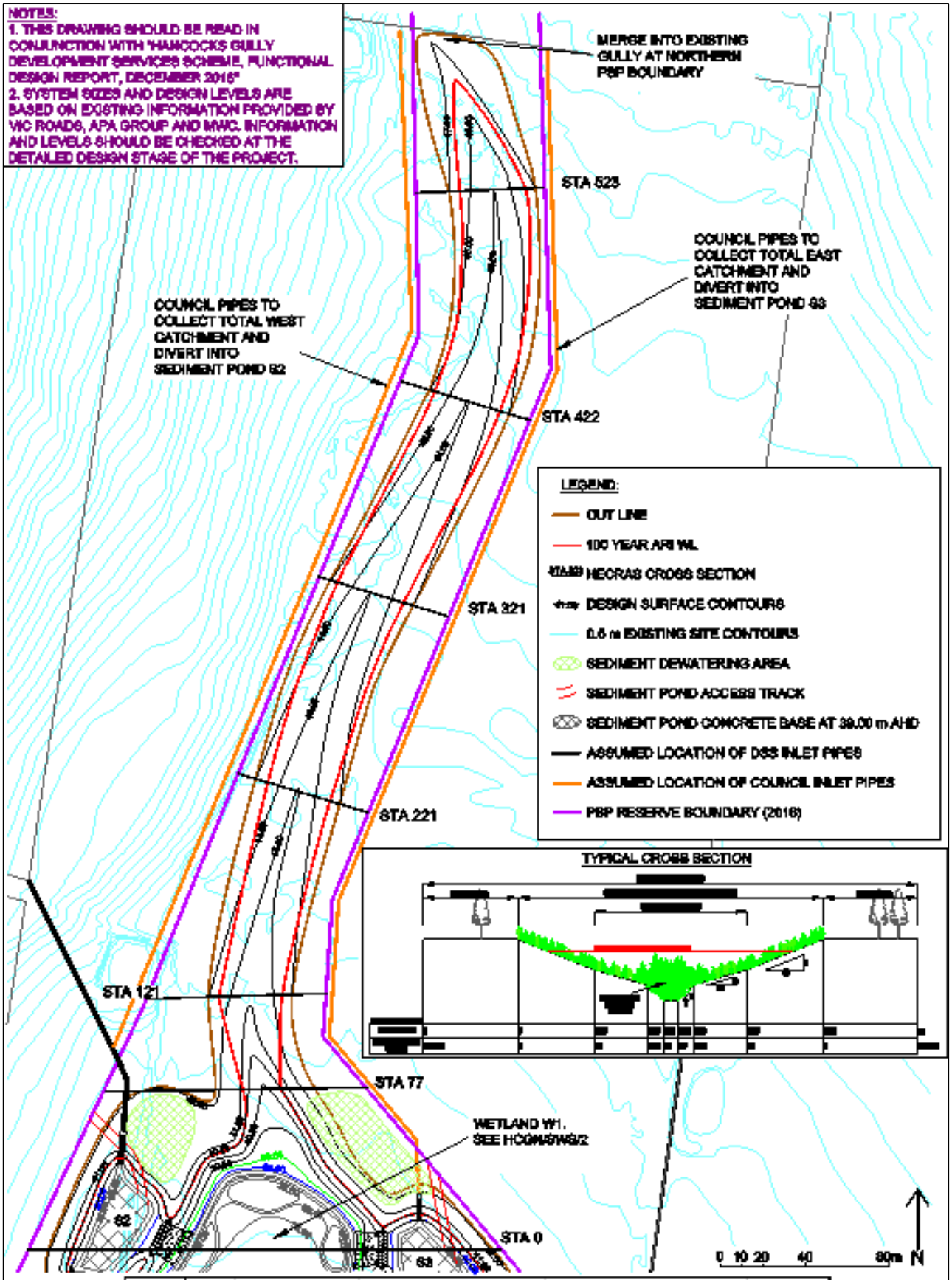
HCGN/SWS/ 4 Longitudional Section over Gas Mains



HCGN/SWS/ 5 Sediment Pond Spillway Connection



HCGN/SWS/ 6 Drawdown and Bypass Detail



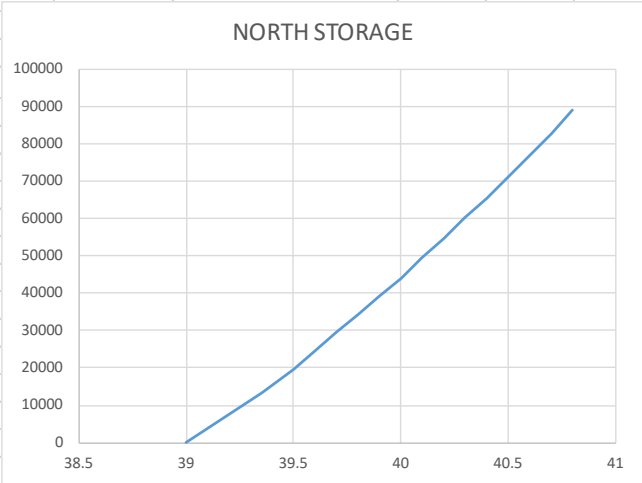
River Sta	Q Total (m^3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
523	15	45.0	45.61	45.05	45.63	0.010	0.67	22.31	32.13	0.31
422	15	44.0	45.61		45.63	0.010	0.67	22.31	32.13	0.31
321	15	43.0	44.60		44.61	0.010	0.59	24.77	32.08	0.21
221	15	42.0	43.60		43.62	0.010	0.59	24.15	32.04	0.21
121	15	41.0	42.46		42.49	0.017	0.91	29.95	29.22	0.27
77	15	40.5	41.48		41.540	0.051	1.11	13.47	20.94	0.44
0	15	39.0	40.50	39.15	40.500	0.000	0.10	150.08	173.54	0.03

A. 2 Functional Design Calculations

Retarding Basin Stage Storage Relationship

Level (m)	Area (m^2)	Average Area (m^2)	delta h (m)	Volume (m^3)	Cumulative Volume (m^3)
39	36169				0
39.35	39559	37864	0.35	13252.4	13252.4
39.5	46141	42850	0.15	6427.5	19679.9
40	54978	50559.5	0.5	25279.75	44959.65
40.5	59446	57212	0.5	28606	73565.65
41	64114	61780	0.5	30890	104455.65

Level (m)	Cumulative Volume (m^3)	
39	0	
39.35	13252	
39.5	19680	
39.6	24736	Linear Approx
39.7	29792	Linear Approx
39.8	34848	Linear Approx
39.9	39904	Linear Approx
40	44960	
40.1	50681	Linear Approx
40.2	56402	Linear Approx
40.3	62123	Linear Approx
40.4	67844	Linear Approx
40.5	73566	
40.6	79744	Linear Approx
40.8	92100	Linear Approx
41	104456	



Wetland rectangular notch ED Control

Q = B × C × Le × h1.5

where

Q = flow rate (m3/s)

h = head on the weir (m)

B = blockage factor (assume no blockage as in manual)

C = weir coefficient = 1.7 sharp crested weir

L = Actual Weir Length = 0.21 m

Area at NWL = 36169 m^2

Area at TED = 39559 m^2 (conservatively same as TED)

Volume of water stored for treatment over Ed range 0.35 m

.= 13252.4 m^3

L e = effective length = L - 0.2h, where L = Actual Weir Length

WL (m AHD)	h (m)	Le (m)	Q (m3/s)	ED Volum	ED Detention Time (hrs)
39	0	0.21	0	0	
39.35	0.35	0.14	0.05	13252	75

Retarding Basin Outflow Relationship

Number of Culverts =	5	
Diameter =	1.050	m
Length =	55	m
Upstream IL =	38.3	m AHD
Downstream IL =	37.4	m AHD
Downstream Obvert =	38.45	m AHD
Long. Slope (1/)=	61.11	
Mannings n =	0.013	
Upstream Obvert =	39.350	m AHD

Water Level (m AHD)	$Q_{Culvert, Outlet Control}$ (m^3/s)	$Q_{Culvert, Inlet Control}$ (m^3/s)	$Q_{Pit as Orifice}$ (m^3/s)	$Q_{Pit as Wier}$ (m^3/s)	Q_{TED} (m^3/s)	$Q_{spillway}$ (m^3/s)	Q_{out} (m^3/s)	Controlling
39.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	NWL
39.35	0.00	0.00	0.00	0.00	0.052	0.000	0.05	TED Range
39.50	7.54	9.45	5.70	1.48	0.052	0.00	1.53	
39.60	8.44	10.13	7.36	3.17	0.052	0.00	3.23	
39.70	9.25	10.76	8.70	5.26	0.052	0.00	5.31	Pit as Weir
39.80	9.99	11.36	9.87	7.67	0.052	0.00	7.72	
39.90	10.67	11.93	10.91	10.36	0.052	0.00	10.41	
40.00	11.32	12.47	11.86	13.31	0.052	0.00	11.37	
40.10	11.93	12.99	12.74	16.50	0.052	0.00	11.98	
40.20	12.52	13.49	13.56	19.90	0.052	0.00	12.57	Culvert
40.30	13.07	13.97	14.34	23.52	0.052	0.00	13.12	Control
40.40	13.61	14.44	15.07	27.33	0.052	0.00	13.66	
40.50	14.12	14.89	15.77	31.32	0.052	0.00	14.17	
40.60	14.62	15.33	16.45	35.49	0.052	0.89	15.56	
40.80	15.56	15.76	17.09	39.84	0.052	9.99	25.59	Spillway
41.00	16.45	16.57	18.31	49.01	0.052	24.10	40.59	

Reatrding Basin Stage/Storage/Discharge Relationship

Level (m)	Storage (m^3)	Flow (m^3/s)	
39.00	0	0.00	NWL
39.35	13252	0.05	TED Range
39.50	19680	1.53	
39.60	24736	3.23	
39.70	29792	5.31	Pit as Weir
39.80	34848	7.72	
39.90	39904	10.41	
40.00	44960	11.37	
40.10	50681	11.98	
40.20	56402	12.57	Culvert
40.30	62123	13.12	Control
40.40	67844	13.66	
40.50	73566	14.17	
40.60	79744	15.56	
40.80	92100	25.59	Spillway
41.00	104456	40.59	

Balance/Connection pipes within Outlet Structure:

BALANCE PIPE FOR ED OUTLET			BALANCE PIPE FROM ED OUTLET TO HIGH FLOW CULVERTS		
head loss = $(Ke+Kex)\times V^2/2g+S_f\times L$			head loss = $(Ke+Kex)\times V^2/2g+S_f\times L$		
$S_f = Q^2n^2/A^2R^{4/3}$			$S_f = Q^2n^2/A^2R^{4/3}$		
pipe dia =	0.375	m	pipe dia =	0.450	m
RCP pipe radius =	0.1875	m	RCP pipe radius =	0.225	m
Design flow =	0.05	m^3/s	Design flow =	0.10	m^3/s
Wetted perimeter =	1.18	m	Wetted perimeter =	1.41	m
Area =	0.11	m^2	Area =	0.16	m^2
Hyd radius =	0.09375	m	Hyd radius =	0.1125	m
V =	0.45	m/s	V =	0.66	m/s
Ke =	0.5		Ke =	0.5	
Kex =	1		Kex =	1	
n =	0.013		n =	0.013	
L=	14		L=	1.45	
S_f =	0.0008		S_f =	0.0013	
Head loss =	0.026	m	Head loss =	0.035	m

Spillways Connecting Sediment Ponds to W1:

NW SED SPILLWAY CONTROL					NE SED SPILLWAY CONTROL				
Q_{100} =	7.6	m^3/s			Q_{100} =	9.8	m^3/s		
$Q = B\times C\times Le\times h^{1.5}$					$Q = B\times C\times Le\times h^{1.5}$				
where					where				
Q = flow rate (m3/s)					Q = flow rate (m3/s)				
h = head on the weir (m)					h = head on the weir (m)				
B = blockage factor (assume no blockage as in manual)					B = blockage factor (assume no blockage as in manual)				
C = weir coefficient =	2	BROAD CREST SLOPE APPROACH			C = weir coefficient =	2	BROAD CREST SLOPE APPROACH		
L = Actual Weir Length =	15	m			L = Actual Weir Length =	15	m		
L e = effective length = L - 0.2h, where L = Actual Weir Length					L e = effective length = L - 0.2h, where L = Actual Weir Length				
WL (m AHD)	h (m)	Le (m)	Q (m^3/s)		WL (m AHD)	h (m)	Le (m)	Q (m^3/s)	
40	0	15	0		40	0	15	0	
40.5	0.5	14.9	10.5		40.5	0.5	14.9	10.5	
Therefore 15m long spillway & embankment at 40.5 provides					Therefore 15m long spillway & embankment at 40.5 provides				
100yr flow conveyance					100yr flow conveyance				

Sediment Ponds S2 & S3 Inlet Pipe Sizing:

Location	NW Sed (S2) (North)	NW Sed (S2) (South)	NE Sed (S3) (North)	NE Sed (S3) (South)
Q_5 (m^3/s, RORB)	1.9	2.6	1.3	2.0
Pipe Size (m)	0.75	0.75	0.75	0.9
HGL slope (1/x)	300	300	300	300
Number of pipes	3	4	2	2
Pipe Capacity (m^3/s)	1.9	2.6	1.3	2.1
Pipe Velocty (m/s)	1.5	1.5	1.5	1.6

Note: RORB model from Addendum D. 2 was altered to provide flows at each inlet location (North and South) for each Sediment Pond.

Princess Highway Culvert Capacity:

Capacity of Handcocks gully at Princes Highway						
Based on vic Roads Drawing 42075						
Assume all culverts flowing under outlet control						
and head loss = 0.3 to ensure carriageway not engaged						
head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$						
$S_f = Q^2n^2/A^2R^{4/3}$						
Twin 2400 by 2800 Box Culvert System						
W =		2.4	m			
D =		2.4	m			
Capacity flow/cell =		10	m^3/s			
Wetted perimeter =		9.60	m			
Area =		5.76	m^2			
Hyd radius =		0.6	m			
V =		1.74	m/s			
Ke =		0.5				
Kex =		1				
n =		0.013				
L=		43.3				
S_f =		0.0010				
Head loss =		0.27	m			
Total system Design Flow =			20	m^3/s		

A. 3 Sediment Pond Design

Calculation of North West (S2) Sediment Pond Size

NWL =		40.00	m AHD					
TED =		40.00	m AHD					
Safety batters at 1 in		8	to 350 below NW					
Base level =		39.00	m AHD					
Area at NWL =		1291	m^2					
Total Volume =		898	m^3					
Sump Volume (volume below350 mm depth) =					513	m^3		
Batters - General								
Batters above NWL =			1 in 6					
Batters NWL to 350 below NWL=				1 in 8				
Batters 350 below NWL to base=				1 in 3				
Batters - access track								
Batters above NWL =			1 in 15					
Batters NWL to 500 below NWL=				1 in 8				
Batters 500 below NWL to base=				1 in 5				
Concrete base								
Sediment Removal								
Fair and Geyer Equation								
$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$			$\lambda = 1 - 1/n;$	$n = \frac{1}{1-\lambda}$				
Vs =	0.011	m/s	*Target = very fine sand*					
d_e =	0	m						
d_p =	1	m						
d* =	1	m						
(d_e+d_p) =	1							
(d_e+d^*)								
Q_{3mth} =	0.58	m^3/s	(RULE OF	Q_5 =	2.9	m^3/s (RORB)		
A =	1291.00	m^2						
V_s =	24.48							
Q/A								
λ =	0.11	pond shape assumption						
n =	1.12							
Fraction of Initial Solids Removed								
R =	97%							
Requirement: Melbourne Water require R = 95% for a 125 micrometer particle								
				for 3 month event				

Cleanout Frequency

Catchment Area =	49.15	ha	Areas N-AF			
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)			
Sump Volume =	513	m^3	area between base and 0.35 m below NWL			
Therefore, cleanout frequency required =		R(1.6+0.4)$A_{catchment}$ =		0.2	per year	
(sediment to 500 below NWL)		sump volume				
			.= every	5	years	OK

Assumes cleanout when sump volume of basin is full (ie sediment 350 mm below NWL)

Sediment Dewatering Area

Dewatering depth =	0.5	m			
Sediment volume collected every 5 years=				477	m^3
Required Dewatering area =	954	m^2			

Ensure this area is provided near the sediment pond and is accessible with machinery/access tracks etc.

Dewatering Provision for Maintenance

Bypass S2 as described in drawing HCGN/SWS/6

Once S2 is isolated open resilient sealed gate valve in sediment pond drawdown pipe to draw S2 down under gravity

Sediment Pond Flood Flow Velocity Check

See Addendum A.4

Calculation of North East (S3) Sediment Pond Size

NWL =		40.00	m AHD				
TED =		40.00	m AHD				
Safety batters at 1 in		8	to 350 below NW				
Base level =		39.00	m AHD				
Area at NWL =		1855	m^2				
Total Volume =		1384	m^3				
Sump Volume (volume below350 mm depth) =					814	m^3	
Batters - General							
Batters above NWL =			1 in 6				
Batters NWL to 350 below NWL=				1 in 8			
Batters 350 below NWL to base=				1 in 3			
Batters - access track							
Batters above NWL =			1 in 15				
Batters NWL to 500 below NWL=				1 in 8			
Batters 500 below NWL to base=				1 in 5			
Concrete base							
Sediment Removal							
Fair and Geyer Equation							
$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$			$\lambda = 1 - 1/n;$ $n = \frac{1}{1-\lambda}$				
Vs =	0.011	m/s	*Target = very fine sand*				
d_e =	0	m					
d_p =	1	m					
d* =	1	m					
(d_e+d_p) =	1						
(d_e+d^*)							
Q_{3mth} =	0.68	m^3/s	(RULE OF	Q_5 =	3.4	m^3/s (RORB)	
A =	1855	m^2					
V_s =	24.48						
Q/A							
λ =	0.11	pond shape assumption					
n =	1.12						
Fraction of Initial Solids Removed							
R =	98%						
Requirement: Melbourne Water require R = 95% for a 125 micrometer particle							
				for 3 month event			

Cleanout Frequency

Catchment Area =	79.74	ha	Areas AG-AO			
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)			
Sump Volume =	814	m^3	area between base and 0.35 m below NWL			
Therefore, cleanout frequency required =	$R(1.6+0.4)A_{catchment}$ =			0.2	per year	
(sediment to 500 below NWL)	sump volume					
			.= every	5	years	OK

Assumes cleanout when sump volume of basin is full (ie sediment 350 mm below NWL)

Sediment Dewatering Area

Dewatering depth =	0.5	m			
Sediment volume collected every 5 years=				778	m^3
Required Dewatering area =	1557	m^2			

Ensure this area is provided near the sediment pond as shown in drawing HCGN/SWS/2

Dewatering Provision for Maintenance

Bypass S3 as described in drawing HCGN/SWS/6

Once S3 is isolated open resilient sealed gate valve in sediment pond drawdown pipe to draw S3 down under gravity

Sediment Pond Flood Flow Velocity Check

See Addendum A.4

A. 4 Wetland Velocity Checks

Initial Wetland Flow Velocity Checks - North West (S2) Sediment Pond								
Constructed Wetlands Design Manual , Part D: Design tools, resources and glossary (2015)								
Hydraulic analysis of flow velocities, Manual calculation								
Initial Velocity Checks								
Q_{100} =	7.6	m^3/s (RORB)						
Q_{10} =	3.7	m^3/s (RORB)						
Q_5 =	2.9	m^3/s (RORB)						
Q_{3mth} =	0.6	m^3/s (Rule of thumb given Q_5)						
Wetland Normal Water Level (NWL) =				40.00	m AHD			
Wetland Top of Extended Detention (TED) =				40.00	m AHD			
Maximum Base level at wetland narrowest width =				39.00	m AHD			
1a	Peak 10 yr flow through sediment pond =					3.7	m^3/s (RORB)	
	Peak 100 yr flow through sediment pond =					7.6	m^3/s (RORB)	
1b	Bypass around Macrophyte zone =					0.0	m^3/s	
	Macrophyte zone inlet capacity =					0.0	m^3/s	
	Peak 3 month flow through macrophyte zone =					0.6	m^3/s	
	Peak 10 yr flow through macrophyte zone =					3.7	m^3/s	
	Peak 100 yr flow through macrophyte zone =					7.6	m^3/s	
Initial Sediment Pond Velocity Check								
2	RORB 100 yr WL =		40.5	m AHD	Assumes when 100 yr flow			
					comes in WL is at 100 yr level			
	At narrowest part of the sediment pond:							
3a	NWL width =		27	m				
3b	Width at 10 yr WL =		33	m				
4	10 yr WL - NWL =		0.50	m				
	Average width =		30	m				
	Cross section flow area =		15	m^2				
5	100 yr Flow velocity =		0.5	m/s < 0.5 m/s		OK		

Initial Wetland Flow Velocity Checks - North East (S3) Sediment Pond

Constructed Wetlands Design Manual , Part D: Design tools, resources and glossary (2015)

Hydraulic analysis of flow velocities, Manual calculation

Initial Velocity Checks

Q_{100} =	9.8	m^3/s (RORB)
Q_{10} =	4.4	m^3/s (RORB)
Q_5 =	3.4	m^3/s (RORB)
Q_{3mth} =	0.7	m^3/s (Rule of thumb given Q_5)

Wetland Normal Water Level (NWL) =	40.00	m AHD
Wetland Top of Extended Detention (TED) =	40.00	m AHD
Maximum Base level at wetland narrowest width =	39.00	m AHD

1a	Peak 10 yr flow through sediment pond =	4.4	m^3/s (RORB)
	Peak 100 yr flow through sediment pond =	9.8	m^3/s (RORB)
1b	Bypass around Macrophyte zone =	0.0	m^3/s
	Macrophyte zone inlet capacity =	0.0	m^3/s
	Peak 3 month flow through macrophyte zone =	0.7	m^3/s
	Peak 10 yr flow through macrophyte zone =	4.4	m^3/s
	Peak 100 yr flow through macrophyte zone =	9.8	m^3/s

Initial Sediment Pond Velocity Check

2	RORB 100 yr WL =	40.5	m AHD	Assumes when 100 yr flow comes in WL is at 100 yr level
	At narrowest part of the sediment pond:			
3a	NWL width =	33	m	
3b	Width at 10 yr WL =	39	m	
4	10 yr WL - NWL =	0.50	m	
	Average width =	36	m	
	Cross section flow area =	18	m^2	
5	100 yr Flow velocity =	0.5	m/s < 0.5 m/s	OK

Initial Wetland Flow Velocity Checks - North Wetland

Constructed Wetlands Design Manual , Part D: Design tools, resources and glossary (2015)

Hydraulic analysis of flow velocities, Manual calculation

Initial Velocity Checks

Q_{100} =	17.9	m^3/s (RORB)
Q_{10} =	8.5	m^3/s (RORB)
Q_5 =	6.6	m^3/s (RORB)
Q_{3mth} =	1.3	m^3/s (Rule of thumb given Q_5)

Wetland Normal Water Level (NWL) =	39.00	m AHD
Wetland Top of Extended Detention (TED) =	39.35	m AHD
Maximum Base level at wetland narrowest width =	38.65	m AHD

1a	Peak 10 yr flow through sediment pond =	8.5	m^3/s (RORB)
	Peak 100 yr flow through sediment pond =	17.9	m^3/s (RORB)
1b	Bypass around Macrophyte zone =	0.0	m^3/s
	Macrophyte zone inlet capacity =	0.0	m^3/s
	Peak 3 month flow through macrophyte zone =	1.3	m^3/s
	Peak 10 yr flow through macrophyte zone =	8.5	m^3/s (accounts for bypass)
	Peak 100 yr flow through macrophyte zone =	17.9	m^3/s (accounts for bypass)

Initial Macrophyte zone Velocity Check

6	RORB 10 yr WL =	39.8	m AHD
	At narrowest part of the macrophyte zone:		
7a	NWL width =	39	m
7b	TED width =	43.2	m (very conservative - curls back on itself)
7c	Width at 10 yr WL =	48.6	m (very conservative - curls back on itself)
8a	TED - base level at narrowest width =	0.7	m
	Average width =	41.1	m
	Cross section flow area =	28.77	m^2
8b	10 yr WL - NWL =	0.8	m
	Average width =	45.9	m
	Cross section flow area =	36.72	m^2
9	3 month Flow velocity =	0.05	m/s OK

A. 5 North Wetland MUSIC results

Results From File:	Hancocks_MUSIC_NV5_SV3_15Aug16.sqz			
Treatment Train Effectiveness at North Outlet:				
Flow (ML/yr)	2,160	2,110	2.2	
Total Suspended Solids (kg/yr)	191,000	57,600	69.9	
Total Phosphorus (kg/yr)	550	267	51.5	
Total Nitrogen (kg/yr)	4,360	3,150	27.7	
Gross Pollutants (kg/yr)	25,700	-	100	
Polutant Generated from North Outside Catchments:				
Flow (ML/yr)	1,480	1,480	0	
Total Suspended Solids (kg/yr)	85,200	85,200	0	
Total Phosphorus (kg/yr)	312	312	0	
Total Nitrogen (kg/yr)	2,520	2,520	0	
Gross Pollutants (kg/yr)	4,110	4,110	0	
Polutant Generated from North Inside Catchments:				
Flow (ML/yr)	680			
Total Suspended Solids (kg/yr)	105,800			
Total Phosphorus (kg/yr)	238			
Total Nitrogen (kg/yr)	1,840			
Gross Pollutants (kg/yr)	21,590			
Polutant Treated inside North Treament Train:				
Flow (ML/yr)	50			
Total Suspended Solids (kg/yr)	133,400			
Total Phosphorus (kg/yr)	283			
Total Nitrogen (kg/yr)	1,210			
Gross Pollutants (kg/yr)	25,700			
Local Treament Train Effectiveness:				
Flow (ML/yr)	7%			
Total Suspended Solids (kg/yr)	100%			
Total Phosphorus (kg/yr)	100%			
Total Nitrogen (kg/yr)	66%			
Gross Pollutants (kg/yr)	100%			

A. 6 Wetland Inundation Check

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 if this type. The check required to be undertaken is:

Check 1: Regular inundation check

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

Critical plant height is defined as the plant height relative to NWL (m AHD). It is assumed that the shortest allowable average plant heights are 1.5 meters in shallow marsh and 1.5 meters in deep marsh zones.

The analysis assumes the MUSIC model detailed in Section 6 with Narre Warren North rainfall data at 6 minute increments (1984 – 1993).

The MUSIC model incorporates internally calculated outflow relationships pertaining to:

- 72 hours' detention time in the ED zone, and
- Detention times calculated assuming weir widths as per the outlet pit as shown in drawing HCGN/SWS/ 3 for water levels above the ED zone.

The analysis was completed using the MUSIC Auditor – Wetland Analysis Tool as shown below. This analysis results in:

- Sea Club-rush being excluded from the specified plant list in shallow marsh zones,
- Common Spike-rush being excluded from the specified plant list in shallow marsh zones, and
- Water Ribbons being excluded from the specified plant list in both the shallow or deep marsh zones.

Wetland Analysis Tool

Welcome to the Wetland Analysis Tool for checking compliance with the Melbourne Water Constructed Wetland Manual. The tool creates an inundation frequency curve and helps the user to assess wetland depths relative to plant heights and the wetland residence time.

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

Shallow Planting Depth 0.15 m
Deep Planting Depth 0.35 m

Please enter the permanent pool volume.

Permanent Pool Volume 10978 m^3

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

Choose file NV5.csv
FILE IS UPLOADED

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

Clear Selection

Name	Average plant height (m)	Shallow marsh plants	Deep marsh plants	Suitability
Jointed Club-rush *Baumea articulata*	1.8	☑	☑	Shallow and Deep
Tall Club-rush *Bolboschoenus fluviatilis*	1.8	☑	☑	
Marsh Club-rush *Bolboschoenus medianus*	1.5	☑	☑	
Leafy Twig-rush *Cladium procerum*	2	☑	☑	
River Club-rush *Schoenoplectus tabernaemontani*	1.8	☑	☑	
Tall Spike-rush *Eleocharis sphacelata*	1.5		☑	Deep Only
Common reed *Phragmites australis*	2.5		☑	
Sea Club-rush *Bolboschoenus caldwellii*	1	☐		Unsuitable
Common Spike-rush *Eleocharis acuta*	0.5	☐		
Water Ribbons *Triglochin procerum*	1	☐	☐	

+ Add user defined plant

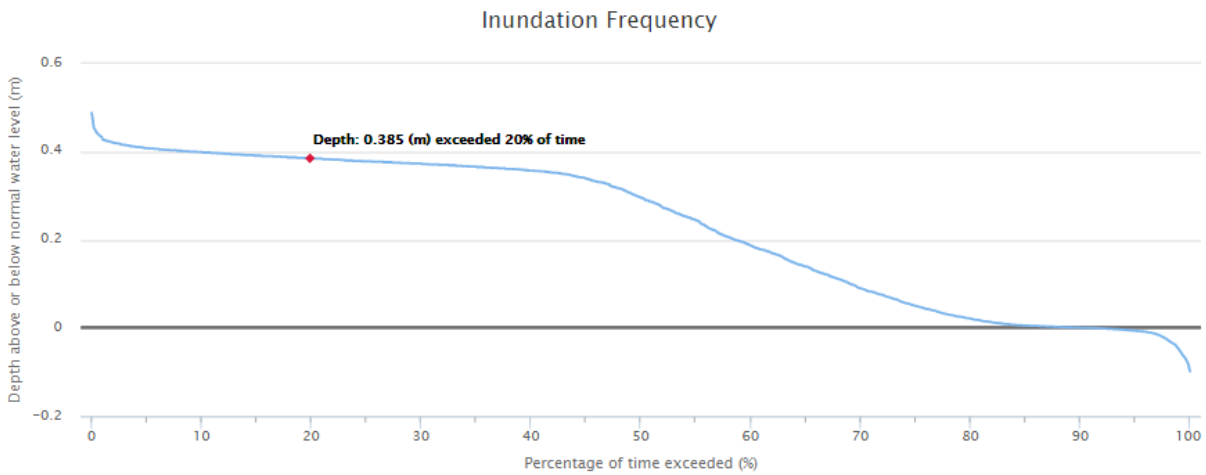
Report

File: NV5.csv
Shallow marsh zone meets deemed to comply criteria

Deep marsh zone meets deemed to comply criteria

Water level exceeded for 20% of time: 0.385 m

90th Percentile Residence Time: 3 days



SWS Check:

Wetland Plant Health Requirements Check - Hancocks Gully North					
Critical Plant Depth = Plant Height relative to NWL (m)			10 Year NWN data 1984 - 1993		
Extended Detention W1 = 0.35 m					
Shortest Allowable Average Plant Heights in Wetland 1 =			1.5	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 and Sea Club 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				Check 1 - Critical Plant Height Relative to NWL	
	Max Permanent Pool Depth	Average Height of Plant in wetland plant list given any exclusions	50% Average Plant Height	(Average plant height/2)-PP	
Shallow Marsh	0.15	1.5	0.75	0.60	
Deep Marsh	0.35	1.5	0.75	0.40	
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 =			
				385.0	mm
Percentile	**WL Relative to NWL - (mm)**				
0.5%	-66				
1.0%	-49	**Water level (relative to NWL) expected 80% of the time or more < CHECK (1) Shallow Marsh?**			
5.0%	-6				**YES**
15.0%	6				**Check1 met**
20.0%	21				
50.0%	296				
70.0%	373				
80.0%	385	**Water level (relative to NWL) expected 80% of the time or more < CHECK (1) Deep Marsh?**			
97.0%	415				**YES**
99.5%	442				**Check1 met**

A. 7 Sediment Pond Isolation Pipe Design

The following details culvert calculations to size bypass pipes for both sediment ponds S2 and S3 capable of bypassing the flow that occurs 96% of the time (from MUSIC modelling). Pit names refer to those detailed in drawing HCGN/SWS/6.

Bypass provisions for maintenance - NW Sediment Pond					
Bypass provisions from S2 (Pit A) to Outlet Pit					
Music File:	Hancocks_main_bypass_NV5_SV3_23Aug16.sqz				
Flow Frequency Analysis		Maximum flow calculated =	1.0	m^3/s	
			< Q 1Yr from RORB		
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%	0.001	Assume sediment pond to be isolated for maintenance.			
70.0%	0.003				
80.0%	0.005	**Bypass flow for maintenance activity > flow which occurs 96% of the time =**			
96.0%	0.055			**0.055**	**m^3/s**
99.5%	0.227				
Culvert A-B					
Upstream Pit IL =			40.1	m AHD	
Upstream Natural Surface =			42	m AHD	
Downsteam Pit IL =			40.1	m AHD	
Downstream Pit WL =			40.4	m AHD	
Maximum Maintenance bypass pipe length =			125	m	
Maximum Head Loss =		1.6	m		
culvert flowing full					
pipe dia =		0.3	m		
RCP pipe radius =		0.15	m		
Design flow =		0.055	m^3/s		
Wetted perimeter =		0.94	m		
Area =		0.07	m^2		
Hyd radius =		0.075	m		
V =		0.77	m/s		
Ke =		0.5			
Kex =		1			
n =		0.013			
L=		80			
S_f =		0.0032			
Head loss =		0.30	m		

Culvert B-C			
Upstream Pit IL =		40.1	m AHD
Upstream Natural Surface =		41.8	m AHD
Downsteam Pit IL =		39	m AHD
Downstream Pit WL =		39.3	m AHD
Maximum Maintenance bypass pipe length =		130	m
Maximum Head Loss =	2.5	m	
culvert flowing full			
pipe dia =	0.3	m	
RCP pipe radius =	0.15	m	
Design flow =	0.055	m^3/s	
Wetted perimeter =	0.94	m	
Area =	0.07	m^2	
Hyd radius =	0.075	m	
V =	0.77	m/s	
Ke =	0.5		
Kex =	1		
n =	0.013		
L=	130		
S_f =	0.0032		
Head loss =	0.46	m	
Culvert C-Outlet			
Upstream Pit IL =		39	m AHD
Upstream Natural Surface =		41	m AHD
Downsteam Pit IL =		38.3	m AHD
Downstream Pit WL =		38.75	m AHD
Maximum Maintenance bypass pipe length =		20	m
Maximum Head Loss =	2.25	m	
culvert flowing full			
pipe dia =	0.3	m	
RCP pipe radius =	0.15	m	
Design flow =	0.055	m^3/s	
Wetted perimeter =	0.94	m	
Area =	0.07	m^2	
Hyd radius =	0.075	m	
V =	0.77	m/s	
Ke =	0.5		
Kex =	1		
n =	0.013		
L=	20		
S_f =	0.0032		
Head loss =	0.11	m	

Bypass provisions for maintenance - NE Sediment Pond

Bypass provisions from S3 (Pit D) to Outlet Pit

Music File: Hancocks_main_bypass_NV5_SV3_15Jun16.sqz

Flow Frequency Analysis — Maximum flow calculated = 1.2 m^3/s

< Q 1Yr from RORB

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%	0.007
80.0%	0.011
96.0%	0.067
99.5%	0.270

Assume sediment pond to be isolated for maintenance.

Bypass flow for maintenance activity > flow which occurs 96% of the time = 0.067 m^3/s

Culvert D-E

Upstream Pit IL =	40.05	m AHD
Upstream Natural Surface =	41.8	m AHD
Downsteam Pit IL =	40.05	m AHD
Downstream Pit WL =	41.02	m AHD
Maximum Maintenance bypass pipe length =	100	m
Maximum Head Loss =	0.78	m

culvert flowing full

pipe dia =	0.3	m
RCP pipe radius =	0.15	m
Design flow =	0.067	m^3/s
Wetted perimeter =	0.94	m
Area =	0.07	m^2
Hyd radius =	0.075	m
V =	0.95	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	80	
S_f =	0.0048	
Head loss =	0.45	m

Culvert E-F			
Upstream Pit IL =		40.1	m AHD
Upstream Natural Surface =		42	m AHD
Downsteam Pit IL =		39.5	m AHD
Downstream Pit WL =		40.35	m AHD
Maximum Maintenance bypass pipe length =		110	m
Maximum Head Loss =	1.65	m	
culvert flowing full			
pipe dia =	0.3	m	
RCP pipe radius =	0.15	m	
Design flow =	0.067	m^3/s	
Wetted perimeter =	0.94	m	
Area =	0.07	m^2	
Hyd radius =	0.075	m	
V =	0.95	m/s	
Ke =	0.5		
Kex =	1		
n =	0.013		
L=	110		
S_f =	0.0048		
Head loss =	0.59	m	
Culvert F-G			
Upstream Pit IL =		39.5	m AHD
Upstream Natural Surface =		42	m AHD
Downsteam Pit IL =		39	m AHD
Downstream Pit WL =		39.90	m AHD
Maximum Maintenance bypass pipe length =		80	m
Maximum Head Loss =	2.10	m	
culvert flowing full			
pipe dia =	0.3	m	
RCP pipe radius =	0.15	m	
Design flow =	0.067	m^3/s	
Wetted perimeter =	0.94	m	
Area =	0.07	m^2	
Hyd radius =	0.075	m	
V =	0.95	m/s	
Ke =	0.5		
Kex =	1		
n =	0.013		
L=	80		
S_f =	0.0048		
Head loss =	0.45	m	
Culvert G-Outlet			
Upstream Pit IL =		39	m AHD
Upstream Natural Surface =		42	m AHD
Downsteam Pit IL =		38.3	m AHD
Downstream Pit Obvert =		38.6	m AHD
Maximum Maintenance bypass pipe length =		258	m
Maximum Head Loss =	3.4	m	
culvert flowing full			
pipe dia =	0.3	m	
RCP pipe radius =	0.15	m	
Design flow =	0.067	m^3/s	
Wetted perimeter =	0.94	m	
Area =	0.07	m^2	
Hyd radius =	0.075	m	
V =	0.95	m/s	
Ke =	0.5		
Kex =	1		
n =	0.013		
L=	258		
S_f =	0.0048		
Head loss =	1.30	m	

A. 8 Liner and Topsoil

The soil profile in the area is expected to be clayey in nature. The development of wetlands proposed in the adjacent Cardinia Industrial DSS were formulated given in-depth hydro geological study performed by Sinclair Knight Merz “Cardinia Road Precinct Development, Installation of Shallow Groundwater Bores, Final Report, 20 August 2008”.

One of the primary conclusions of the 2008 study was that the insitu clayey soil is ideal for formation of wetland systems as the interaction between groundwater and surface water is extremely slow. As such the surface water systems (i.e. wetlands) can essentially be considered separate from the groundwater systems. SKM concluded that the NWL’s etc proposed in the Cardinia Industrial DSS can be achieved and should not adversely affect surface water/groundwater interaction in the area. Given this finding clay lining was not required as part of the concept design proposals for the Cardinia Industrial DSS systems.

As such, it is assumed at this stage that clay lining will not be required for the Hancocks Gully North DSS wetland. Compaction of the existing clayey soil and placement of topsoil as required should be specified at the detailed design stage of the project.

The detailed design stage of the project should include compressive soil tests of the subject site to confirm the above assumption.

A. 9 Constructed Wetlands Design Manual, Part A.2: Checklist

General		
Criteria	*Description*	*Reference*
GN1	The treatment and flow regime performance of the wetland must be modelled in MUSIC, or similar conceptual modelling software as approved by Melbourne Water.	Met with MUSIC modelling (Section 6)
GN2	The meteorological data in the conceptual modelling data or software (i.e. MUSIC) must be:	MUSIC modelling (Section 6) meets criteria
	• Based on at least 10 years of historical records	
	• Recorded at six minutes' intervals	
	• Sourced from a pluviographic station as close as possible to the wetland site	
	• Have a mean annual rainfall depth within 10% of the long term rainfall depth at the rainfall station closest to the wetland site	
GN3	The system configuration shown on the design plans must be consistent with the conceptual modelling parameters (e.g. MUSIC) (including the stage/discharge relationship) and sediment pond calculator/calculations.	Shown in Section 6 and Addendum A. 1
GN4	Peak design flows must be estimated in accordance with methods in Australian Rainfall and Runoff.	Met with RORB modelling (Section 4)

Maintenance Provisions		
Criteria	*Description*	*Reference*
MN1	Sediment ponds must be able to be drained whilst maintaining the macrophyte zone water level at normal water level.	Met as Sediment Ponds have a higher WL 1m than W1 (See HCGN/SWS/ 6)
MN2	All parts of the base of a sediment pond must be accessible:	Shown in: HCGN/SWS/ 2
	• Within seven metres of a designated hard stand area for excavation vehicles ("edge cleaned") OR	
	• Via a maintenance access ramp into the base of the sediment pond	
MN5	Maintenance access ramps are required on all sediment ponds that cannot be 'edge cleaned'. The maintenance access ramp into a sediment pond must:	Shown in: HCGN/SWS/ 2
	• Extend from the base of the sediment pond to at least 0.5 metres above TED,	
	• Be at least 4 metres wide,	
	• Be no steeper than 1:5	
	• Be capable of supporting a 20 tonne excavator	
	• Constructed of either:	
	- 200 mm deep layer of cement treated crushed rock (6%), or	
	- 200 mm compacted FCR	
	• Have a barrier to prevent unauthorised vehicle access (e.g. gate, bollard and/or fence).	
MN6	A maintenance access track must be provided to the sediment pond maintenance access ramp and to enable maintenance vehicles to safely access and exit the site. The maintenance access track must:	Shown in: HCGN/SWS/ 2
	• Be at least 4 metres wide	
	• Comprise of compacted FCR minimum 200 mm depth	
	• Be reinforced to take a 20 tonne vehicle	
	• At the road edge, have an industrial crossover to Council standard and rolled kerb adjoining it.	
MN7	A hardstand area with a minimum turning circle appropriate to the types of maintenance vehicles to be used must be provided adjacent to the sediment pond maintenance access ramp to enable maintenance vehicles to safely reverse and exit the sediment loading area. (Designers should seek advice from Melbourne Water on the types of maintenance vehicles that will be used.)	Shown in: HCGN/SWS/ 2
	Note: The turning circle must be in accordance with the Austroads Design Vehicles and Turning Path Templates Guide: (http://www.austroads.com.au/images/stories/ap-g34-13.pdf)	
MN9	Dedicated sediment dewatering areas must be provided and:	Shown in: HCGN/SWS/ 2 Note: 15m offset from reserve boundary cannot be met for S3 due to assumption that roads will be located to the north of the boundary. Can be altered to the south (adequate room) if required in future with ramp to come in from the south as well.
	• Be accessible from the maintenance ramp,	
	• Have a length to width ratio no narrower than 10:1,	
	• Be able to contain all sediment removed from the sediment accumulation volume spread out at 500 mm depth	
	• Be located above the peak 10 year ARI water level and within 25 metres of each sediment pond or as close as possible,	
	• Be located at least 15 metres from residential areas and public access areas (like pathways, roads, playgrounds, sports fields etc), and consider potential odour and visual issues for local residents	
	• Address public safety and potential impacts on public access to open space areas,	
	• Be free from above ground obstructions (e.g. light poles) and be an area that Melbourne Water has legal or approved access to for the purpose of dewatering sediment.	
MN10	The wetland must be configured to enable maintenance vehicles to drive around at least 50% of the wetland perimeter. Vehicular access must be provided as close as possible to wetland structures that may catch debris (e.g. provide access to the closest bank where structures are within the water body).	To be designed when landscape architect engaged

Sediment Pond		
Criteria	*Description*	*Reference*
SP1	Sediment ponds must be located offline of waterways but online to the pipe or lined channel they are treating water from. Refer to Part A3 of the Manual for guidance on offline configurations.	Met assuming DSS pipeline alignments do not change. See HCGN/SWS/ 2
SP2	Sediment ponds must be located at each point stormwater enters the “wetland system” unless:	Met assuming DSS pipeline alignments do not change. See HCGN/SWS/ 2
	• The catchment of the incoming stormwater is < 5% of the total wetland catchment OR	
	• The incoming stormwater has already passed through a bioretention system or wetland immediately upstream	
SP3	Sediment ponds must be sized to:	Calculations Shown in Section 5 and Addendum A. 3
	• Capture 95% of coarse particles ≥ 125 µm diameter for the peak three month ARI AND	
	• Provide adequate sediment storage volume to store three to five years sediment. The top of the sediment accumulation zone must be assumed to be 500 mm below NWL AND	
	• Ensure that velocity through the sediment pond during the peak 100 year ARI event is ≤ 0.5 m/s. (The flow area must be assumed to be the EDD multiplied by the narrowest width of the sediment pond, at NWL, between the inlet and overflow outlet)	
	Sediment ponds must be ≤ 120% of the size needed to meet the limiting of the above three criteria. Compliance with the above criteria must be demonstrated using the methods described in WSUD Engineering Procedures: Stormwater (Melbourne Water, 2005). Alternatively, the velocity criteria can be checked using a hydraulic model such as HEC-RAS. Refer to Part D of the Manual for guidance on undertaking velocity checks).	
SP4	The sediment pond EDD must be ≤ 350 mm.	S1 & S2 ED = 0 mm

Macrophyte Zone		
Criteria	*Description*	*Reference*
MZ1	At least 80% of the area of the macrophyte zone at NWL must be ≤ 350 mm deep to support shallow and deep marsh vegetation. The wetland bathymetry should provide approximately equal amounts of shallow marsh (≤ 150 mm deep) and deep marsh (150 mm to 350 mm deep).	Calculation shown in Section 3.1. Also refer to HCGN/SWS/ 2
MZ2	The macrophyte zone EDD must be ≤ 350 mm.	Shown in Section 3.1
MZ3	Macrophyte zones must be located offline from all waterways and drains (i.e. there must be a bypass route around the macrophyte zone).	FAIL Impossible due to nature of Hancocks Gully and PSP boundary allocated
MZ4	The length of the macrophyte zone must be ≥ four times the average width of the macrophyte zone.	Shown in: HCGN/SWS/ 2 Length = 600 m Width = 40 m
MZ5	Major inlets to the macrophyte zone (i.e. those draining > 10% of the catchment to be treated) must be located within the first 20% of the macrophyte zone.	Shown in: HCGN/SWS/ 1
MZ6	The macrophyte zone outlet must be located at the opposite end of the macrophyte zone to the inlet(s).	Shown in: HCGN/SWS/ 2
MZ7	The macrophyte zone must have a sequence and mix of submerged, shallow and deep marsh zones arranged in a banded manner perpendicular to the direction of flow.	Shown in: HCGN/SWS/ 1
MZ8	Inlet and outlet pools must be ≤ 1.5 m depth.	Shown in: HCGN/SWS/ 2
MZ9	Intermediate pools (between the inlet and outlet pool) must be ≤ 1.2 m deep.	Shown in: HCGN/SWS/ 2
MZ10	Velocities in the macrophyte zone must be: • less than 0.5 m/s for the peak 100 year ARI flow • less than 0.05 m/s for the peak three month ARI Compliance with the above criteria must be demonstrated using the methods described in WSUD Engineering Procedures: Stormwater (Melbourne Water, 2005) or using a hydraulic model such as HEC-RAS or TUFLOW. Refer to Part D of the Manual for guidance on undertaking velocity checks.	Calculations shown in Addendum A. 4
MZ11	The macrophyte zone must provide a 90th percentile residence time of 72 hours (assuming plug flow between inlet and outlet through the EDD and 50% of the permanent pool volume). Refer to the Melbourne Water online tool and Part D of the Manual for guidance on determining residence time.	MUSIC Auditor Results = 3 days (Addendum A. 6)
MZ12	A minimum grade of 1:150 must be provided between marsh zones (longitudinally through the macrophyte zone) to enable the wetland to freely drain. Intermediate pools will generally be needed to transition between marsh zones.	Shown in: HCGN/SWS/ 2

Bypass		
Criteria	*Description*	*Reference*
BY1	The bypass route must be sized to convey the maximum overflow from the sediment pond that will occur during the peak 100 year ARI event. A bypass is still required for a wetland located within a retarding basin. Where a sediment pond is within a retarding basin, the bypass must convey at least the peak one year ARI flow.	Fail No bypass due to nature of Hancocks Gully and PSP boundary allocated
	Note: Refer to Melbourne Water Waterways Manual for channel design specifications when designing bypass routes.	

Inlets and Outlets		
Criteria	*Description*	*Reference*
IO4	The connection between the sediment pond and macrophyte zone must be sized such that:	Fail All sediment pond flows are transferred to W1 due to nature of Hancocks Gully and PSP boundary allocated. Connection shown in: HCGN/SWS/ 4
	• All flows ≤ the peak three month ARI event are transferred into the macrophyte zone when the EDD in the macrophyte zone is at NWL, AND	
	• 60% of the peak 1 year ARI flow overflows from the sediment pond into the bypass channel/pipe when the water level in the macrophyte zone is at TED (and not enter the macrophyte zone), AND	
	• The velocity through the macrophyte zone is ≤ 0.5 m/s during the peak 100 year ARI event:	
	i. Assuming the macrophyte zone is at TED if the wetland is not within a retarding basin or flood plain	
	ii. Assuming the water level is at the peak 10 year ARI water level if the wetland is within a retarding basin or flood plain	
IO6	The macrophyte zone controlled outlet must be configured so that:	All achievable with configuration shown in HCGN/SWS/ 3
	• The NWL can be drawn down by up to 150 mm during plant establishment and maintenance.	
	• The NWL can be permanently adjusted up or down by 100 mm to respond to changes in wetland hydrology due to potential future climate conditions.	
	• The stage/discharge rate can be adjusted if required to achieve suitable residence times and/or inundation patterns	
IO7	Balance pipes must be placed between all open water zones (inlet, intermediate and outlet pools) to enable water levels to be drawn down for maintenance or water level management purposes. Balance pipes must comprise of minimum 225 mm (e.g. sewer class PVC), the invert level of the pipes must be at no more than 100 mm above the base of the macrophyte zone and fitted with a truncated pit to minimise the risk of clogging (refer to Melbourne Water Standard Drawing for truncated pit details).	Shown in: HCGN/SWS/ **6**

Vegetation		
Criteria	*Description*	*Reference*
VG1	The macrophyte zone must contain a minimum 80% cover of emergent macrophytes calculated at NWL comprising of shallow and deep marsh zones. Open water areas (maximum 20% of the wetland area calculated at NWL) must include submerged vegetation.	Shown in Section 3.1 and Addendum A. 6
VG2	Any open water areas in excess of 20% of the macrophyte zone area (at NWL) must be located as a separate water body. These separate water bodies are not considered by Melbourne Water to be constructed wetlands for the purpose of treating stormwater, and are therefore beyond the scope of the Maunal. For further information, refer to Part A3 for open water, landscape design and amenity design considerations and the Planning and Building website for ownership and maintenance responsibilities. Conceptual models of wetlands and other parts of the treatment train (e.g. MUSIC) must assume there is no reduction in pollutant loads within these separate waterbodies.	No open water areas > 20%. See Section 3.1
VG3	Ephemeral batters (NWL to 200 mm above NWL) of the macrophyte zone and sediment pond must be densely planted with plants suited to intermittent wetting. 80% of the plants used in the ephemeral batters must be in accordance with the species shown in the Manual.	Zones Specified in HCGN/SWS/ 2 and plant list specified in Addendum A. 6
VG5	The shallow marsh (NWL to 150 mm below NWL) of the macrophyte zone and sediment pond must be densely planted. 90% of the plants used in the shallow marsh must be in accordance with the species and densities shown in the Manual. A minimum of three species must be specified for the shallow marsh zone.	Zones Specified in HCGN/SWS/ 2 and plant list specified in Addendum A. 6
VG6	The deep marsh (150 to 350 mm below NWL) of the macrophyte zone must be densely planted. 90% of the plants used in the deep marsh must be in accordance with the species and densities shown in the Manual. A minimum of three species must be specified for the deep marsh zone.	Zones Specified in HCGN/SWS/ 2 and plant list specified in Addendum A. 6
VG7	The submerged marsh (350 to 700 mm below NWL) of the macrophyte zone must be densely planted. 90% of the plants used in the submerged marsh must be in accordance with the species and densities shown in the Manual.	Zones Specified in HCGN/SWS/ 2 and plant list specified in Addendum A. 6
VG10	The effective water depth (permanent pool depth plus EDD) must not exceed half of the average plant height for more than 20% of the time. This must be demonstrated using inundation frequency analysis assuming the plants heights are in accordance with those shown in the Manual.	Zones Specified in HCGN/SWS/ 2 and plant list specified in Addendum A. 6
	Refer to online tool and Part D of the Manual for guidance on the inundation frequency analysis.	
VG11	Where stormwater is harvested from the permanent pool of a wetland, the extraction must not occur if the water level is more than 100 mm below NWL.	No harvesting assumed but needs to be met if harvesting is to occur as per 2016 DCE report
	Note: a diversion licence is required to harvest water from Melbourne Water assets (see Melbourne Water's Stormwater Harvesting Guidelines for more information).	

Liner and Topsoil		
Criteria	*Description*	*Reference*
LN1	The exfiltration rate from the base and the sides of the wetland must be accurately represented in the conceptual modelling software analysis (e.g. MUSIC). Wetlands with a permanent pool generally have a compacted clay liner made from site soils and/or imported material where site soils are unsuitable. Where no liner is proposed, in-situ geotechnical testing (at the depth of the wetland base) must be undertaken and used to justify the selected exfiltration rate used in modelling.	Discussion provided in Addendum A. 8
LN3	At least 200 mm topsoil must be provided in all areas of the macrophyte zone; and in sediment ponds to 350 mm below NWL.	See note in HCGN/SWS/ 2

Landscape Design Structures		
Criteria	*Description*	*Reference*
LDS2	All boardwalks, bridges and formal pedestrian paths, must be at or above the peak 10 year ARI water level. Refer to Melbourne Water's Shared Pathway Guidelines and Jetties Guidelines for more information.	To be designed when landscape architect engaged
LDS3	Boardwalks or viewing platforms are not permitted over sediment ponds.	To be designed when landscape architect engaged

Edge Treatment		
Criteria	*Description*	*SWS Location*
ET1	The edge of any deep open water should not be hidden or obscured by embankments or terrestrial planting unless measures are taken to preclude access. Public access to structures, the top of weirs, orifice pits and outlet structures must be restricted by appropriate safety fences and other barriers. Permanent fencing is required adjacent to potentially unsafe structures (i.e. deep water zones, steep drops, top of weirs, outlet structures etc).	See note in HCGN/SWS/ 2
ET2	All wetland edges must have: • Vegetated approach batters no steeper than 1:5, a 2.8 metre wide vegetated safety bench at 1:8 between NWL and 350 mm below NWL and a maximum 1:3 slope beyond 350 mm below NWL OR • Batters no steeper than 1:4 between TED and 350 mm below NWL with dense impenetrable planting that is a minimum of 2.8 metres wide and 1.2 metres high.	See note in HCGN/SWS/ 2
ET4	A minimum offset of 15 metres must be provided from the edge of the water at NWL to any allotment or road reserve (not including shared pathways). A safety design audit is required for any proposal that does not achieve this condition. Refer to Part A3 of the Manual for design consideration and guidance on safety in design.	Met as shown in HCGN/SWS/ 2

Addendum B – Hancocks Gully South Wetland/Retarding Basin (W2) Design

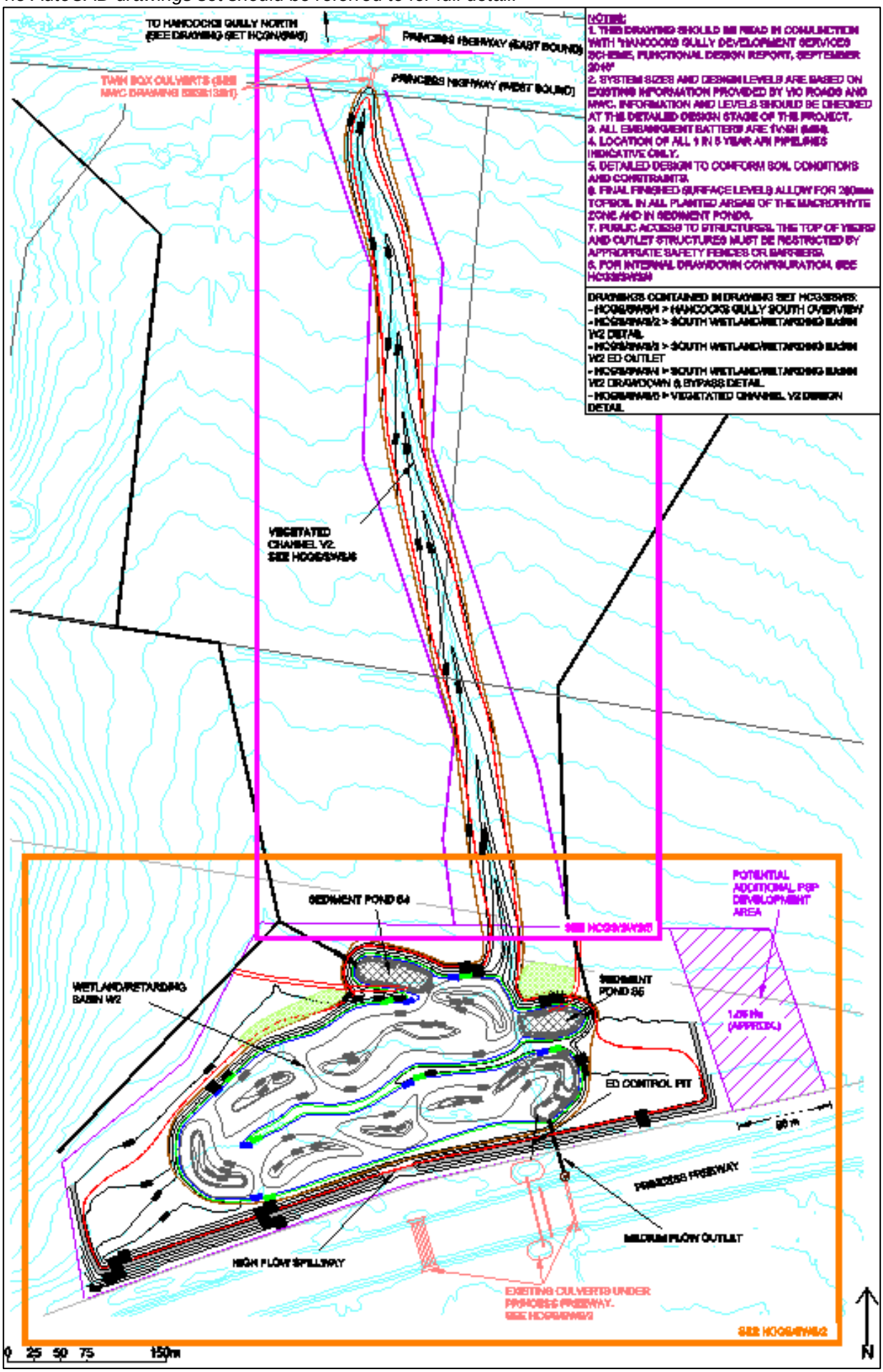
The following section indicates the design and calculations for Hancocks Gully South's (W2) functional design.

Addendum B Contents:

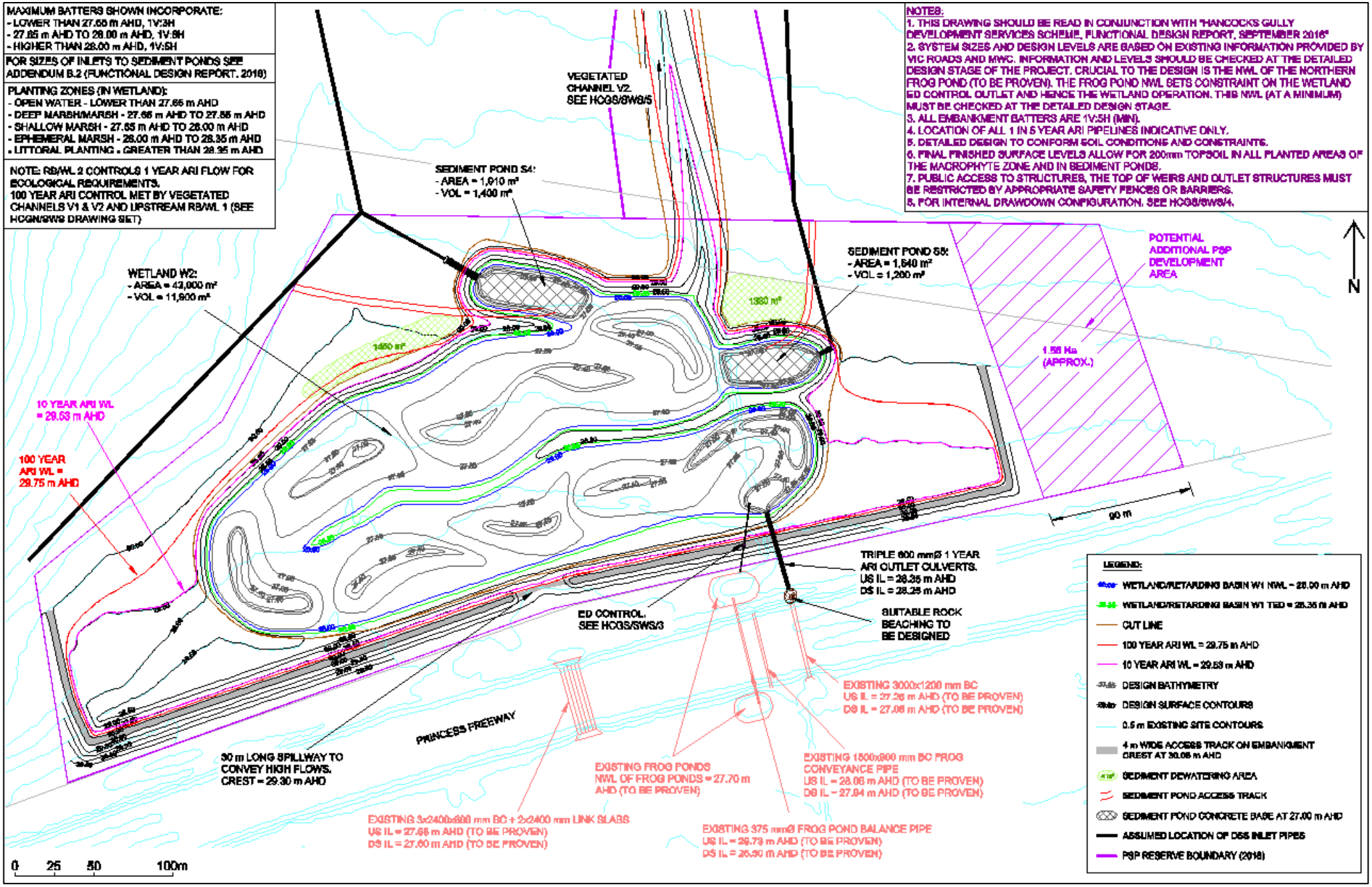
B. 1 Functional Design Drawings 73
B. 2 Functional Design Calculations 78
B. 3 Sediment Pond Design 81
B. 4 Wetland Velocity Checks 85
B. 5 South Wetland MUSIC Results 88
B. 6 Wetland Inundation Check 89
B. 7 Liner and Topsoil 92
B. 8 Constructed Wetlands Design Manual, Part A.2: Checklist 93

B. 1 Functional Design Drawings

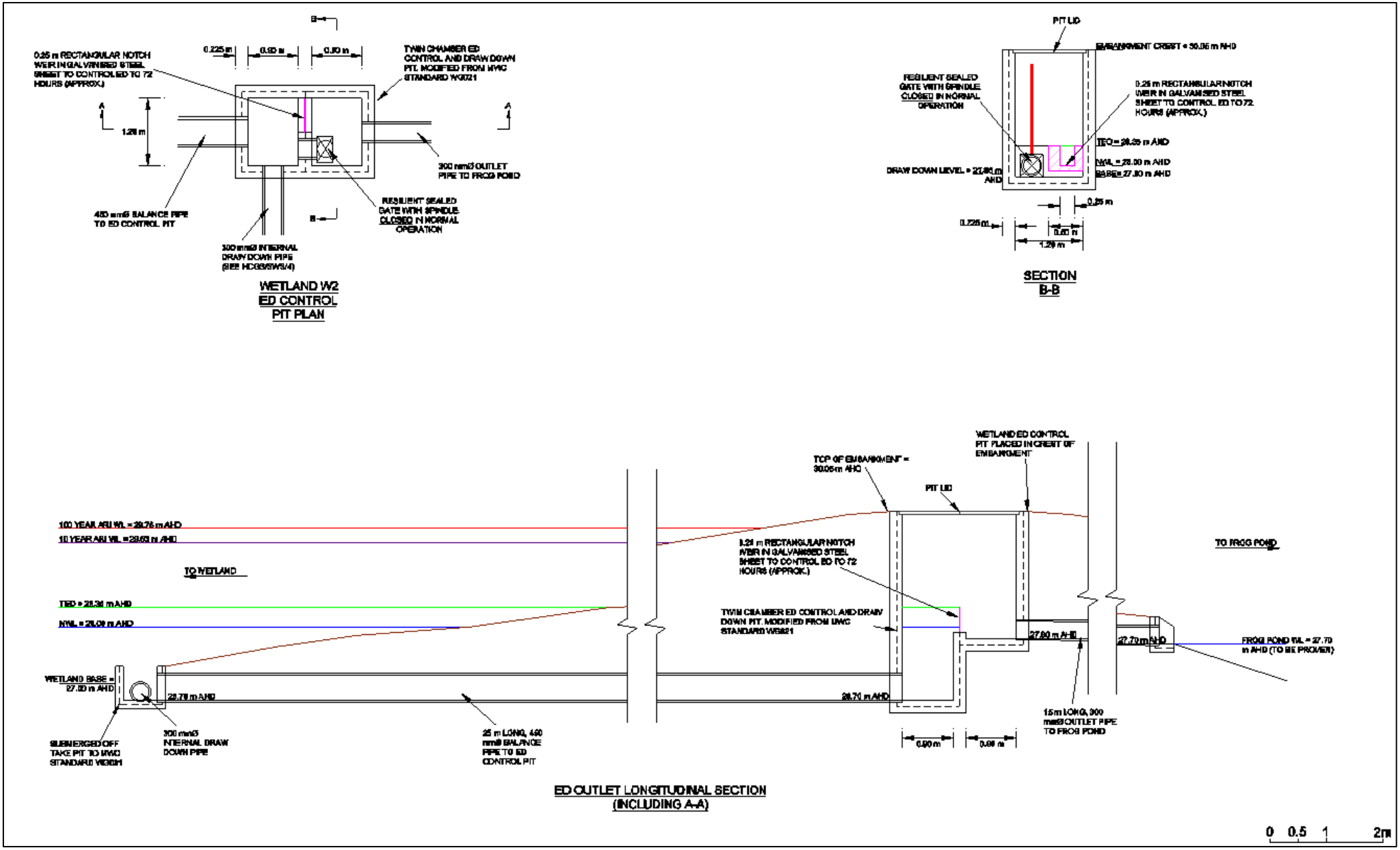
Note: The AutoCAD drawings set should be referred to for full detail.



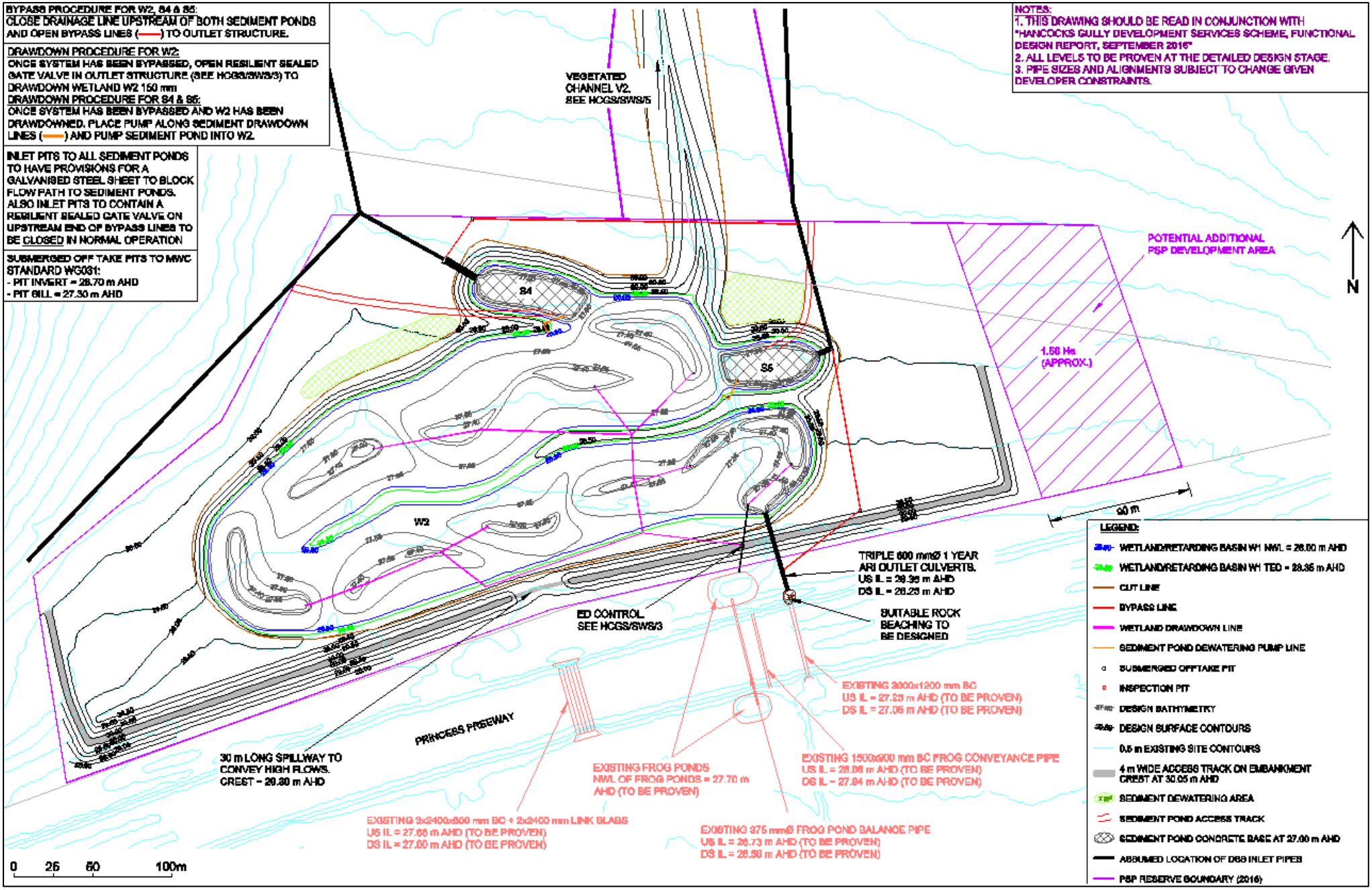
HCGS/SWS/ 1 Hancocks Gully South Overview



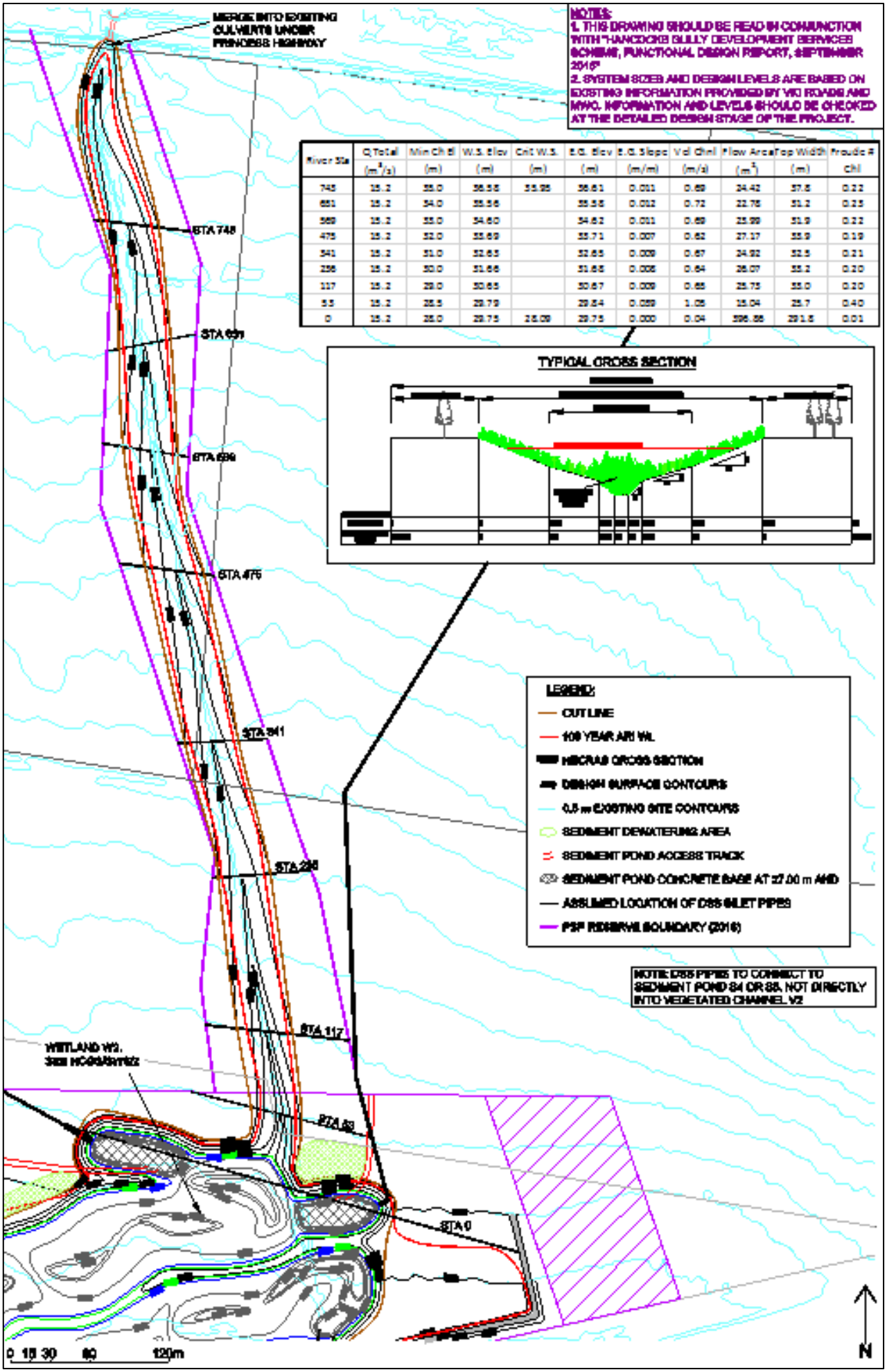
HCGS/SWS/ 2 South Wetland/Retarding Basin W2 Detail



HCGS/SWS/ 3 South Wetland/Retarding Basin W2 ED Outlet



HCGS/SWS/ 4 South Wetland/Retarding Basin W2 Drawdown & Bypass Detail



River Sta	Q Total (m^3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m^2)	Top Width (m)	Froude # Chl
745	15.2	55.0	56.56	55.95	56.61	0.011	0.69	24.42	57.6	0.22
651	15.2	54.0	55.56		55.58	0.012	0.72	22.76	51.2	0.23
569	15.2	53.0	54.60		54.62	0.011	0.69	25.99	51.9	0.22
475	15.2	52.0	53.69		53.71	0.007	0.62	27.17	53.9	0.19
341	15.2	51.0	52.65		52.65	0.009	0.67	24.92	52.5	0.21
256	15.2	50.0	51.66		51.68	0.006	0.64	26.07	53.2	0.20
117	15.2	29.0	30.65		30.67	0.009	0.65	25.75	53.0	0.20
53	15.2	28.5	29.79		29.84	0.059	1.05	15.04	25.7	0.40
0	15.2	28.0	29.75	28.09	29.75	0.000	0.04	396.66	291.5	0.01

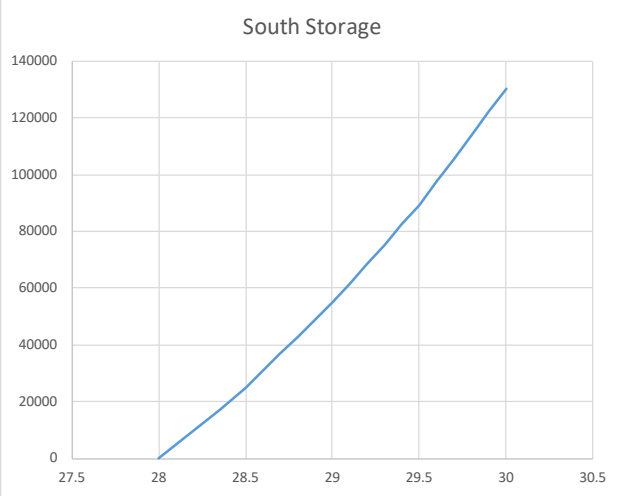
HCGS/SWS/ 5 Vegetated Channel V2 Design Detail

B. 2 Functional Design Calculations

Retarding Basin Stage Storage Relationship

Level (m)	Area (m^2)	Average Area (m^2)	delta h (m)	Volume (m^3)	Cumulative Volume (m^3)
28	46408				0
28.35	50402	48405	0.35	16941.75	16941.75
28.5	56357	53379.5	0.15	8006.925	24948.675
29	63300	59828.5	0.5	29914.25	54862.925
29.5	74003	68651.5	0.5	34325.75	89188.675
30	90595	82299	0.5	41149.5	130338.175

Level (m)	Cumulative Volume (m^3)	
28	0	
28.35	16942	
28.50	24949	
28.6	30932	Linear Approx
28.7	36914	Linear Approx
28.8	42897	Linear Approx
28.95	51871	Linear Approx
29	54863	
29.1	61728	Linear Approx
29.2	68593	Linear Approx
29.3	75458	Linear Approx
29.4	82324	Linear Approx
29.5	89189	
29.6	97419	Linear Approx
29.7	105648	Linear Approx
29.8	113878	Linear Approx
29.9	122108	Linear Approx
30	130338	



Wetland rectangular notch ED Control

Q = B × C × Le × h1.5

where

Q = flow rate (m3/s)

h = head on the weir (m)

B = blockage factor (assume no blockage as in manual)

C = weir coefficient = 1.7 sharp crested weir

L = Actual Weir Length = 0.25 m

Area at NWL = 46548 m^2

Area at TED = 50422 m^2

Volume of water stored for treatment over Ed range 0.35 m

.= 16969.75 m^3

L e = effective length = L - 0.2h, where L = Actual Weir Length

WL (m AHD)	h (m)	Le (m)	Q (m^3/s)	ED Volum	ED Detention Time (hrs)
28	0	0.25	0.000	0	
28.35	0.35	0.18	0.063	16970	74

Calculation of Outflow Relationship from Southern Hancocks Gully

All levels approximate lony and to be confimed

Number of Culverts =	3	
Diameter =	0.600	m
Length =	50	m
Upstream IL =	28.35	m AHD
Downstream IL =	28.25	m AHD
Downstream Obvert =	28.85	m AHD
Long. Slope (1/)=	500.00	
Mannings n =	0.013	
Upstream Obvert =	28.950	m AHD

Water Level (m AHD)	$Q_{Culvert, Outlet Control}$ (m^3/s)	$Q_{Culvert, Inlet Control}$ (m^3/s)	$Q_{High Flow Weir}$ (m^3/s)	Q_{TED} (m^3/s)	Q_{out} (m^3/s)	Controlling
28.000	0.00	0.00		0.000	0.00	NWL
28.350	0.00	0.00		0.063	0.06	TED Range
28.65	0.32	0.62		0.063	0.38	Culvert Control
28.95	0.63	1.24		0.063	0.69	
29.10	1.00	1.51		0.063	1.06	
29.20	1.18	1.67		0.063	1.24	
29.30	1.33	1.82	0.00	0.063	1.40	
29.40	1.47	1.95	1.90	0.063	3.43	Spillway
29.50	1.60	2.08	5.36	0.063	7.03	
29.60	1.72	2.20	9.84	0.063	11.62	
29.70	1.83	2.31	15.14	0.063	17.04	
29.80	1.94	2.42	21.14	0.063	23.15	
29.90	2.03	2.52	27.77	0.063	29.87	
30.00	2.13	2.62	34.98	0.063	37.17	

Retarding Basin Stage/Storgae/Discharge Relationship

Level (m)	Storage (m^3)	Flow (m^3/s)	
28.0	0	0.00	NWL
28.35	16942	0.06	TED Range
28.7	36914	0.38	Culvert Control
29.0	54863	0.69	
29.1	61728	1.06	
29.2	68593	1.24	
29.3	75458	1.40	
29.4	82324	3.43	Spillway
29.5	89189	7.03	
29.6	97419	11.62	
29.7	105648	17.04	
29.8	113878	23.15	
29.9	122108	29.87	
30.0	130338	37.17	

Balance/Connection Pipes to ED control Structure and to Frog Pond:

BALANCE PIPE FOR ED OUTLET			OUTLET PIPE FROM ED OUTLET TO FROG PONDS		
head loss = (Ke+Kex)×V^2/2g+ S_f×L			head loss = (Ke+Kex)×V^2/2g+ S_f×L		
$S_f = Q^2n^2/A^2R^{4/3}$			$S_f = Q^2n^2/A^2R^{4/3}$		
pipe dia =	0.450	m	pipe dia =	0.300	m
RCP pipe radius =	0.225	m	RCP pipe radius =	0.15	m
Design flow =	0.06	m^3/s	Design flow =	0.06	m^3/s
Wetted perimeter =	1.41	m	Wetted perimeter =	0.94	m
Area =	0.16	m^2	Area =	0.07	m^2
Hyd radius =	0.1125	m	Hyd radius =	0.075	m
V =	0.40	m/s	V =	0.90	m/s
Ke =	0.5		Ke =	0.5	
Kex =	1		Kex =	1	
n =	0.013		n =	0.013	
L=	25		L=	15	
S_f =	0.0005		S_f =	0.0043	
Head loss =	0.024	m	Head loss =	0.126	m
				<350mm Head loss - OK	

Sediment Ponds S4 & S5 Inlet Pipe Sizing:

Location	SW Sed (S4)	SE Sed (S5)
Q_5 (m^3/s, RORB)	5.4	3.6
Pipe Size (m)	1.2	1.2
HGL slope (1/x)	300	300
Number of pipes	3	2
Pipe Capacity (m^3/s)	6.8	4.5
Pipe Velocty (m/s)	1.99	1.99

Princess Freeway Culvert Capacity:

Capacity of Hancocks Gully culvert system at Freeway

Based on vic Roads Drawing 583249
Assume all culverts flowing under outlet control
and head loss = 0.3 to ensure carriageway not engaged

head loss = (Ke+Kex)×V^2/2g+ S_f×L
$S_f = Q^2n^2/A^2R^{4/3}$

1. Five 2400 by 600 Box Culvert System			**2. Frog pond connection**			**3. Frog Culvert - One 1500 by 900**			**4. Box Culvert - One 3000 by 1200**		
			One 375 mm dia								
W =	2.4	m	pipe dia =	0.375	m	**W =**	1.5	m	**W =**	3	m
D =	0.6	m	RCP pipe radius =	0.1875	m	D =	0.9	m	D =	1.2	m
			Design flow =	0.15	m^3/s						
Capacity flow/cell =	2.27	m^3/s	Wetted perimeter =	1.18	m	Capacity flow/cell =	2.2	m^3/s	Capacity flow/cell =	6.3	m^3/s
Wetted perimeter =	6.00	m	Area =	0.11	m^2	Wetted perimeter =	4.80	m	Wetted perimeter =	8.40	m
Area =	1.44	m^2	Hyd radius =	0.09375	m	Area =	1.35	m^2	Area =	3.60	m^2
Hyd radius =	0.24	m	V =	1.36	m/s	Hyd radius =	0.28125	m	Hyd radius =	0.428571	m
V =	1.58	m/s	Ke =	0.5		V =	1.63	m/s	V =	1.75	m/s
Ke =	0.5		Kex =	1		Ke =	0.5		Ke =	0.5	
Kex =	1		n =	0.013		Kex =	1		Kex =	1	
n =	0.013		L=	25		n =	0.013		n =	0.013	
L=	43.3		S_f =	0.0073		L=	43.3		L=	43.3	
S_f =	0.0028					S_f =	0.0024		S_f =	0.0016	
			Head loss =	0.3	m						
Head loss =	0.31	m				Head loss =	0.31	m	Head loss =	0.30	m
Total Flow =	11.35	m^3/s	Total Flow =	0.15	m^3/s	Total Flow =	2.2	m^3/s	Total Flow =	6.3	m^3/s
Total system Design Flow =	20	m^3/s									

B. 3 Sediment Pond Design

Calculation of South West (S4) Sediment Pond Size

NWL =		28.00	m AHD					
TED =		28.35	m AHD					
Safety batters at 1 in		8.00	to 350 below NW					
Base level =		27.00	m AHD					
Area at NWL =		1907.00	m^2					
Total Volume =		1401.73	m^3					
Sump Volume (volume below350 mm depth) =					819.32	m^3		
Batters - General								
Batters above NWL =			1 in 6					
Batters NWL to 350 below NWL=				1 in 8				
Batters 350 below NWL to base=				1 in 3				
Batters - access track								
Batters above NWL =			1 in 15					
Batters NWL to 500 below NWL=				1 in 8				
Batters 500 below NWL to base=				1 in 5				
Concrete base								
Sediment Removal								
Fair and Geyer Equation								
$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$			$\lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$					
Vs =	0.011	m/s	*Target = very fine sand*					
d_e =	0.35	m						
d_p =	1	m						
d* =	1	m						
(d_e+d_p) =	1							
(d_e+d^*)								
Q_{3mth} =	1.08	m^3/s	(RULE OF	Q_5 =	5.4	m^3/s (RORB)		
A =	1907.00	m^2						
V_s =	19.42							
Q/A								
λ =	0.11	pond shape assumption						
n =	1.12							
Fraction of Initial Solids Removed								
R =	96%							
Requirement: Melbourne Water require R = 95% for a 125 micrometer particle								
				for 3 month event				

Cleanout Frequency

Catchment Area =	75.478	ha	Areas AW-BG			
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)			
Sump Volume =	819	m^3	area between base and 0.35 m below NWL			
Therefore, cleanout frequency required =		$R(1.6+0.4)A_{catchment}$ / sump volume		0.2	per year	
(sediment to 500 below NWL)			.= every	6	years	OK

Assumes cleanout when sump volume of basin is full (ie sediment 350 mm below NWL)

Sediment Dewatering Area

Dewatering depth =	0.5	m			
Sediment volume collected every 5 years=				726	m^3
Required Dewatering area =	1452	m^2			

Ensure this area is provided near the sediment pond and is accessible with machinery/access tracks etc.

Dewatering Provision for Maintenance

Bypass as described in drawing HCGS/SWS/3

Drawdown wetland to 27.85 m AHD with resilient gate valve shown in HCGS/SWS/2

Place pump in pump pit and pump water into wetland

Sediment Pond Flood Flow Velocity Check

See Addendum B.4

Calculation of South East (S5) Sediment Pond Size

NWL =		28.00	m AHD					
TED =		28.35	m AHD					
Safety batters at 1 in		8.00	to 350 below NW					
Base level =		27.00	m AHD					
Area at NWL =		1641.00	m^2					
Total Volume =		1197.68	m^3					
Sump Volume (volume below350 mm depth) =					698.75	m^3		
Batters - General								
Batters above NWL =			1 in 6					
Batters NWL to 350 below NWL=				1 in 8				
Batters 350 below NWL to base=				1 in 3				
Batters - access track								
Batters above NWL =			1 in 15					
Batters NWL to 500 below NWL=				1 in 8				
Batters 500 below NWL to base=				1 in 5				
Concrete base								
Sediment Removal								
Fair and Geyer Equation								
$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$			$\lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$					
Vs =	0.011	m/s	*Target = very fine sand*					
d_e =	0.35	m						
d_p =	1	m						
d* =	1	m						
(d_e+d_p) =	1							
(d_e+d^*)								
Q_{3mth} =	0.72	m^3/s	(RULE OF	Q_5 =	3.60	m^3/s (RORB)		
A =	1641	m^2						
V_s =	25.1							
Q/A								
λ =	0.11	pond shape assumption						
n =	1.12							
Fraction of Initial Solids Removed								
R =	97%							
Requirement: Melbourne Water require R = 95% for a 125 micrometer particle								
				for 3 month event				

Cleanout Frequency

Catchment Area =	68.471	ha	Areas BH-BQ			
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)			
Sump Volume =	699	m^3	area between base and 0.35 m below NWL			
Therefore, cleanout frequency required =		$\frac{R(1.6+0.4)A_{catchment}}{\text{sump volume}}$ =		0.2	per year	
(sediment to 500 below NWL)			.= every	5	years	OK

Assumes cleanout when sump volume of basin is full (ie sediment 350 mm below NWL)

Sediment Dewatering Area

Dewatering depth =	0.5	m			
Sediment volume collected every 5 years=				665	m^3
Required Dewatering area =	1330	m^2			

Ensure this area is provided near the sediment pond and is accessible with machinery/access tracks etc.

Dewatering Provision for Maintenance

Bypass as described in drawing HCGS/SWS/3

Drawdown wetland to 27.85 m AHD with resilient gate valve shown in HCGS/SWS/2

Place pump in pump pit and pump water into wetland

Sediment Pond Flood Flow Velocity Check

See Addendum B.4

B. 4 Wetland Velocity Checks

Initial Wetland Flow Velocity Checks -South West (S4) Sed Pond							
Constructed Wetlands Design Manual , Part D: Design tools, resources and glossary (2015)							
Hydraulic analysis of flow velocities, Manual calculation							
Initial Velocity Checks							
Q_{100} =	12.9	m^3/s (RORB)					
Q_{10} =	6.7	m^3/s (RORB)					
Q_5 =	5.4	m^3/s (RORB)					
Q_{3mth} =	1.1	m^3/s (Rule of Thumb given 5 Year Flow)					
Wetland Normal Water Level (NWL) =				28.00	m AHD		
Wetland Top of Extended Detention (TED) =				28.35	m AHD		
Maximum Base level at wetland narrowest width =				27.00	m AHD		
1a	Peak 10 yr flow through sediment pond =				6.7	m^3/s (RORB)	
	Peak 100 yr flow through sediment pond =				12.9	m^3/s (RORB)	
1b	Bypass around Macrophyte zone =				0.0	m^3/s	
	Macrophyte zone inlet capacity =				0.0	m^3/s	
	Peak 3 month flow through macrophyte zone =				1.1	m^3/s	
	Peak 10 yr flow through macrophyte zone =				6.7	m^3/s	
	Peak 100 yr flow through macrophyte zone =				12.9	m^3/s	
Initial Sediment Pond Velocity Check							
2	RORB 10 yr WL =		29.6	m AHD	Assumes when 100 yr flow		
					comes in WL is at 10 yr level		
	At narrowest part of the sediment pond:						
3a	NWL width =		25	m			
3b	Width at 10 yr WL =		44.2	m			
4	10 yr WL - NWL =		1.60	m			
	Average width =		34.6	m			
	Cross section flow area =		55.36	m^2			
5	100 yr Flow velocity =		0.23	m/s < 0.5 m/s		OK	

Initial Wetland Flow Velocity Checks -South East (S5) Sed Pond

Constructed Wetlands Design Manual , Part D: Design tools, resources and glossary (2015)

Hydraulic analysis of flow velocities, Manual calculation

Initial Velocity Checks

Q_{100} =	8.5	m^3/s (RORB)
Q_{10} =	4.4	m^3/s (RORB)
Q_5 =	3.6	m^3/s (RORB)
Q_{3mth} =	0.7	m^3/s (Rule of Thumb given 5 Year Flow)

Wetland Normal Water Level (NWL) =	28.00	m AHD
Wetland Top of Extended Detention (TED) =	28.35	m AHD
Maximum Base level at wetland narrowest width =	27.00	m AHD

1a	Peak 10 yr flow through sediment pond =	4.4	m^3/s (RORB)
	Peak 100 yr flow through sediment pond =	8.5	m^3/s (RORB)
1b	Bypass around Macrophyte zone =	0.0	m^3/s
	Macrophyte zone inlet capacity =	0.0	m^3/s
	Peak 3 month flow through macrophyte zone =	0.7	m^3/s
	Peak 10 yr flow through macrophyte zone =	4.4	m^3/s
	Peak 100 yr flow through macrophyte zone =	8.5	m^3/s

Initial Sediment Pond Velocity Check

2	RORB 10 yr WL =	29.6	m AHD	Assumes when 100 yr flow comes in WL is at 10 yr level
	At narrowest part of the sediment pond:			
3a	NWL width =	20	m	
3b	Width at 10 yr WL =	39.2	m	
4	10 yr WL - NWL =	1.60	m	
	Average width =	29.6	m	
	Cross section flow area =	47.36	m^2	
5	100 yr Flow velocity =	0.18	m/s < 0.5 m/s	OK

Initial Wetland Flow Velocity Checks -South Wetland W2

Constructed Wetlands Design Manual , Part D: Design tools, resources and glossary (2015)

Hydraulic analysis of flow velocities, Manual calculation

Initial Velocity Checks

Q_{100} =	22.6	m^3/s (RORB)
Q_{10} =	11.5	m^3/s (RORB)
Q_5 =	9.3	m^3/s (RORB)
Q_{3mth} =	1.9	m^3/s (Rule of Thumb given 5 Year Flow)

Wetland Normal Water Level (NWL) =	28.00	m AHD
Wetland Top of Extended Detention (TED) =	28.35	m AHD
Maximum Base level at wetland narrowest width =	27.65	m AHD

1a	Peak 10 yr flow through sediment pond =	11.5	m^3/s (RORB)
	Peak 100 yr flow through sediment pond =	22.6	m^3/s (RORB)
1b	Bypass around Macrophyte zone =	0.0	m^3/s
	Macrophyte zone inlet capacity =	0.0	m^3/s
	Peak 3 month flow through macrophyte zone =	1.9	m^3/s
	Peak 10 yr flow through macrophyte zone =	11.5	m^3/s
	Peak 100 yr flow through macrophyte zone =	22.6	m^3/s

Initial Macrophyte zone Velocity Check

6	RORB 10 yr WL =	29.6	m AHD
	At narrowest part of the macrophyte zone:		
7a	NWL width =	50	m
7b	TED width =	54.2	m (very conservative - curls back on itself)
7c	Width at 10 yr WL =	69.2	m (very conservative - curls back on itself)
8a	TED - base level at narrowest width =	0.7	m
	Average width =	52.1	m
	Cross section flow area =	36.47	m^2
8b	10 yr WL - NWL =	1.6	m
	Average width =	61.7	m
	Cross section flow area =	98.72	m^2
9	3 month Flow velocity =	0.05	m/s OK
10	100 Year ARI Flow velocity =	0.23	m/s < 0.5 m/s OK

B. 5 South Wetland MUSIC Results

Results From File:	Hancocks_MUSIC_NV5_SV3_15Aug16.sqz			
Treatment Train Effectiveness at Outlet:				
Flow (ML/yr)	3,300	3,190	3.2	
Total Suspended Solids (kg/yr)	381,000	81,800	78.5	
Total Phosphorus (kg/yr)	967	379	60.8	
Total Nitrogen (kg/yr)	7,420	4,980	32.9	
Gross Pollutants (kg/yr)	62,800	-	100	
Polutant Generated from All Outside Catchments:				
Flow (ML/yr)	1,590	1,590	0	
Total Suspended Solids (kg/yr)	92,000	92,000	0	
Total Phosphorus (kg/yr)	337	337	0	
Total Nitrogen (kg/yr)	2,720	2,720	0	
Gross Pollutants (kg/yr)	4,440	4,440	0	
Polutant Generated from ALL Inside Catchments:				
Flow (ML/yr)	1,710			
Total Suspended Solids (kg/yr)	289,000			
Total Phosphorus (kg/yr)	630			
Total Nitrogen (kg/yr)	4,700			
Gross Pollutants (kg/yr)	58,360			
Polutant Treated inside Total Treament Train:				
Flow (ML/yr)	110			
Total Suspended Solids (kg/yr)	299,200			
Total Phosphorus (kg/yr)	588			
Total Nitrogen (kg/yr)	2,440			
Gross Pollutants (kg/yr)	62,800			
Treament Train Effectiveness:				
Flow (ML/yr)	6%			
Total Suspended Solids (kg/yr)	100%			
Total Phosphorus (kg/yr)	93%			
Total Nitrogen (kg/yr)	52%			
Gross Pollutants (kg/yr)	100%			

B. 6 Wetland Inundation Check

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 if this type. The check required to be undertaken is:

Check 1: Regular inundation check

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

Critical plant height is defined as the plant height relative to NWL (m AHD). It is assumed that the shortest allowable average plant heights are 1.5 meters in shallow marsh and 1.5 meters in deep marsh zones.

The analysis assumes the MUSIC model detailed in Section 6 with Narre Warren North rainfall data at 6 minute increments (1984 – 1993).

The MUSIC model incorporates internally calculated outflow relationships pertaining to:

- 72 hours' detention time in the ED zone, and
- Detention times calculated assuming a 1.2 metre weir width as this width (assuming a broad crested vertical upstream face) provides a similar outflow relationship to the outlet culverts for water levels above the ED zone.

The analysis was completed using the MUSIC Auditor – Wetland Analysis Tool as shown below. This analysis results in:

- Sea Club-rush being excluded from the specified plant list in shallow marsh zones,
- Common Spike-rush being excluded from the specified plant list in shallow marsh zones,
- Tall Spike-rush being excluded from the specified plant list in the deep marsh zones, and
- Water Ribbons being excluded from the specified plant list in both the shallow or deep marsh zones.

Wetland Analysis Tool

Welcome to the Wetland Analysis Tool for checking compliance with the Melbourne Water Constructed Wetland Manual. The tool creates an inundation frequency curve and helps the user to assess wetland depths relative to plant heights and the wetland residence time.

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

Shallow Planting Depth 0.15 m
Deep Planting Depth 0.35 m

Please enter the permanent pool volume.

Permanent Pool Volume 11897 m

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

Choose file SV3.csv
FILE IS UPLOADED

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

Clear Selection

Name	Average plant height (m)	Shallow marsh plants	Deep marsh plants	Suitability
Marsh Club-rush *Bolboschoenus medianus*	1.5	☑	☐	Shallow Only
Jointed Club-rush *Baumea articulata*	1.8	☑	☑	Shallow and Deep
Tall Club-rush *Bolboschoenus fluviatilis*	1.8	☑	☑	
Leafy Twig-rush *Cladium procerum*	2	☑	☑	
River Club-rush *Schoenoplectus tabernaemontani*	1.8	☑	☑	
Common reed *Phragmites australis*	2.5		☑	Deep Only
Sea Club-rush *Bolboschoenus caldwellii*	1	☐		Unsuitable
Common Spike-rush *Eleocharis acuta*	0.5	☐		
Tall Spike-rush *Eleocharis sphacelata*	1.5		☐	
Water Ribbons *Triglochin procerum*	1	☐	☐	

+ Add user defined plant

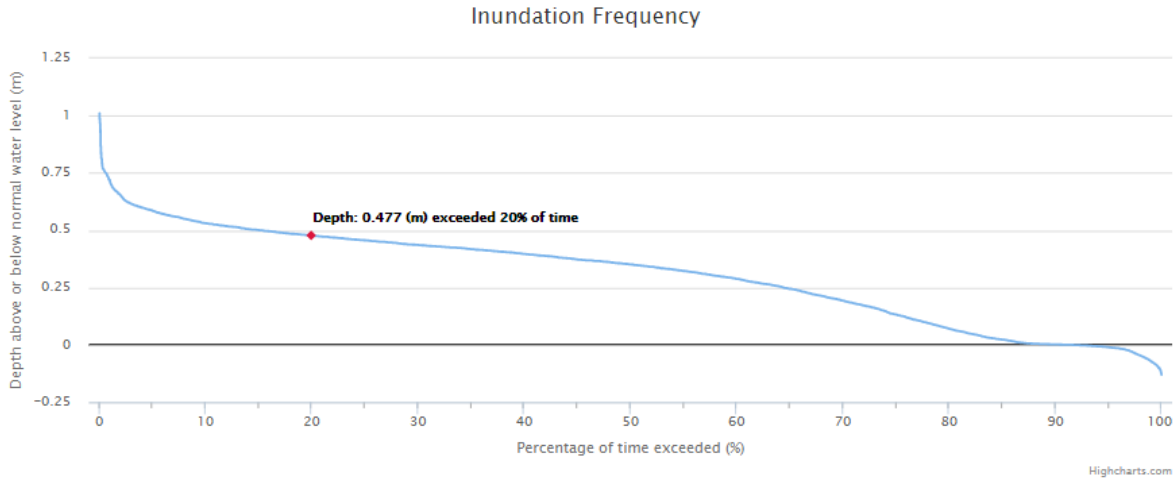
Report

File: SV3.csv
Shallow marsh zone meets deemed to comply criteria

Deep marsh zone meets deemed to comply criteria

Water level exceeded for 20% of time: 0.477 m

90th Percentile Residence Time: 4 days



SWS Check:

Wetland Plant Health Requirements Check - Hancocks Gully South					
Critical Plant Depth = Plant Height relative to NWL (m)			10 Year NWN data 1984 - 1993		
Extended Detention W2 = 0.35 m					
Shortest Allowable Average Plant Heights in Wetland 2 =			1.5	m	Shallow Marsh
			1.8	m	Deep Marsh
Plants to be excluded from plant list for shallow marsh and/or deep marsh					
Note:	***Common Spike Rush and Sea Club Rush not to be specified in plant list in shallow marsh zones***				
	Water Ribbons not to be specified in plant list for W2 in both marsh zones				
	Tall Spike Rush not to be specified in deep marsh zones				
Wetland specifications				Check 1 - Critical Plant Height Relative to NWL	
	Max Permanent Pool Depth	Average Height of Plant in wetland plant list given any exclusions	50% Average Plant Height	(Average plant height/2)-PP	
Shallow Marsh	0.15	1.5	0.75	0.60	
Deep Marsh	0.35	1.8	0.90	0.55	
CHECK 1: Regular inundation check					
Water level 80% of the time (or more) < 50% Critical Plant Height (CHECK 1)					
W2 Inundation Frequency Analysis		Water level (relative to NWL) expected 80% of the time or more =			
				477.0	mm
Percentile	**WL Relative to NWL - (mm)**				
0.5%	-87				
1.0%	-72	**Water level (relative to NWL) expected 80% of the time or more < CHECK (1) Shallow Marsh?**			
5.0%	-10				**YES**
15.0%	23				**Check1 met**
20.0%	70				
50.0%	351				
70.0%	436				
80.0%	477	**Water level (relative to NWL) expected 80% of the time or more < CHECK (1) Deep Marsh?**			
97.0%	615				**YES**
99.5%	755				**Check1 met**

B. 7 Liner and Topsoil

The soil profile in the area is expected to be clayey in nature. The development of wetlands proposed in the adjacent Cardinia Industrial DSS were formulated given in-depth hydro geological study performed by Sinclair Knight Merz “Cardinia Road Precinct Development, Installation of Shallow Groundwater Bores, Final Report, 20 August 2008”.

One of the primary conclusions of the 2008 study was that the insitu clayey soil is ideal for formation of wetland systems as the interaction between groundwater and surface water is extremely slow. As such the surface water systems (i.e. wetlands) can essentially be considered separate from the groundwater systems. SKM concluded that the NWL’s etc proposed in the Cardinia Industrial DSS can be achieved and should not adversely affect surface water/groundwater interaction in the area. Given this finding clay lining was not required as part of the concept design proposals for the Cardinia Industrial DSS systems.

As such, it is assumed at this stage that clay lining will not be required for the Hancocks Gully South DSS wetland. Compaction of the existing clayey soil and placement of topsoil as required should be specified at the detailed design stage of the project. Also the detailed design stage of the project should include compressive soil tests of the subject site to confirm the above assumption.

B. 8 Constructed Wetlands Design Manual, Part A.2: Checklist

General		
Criteria	*Description*	*Reference*
GN1	The treatment and flow regime performance of the wetland must be modelled in MUSIC, or similar conceptual modelling software as approved by Melbourne Water.	Met with MUSIC modelling (Section 6)
GN2	The meteorological data in the conceptual modelling data or software (i.e. MUSIC) must be:	MUSIC modelling (Section 6) meets criteria
	• Based on at least 10 years of historical records	
	• Recorded at six minutes' intervals	
	• Sourced from a pluviographic station as close as possible to the wetland site	
	• Have a mean annual rainfall depth within 10% of the long term rainfall depth at the rainfall station closest to the wetland site	
GN3	The system configuration shown on the design plans must be consistent with the conceptual modelling parameters (e.g. MUSIC) (including the stage/discharge relationship) and sediment pond calculator/calculations.	Shown in Section 6 and Addendum B. 1
GN4	Peak design flows must be estimated in accordance with methods in Australian Rainfall and Runoff.	Met with RORB modelling (Section 4)

Maintenance Provisions		
Criteria	*Description*	*Reference*
MN1	Sediment ponds must be able to be drained whilst maintaining the macrophyte zone water level at normal water level.	Design based on MWC standard drawing WG010 (B)
MN2	All parts of the base of a sediment pond must be accessible:	Shown in: HCGS/SWS/ 2
	• Within seven metres of a designated hard stand area for excavation vehicles ("edge cleaned") OR	
	• Via a maintenance access ramp into the base of the sediment pond	
MN5	Maintenance access ramps are required on all sediment ponds that cannot be 'edge cleaned'. The maintenance access ramp into a sediment pond must:	Shown in: HCGS/SWS/ 2
	• Extend from the base of the sediment pond to at least 0.5 metres above TED,	
	• Be at least 4 metres wide,	
	• Be no steeper than 1:5	
	• Be capable of supporting a 20 tonne excavator	
	• Constructed of either:	
	- 200 mm deep layer of cement treated crushed rock (6%), or	
	- 200 mm compacted FCR	
	• Have a barrier to prevent unauthorised vehicle access (e.g. gate, bollard and/or fence).	
MN6	A maintenance access track must be provided to the sediment pond maintenance access ramp and to enable maintenance vehicles to safely access and exit the site. The maintenance access track must:	Shown in: HCGS/SWS/ 2
	• Be at least 4 metres wide	
	• Comprise of compacted FCR minimum 200 mm depth	
	• Be reinforced to take a 20 tonne vehicle	
	• At the road edge, have an industrial crossover to Council standard and rolled kerb adjoining it.	
MN7	A hardstand area with a minimum turning circle appropriate to the types of maintenance vehicles to be used must be provided adjacent to the sediment pond maintenance access ramp to enable maintenance vehicles to safely reverse and exit the sediment loading area. (Designers should seek advice from Melbourne Water on the types of maintenance vehicles that will be used.)	Shown in: HCGS/SWS/ 2
	Note: The turning circle must be in accordance with the Austroads Design Vehicles and Turning Path Templates Guide: (http://www.austroads.com.au/images/stories/ap-g34-13.pdf)	
MN9	Dedicated sediment dewatering areas must be provided and:	Shown in: HCGS/SWS/ 2
	• Be accessible from the maintenance ramp,	
	• Have a length to width ratio no narrower than 10:1,	
	• Be able to contain all sediment removed from the sediment accumulation volume spread out at 500 mm depth	
	• Be located above the peak 10 year ARI water level and within 25 metres of each sediment pond or as close as possible,	
	• Be located at least 15 metres from residential areas and public access areas (like pathways, roads, playgrounds, sports fields etc), and consider potential odour and visual issues for local residents	
	• Address public safety and potential impacts on public access to open space areas,	
	• Be free from above ground obstructions (e.g. light poles) and be an area that Melbourne Water has legal or approved access to for the purpose of dewatering sediment.	
MN10	The wetland must be configured to enable maintenance vehicles to drive around at least 50% of the wetland perimeter. Vehicular access must be provided as close as possible to wetland structures that may catch debris (e.g. provide access to the closest bank where structures are within the water body).	To be designed when landscape architect engaged. Ed control pit in embankment crest for easy access

Sediment Pond		
Criteria	*Description*	*Reference*
SP1	Sediment ponds must be located offline of waterways but online to the pipe or lined channel they are treating water from. Refer to Part A3 of the Manual for guidance on offline configurations.	Met assuming DSS pipeline alignments do not change. See HCGS/SWS/ 2
SP2	Sediment ponds must be located at each point stormwater enters the “wetland system” unless:	Met assuming DSS pipeline alignments do not change. See HCGS/SWS/ 2
	• The catchment of the incoming stormwater is < 5% of the total wetland catchment OR	
	• The incoming stormwater has already passed through a bioretention system or wetland immediately upstream	
SP3	Sediment ponds must be sized to:	Calculations Shown in Section 5 and Addendum B. 3
	• Capture 95% of coarse particles ≥ 125 µm diameter for the peak three month ARI AND	
	• Provide adequate sediment storage volume to store three to five years sediment. The top of the sediment accumulation zone must be assumed to be 500 mm below NWL AND	
	• Ensure that velocity through the sediment pond during the peak 100 year ARI event is ≤ 0.5 m/s. (The flow area must be assumed to be the EDD multiplied by the narrowest width of the sediment pond, at NWL, between the inlet and overflow outlet)	
	Sediment ponds must be ≤ 120% of the size needed to meet the limiting of the above three criteria. Compliance with the above criteria must be demonstrated using the methods described in WSUD Engineering Procedures: Stormwater (Melbourne Water, 2005). Alternatively, the velocity criteria can be checked using a hydraulic model such as HEC-RAS. Refer to Part D of the Manual for guidance on undertaking velocity checks).	
SP4	The sediment pond EDD must be ≤ 350 mm.	Calculations Shown in Section 5 and Addendum B. 3

Macrophyte Zone		
Criteria	*Description*	*Reference*
MZ1	At least 80% of the area of the macrophyte zone at NWL must be ≤ 350 mm deep to support shallow and deep marsh vegetation. The wetland bathymetry should provide approximately equal amounts of shallow marsh (≤ 150 mm deep) and deep marsh (150 mm to 350 mm deep).	Calculation shown in Section 3.2. Also refer to HCGS/SWS/ 2
MZ2	The macrophyte zone EDD must be ≤ 350 mm.	Shown in Section 3.2
MZ3	Macrophyte zones must be located offline from all waterways and drains (i.e. there must be a bypass route around the macrophyte zone).	FAIL Impossible due to nature of Hancocks Gully and PSP boundary allocated
MZ4	The length of the macrophyte zone must be ≥ four times the average width of the macrophyte zone.	Shown in: HCGS/SWS/ 2 Length = 650 m Width = 50 m
MZ5	Major inlets to the macrophyte zone (i.e. those draining > 10% of the catchment to be treated) must be located within the first 20% of the macrophyte zone.	Shown in: HCGS/SWS/ 1
MZ6	The macrophyte zone outlet must be located at the opposite end of the macrophyte zone to the inlet(s).	Shown in: HCGS/SWS/ 2
MZ7	The macrophyte zone must have a sequence and mix of submerged, shallow and deep marsh zones arranged in a banded manner perpendicular to the direction of flow.	Shown in: HCGS/SWS/ 2
MZ8	Inlet and outlet pools must be ≤ 1.5 m depth.	Shown in: HCGS/SWS/ 2
MZ9	Intermediate pools (between the inlet and outlet pool) must be ≤ 1.2 m deep.	Shown in: HCGS/SWS/ 2
MZ10	Velocities in the macrophyte zone must be: • less than 0.5 m/s for the peak 100 year ARI flow • less than 0.05 m/s for the peak three month ARI Compliance with the above criteria must be demonstrated using the methods described in WSUD Engineering Procedures: Stormwater (Melbourne Water, 2005) or using a hydraulic model such as HEC-RAS or TUFLOW. Refer to Part D of the Manual for guidance on undertaking velocity checks.	Calculations shown in Addendum B. 4
MZ11	The macrophyte zone must provide a 90th percentile residence time of 72 hours (assuming plug flow between inlet and outlet through the EDD and 50% of the permanent pool volume). Refer to the Melbourne Water online tool and Part D of the Manual for guidance on determining residence time.	MUSIC Auditor Results = 4 days (Addendum B. 6)
MZ12	A minimum grade of 1:150 must be provided between marsh zones (longitudinally through the macrophyte zone) to enable the wetland to freely drain. Intermediate pools will generally be needed to transition between marsh zones.	Shown in: HCGS/SWS/ 2

Bypass		
Criteria	*Description*	*Reference*
BY1	The bypass route must be sized to convey the maximum overflow from the sediment pond that will occur during the peak 100 year ARI event. A bypass is still required for a wetland located within a retarding basin. Where a sediment pond is within a retarding basin, the bypass must convey at least the peak one year ARI flow.	Fail No bypass due to nature of Hancocks Gully and PSP boundary allocated
	Note: Refer to Melbourne Water Waterways Manual for channel design specifications when designing bypass routes.	

Inlets and Outlets		
Criteria	*Description*	*Reference*
IO4	The connection between the sediment pond and macrophyte zone must be sized such that:	Fail Sediment ponds connected to W2 due to design being based on MWC drawing WG010 (B)
	• All flows ≤ the peak three month ARI event are transferred into the macrophyte zone when the EDD in the macrophyte zone is at NWL, AND	
	• 60% of the peak 1 year ARI flow overflows from the sediment pond into the bypass channel/pipe when the water level in the macrophyte zone is at TED (and not enter the macrophyte zone), AND	
	• The velocity through the macrophyte zone is ≤ 0.5 m/s during the peak 100 year ARI event:	
	i. Assuming the macrophyte zone is at TED if the wetland is not within a retarding basin or flood plain	
	ii. Assuming the water level is at the peak 10 year ARI water level if the wetland is within a retarding basin or flood plain	
IO6	The macrophyte zone controlled outlet must be configured so that:	All achievable with configuration shown in HCGS/SWS/ 3
	• The NWL can be drawn down by up to 150 mm during plant establishment and maintenance.	
	• The NWL can be permanently adjusted up or down by 100 mm to respond to changes in wetland hydrology due to potential future climate conditions.	
	• The stage/discharge rate can be adjusted if required to achieve suitable residence times and/or inundation patterns	
IO7	Balance pipes must be placed between all open water zones (inlet, intermediate and outlet pools) to enable water levels to be drawn down for maintenance or water level management purposes. Balance pipes must comprise of minimum 225 mm (e.g. sewer class PVC), the invert level of the pipes must be at no more than 100 mm above the base of the macrophyte zone and fitted with a truncated pit to minimise the risk of clogging (refer to Melbourne Water Standard Drawing for truncated pit details).	Shown in: HCGS/SWS/ 4

Vegetation		
Criteria	*Description*	*Reference*
VG1	The macrophyte zone must contain a minimum 80% cover of emergent macrophyte calculated at NWL comprising of shallow and deep marsh zones. Open water areas (maximum 20% of the wetland area calculated at NWL) must include submerged vegetation.	Shown in Section 3.2 and Addendum B. 6
VG2	Any open water areas in excess of 20% of the macrophyte zone area (at NWL) must be located as a separate water body. These separate water bodies are not considered by Melbourne Water to be constructed wetlands for the purpose of treating stormwater, and are therefore beyond the scope of the Manual. For further information, refer to Part A3 for open water, landscape design and amenity design considerations and the Planning and Building website for ownership and maintenance responsibilities. Conceptual models of wetlands and other parts of the treatment train (e.g. MUSIC) must assume there is no reduction in pollutant loads within these separate waterbodies.	No open water areas > 20%. See Section 3.2
VG3	Ephemeral batters (NWL to 200 mm above NWL) of the macrophyte zone and sediment pond must be densely planted with plants suited to intermittent wetting. 80% of the plants used in the ephemeral batters must be in accordance with the species shown in the Manual.	Zones Specified in HCGS/SWS/ 2 and plant list specified in Addendum B. 6
VG5	The shallow marsh (NWL to 150 mm below NWL) of the macrophyte zone and sediment pond must be densely planted. 90% of the plants used in the shallow marsh must be in accordance with the species and densities shown in the Manual. A minimum of three species must be specified for the shallow marsh zone.	Zones Specified in HCGS/SWS/ 2 and plant list specified in Addendum B. 6
VG6	The deep marsh (150 to 350 mm below NWL) of the macrophyte zone must be densely planted. 90% of the plants used in the deep marsh must be in accordance with the species and densities shown in the Manual. A minimum of three species must be specified for the deep marsh zone.	Zones Specified in HCGS/SWS/ 2 and plant list specified in Addendum B. 6
VG7	The submerged marsh (350 to 700 mm below NWL) of the macrophyte zone must be densely planted. 90% of the plants used in the submerged marsh must be in accordance with the species and densities shown in the Manual.	Zones Specified in HCGS/SWS/ 2 and plant list specified in Addendum B. 6
VG10	The effective water depth (permanent pool depth plus EDD) must not exceed half of the average plant height for more than 20% of the time. This must be demonstrated using inundation frequency analysis assuming the plants heights are in accordance with those shown in the Manual. Refer to online tool and Part D of the Manual for guidance on the inundation frequency analysis.	Zones Specified in HCGS/SWS/ 2 and plant list specified in Addendum B. 6
VG11	Where stormwater is harvested from the permanent pool of a wetland, the extraction must not occur if the water level is more than 100 mm below NWL. Note: a diversion licence is required to harvest water from Melbourne Water assets (see Melbourne Water's Stormwater Harvesting Guidelines for more information).	No harvesting assumed but needs to be met if harvesting is to occur as per 2016 DCE report

Liner and Topsoil		
Criteria	*Description*	*Reference*
LN1	The exfiltration rate from the base and the sides of the wetland must be accurately represented in the conceptual modelling software analysis (e.g. MUSIC). Wetlands with a permanent pool generally have a compacted clay liner made from site soils and/or imported material where site soils are unsuitable. Where no liner is proposed, in-situ geotechnical testing (at the depth of the wetland base) must be undertaken and used to justify the selected exfiltration rate used in modelling.	Discussion provided in Addendum B. 7
LN3	At least 200 mm topsoil must be provided in all areas of the macrophyte zone; and in sediment ponds to 350 mm below NWL.	See note in HCGS/SWS/ 2

Landscape Design Structures		
Criteria	*Description*	*Reference*
LDS2	All boardwalks, bridges and formal pedestrian paths, must be at or above the peak 10 year ARI water level. Refer to Melbourne Water's Shared Pathway Guidelines and Jetties Guidelines for more information.	To be designed when landscape architect engaged
LDS3	Boardwalks or viewing platforms are not permitted over sediment ponds.	To be designed when landscape architect engaged

Edge Treatment		
Criteria	*Description*	*SWS Location*
ET1	The edge of any deep open water should not be hidden or obscured by embankments or terrestrial planting unless measures are taken to preclude access. Public access to structures, the top of weirs, orifice pits and outlet structures must be restricted by appropriate safety fences and other barriers. Permanent fencing is required adjacent to potentially unsafe structures (i.e. deep water zones, steep drops, top of weirs, outlet structures etc).	See note in HCGS/SWS/ 2
ET2	All wetland edges must have: • Vegetated approach batters no steeper than 1:5, a 2.8 metre wide vegetated safety bench at 1:8 between NWL and 350 mm below NWL and a maximum 1:3 slope beyond 350 mm below NWL OR • Batters no steeper than 1:4 between TED and 350 mm below NWL with dense impenetrable planting that is a minimum of 2.8 metres wide and 1.2 metres high.	See note in HCGS/SWS/ 2
ET4	A minimum offset of 15 metres must be provided from the edge of the water at NWL to any allotment or road reserve (not including shared pathways). A safety design audit is required for any proposal that does not achieve this condition. Refer to Part A3 of the Manual for design consideration and guidance on safety in design.	Met as shown in HCGS/SWS/ 2

Addendum C – Hancocks Gully Vegetated Waterways V1 & V2 Design

The following section indicates the design and calculations for Hancocks Gully Vegetated Waterway Design.

Addendum C Contents:

C. 1 Hancocks Gully North Vegetated Waterway (V1) Design 100
C. 2 Hancocks Gully South Vegetated Waterway (V2) Design 102

C. 1 Hancocks Gully North Vegetated Waterway (V1) Design

The Hancocks Gully North Vegetated Waterway (V1) had been design using HecRas.

Drawing HCGN/SWS/7 shows the cross section locations entered into the model and Figure 10 (Section 7) provides a general representation of the designed channel.

A uniform Mannings n of 0.15 has been set across all cross sections as described in Section 7.

A flow of 15.0 m^3/s (Table 9, Section 4.2.4) was entered at the most upstream cross section. The known 100 Year ARI water level of 40.50 m AHD (Table 9, Section 4.2.4) was set as the downstream boundary condition and a normal water depth at a slope of 1/100 was set as the upstream boundary condition. A mixed flow regime was simulated with 5 m interpolated cross sections.

The profile summary table is shown below in Table 12 and Figure 11 details the northern waterway longitudinal section.

Table 12 Northern Waterway V1 Profile Summary

River Sta	Q Total (m^3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
523	15	45.0	46.61	45.95	46.63	0.010	0.67	24.31	32.13	0.21
422	15	44.0	45.61		45.63	0.010	0.67	24.31	32.13	0.21
321	15	43.0	44.60		44.63	0.010	0.68	24.22	32.08	0.21
221	15	42.0	43.60		43.62	0.010	0.68	24.16	32.04	0.21
121	15	41.0	42.46		42.49	0.017	0.81	19.85	29.22	0.27
77	15	40.5	41.48		41.540	0.051	1.11	13.47	20.94	0.44
0	15	39.0	40.50	39.15	40.500	0.000	0.10	169.08	173.34	0.03

File: Hanc_Nor_V5_15Aug16.prj

The modelling shows that the 100 Year ARI flow can be contained to the channel as detailed in drawing HCGN/SWS/ 7.

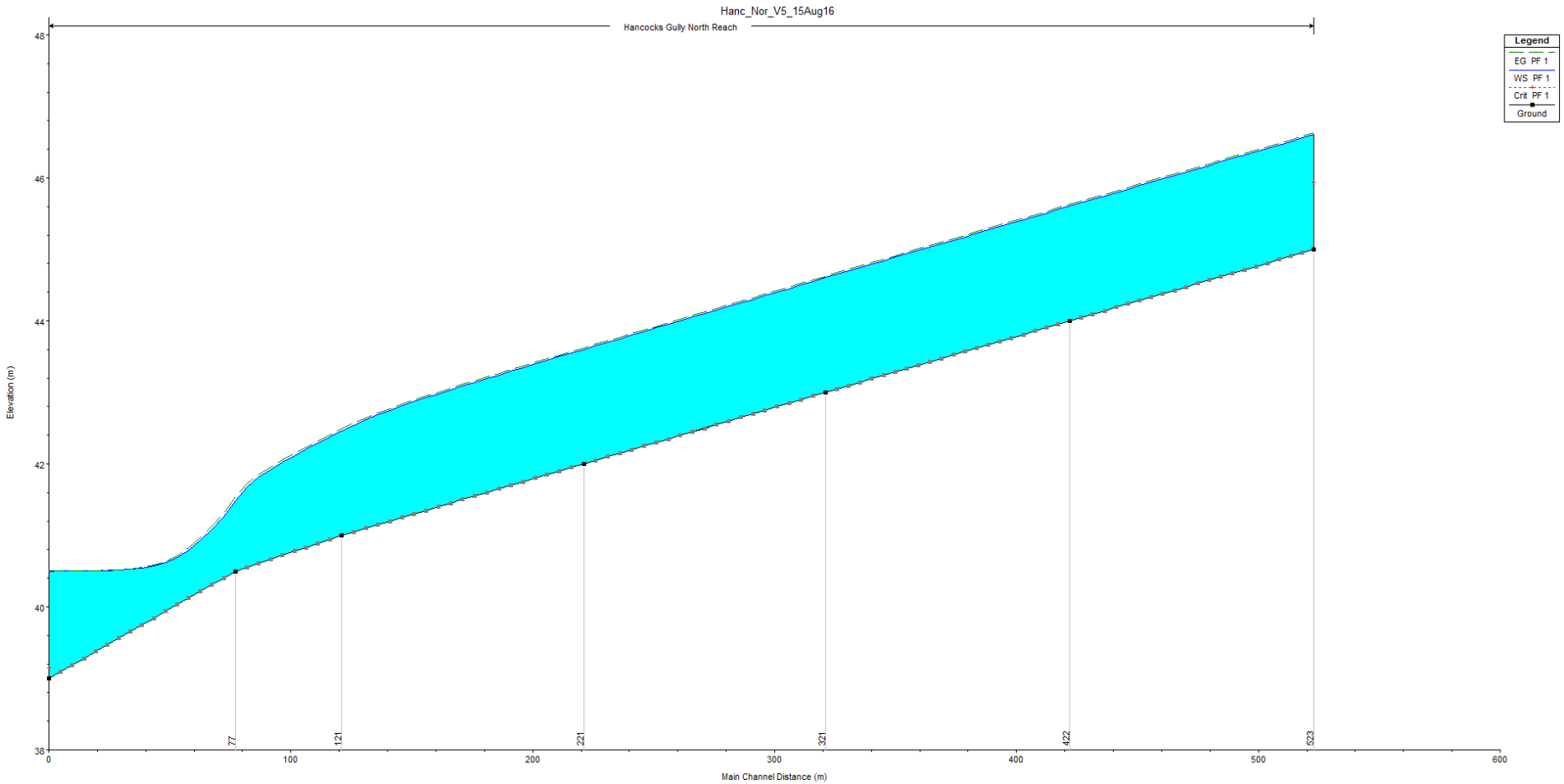


Figure 11 Northern Waterway V1 Longitudinal Profile

C. 2 Hancocks Gully South Vegetated Waterway (V2) Design

The Hancocks Gully South Vegetated Waterway (V2) had been design using HecRas.

Drawing HCGS/SWS/ 5 shows the cross section locations entered into the model and Figure 10 (Section 7) provides a general representation of the designed channel.

A uniform Mannings n of 0.15 has been set across all cross sections as described in Section 7.

A flow of 15.2 m^3/s (Table 10, Section 4.2.4) was entered at the most upstream cross section. The known 100 Year ARI water level of 29.75 m AHD (Table 10, Section 4.2.4) was set as the downstream boundary condition and a normal water depth at a slope of 1/90 was set as the upstream boundary condition. A mixed flow regime was simulated with 5 m interpolated cross sections.

The profile summary table is shown below in Table 13 and Figure 12 details the northern waterway longitudinal section.

Table 13 Southern Waterway V2 Profile Summary

River Sta	Q Total (m^3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m^2)	Top Width (m)	Froude # Chl
743	15.2	35.0	36.58	35.95	36.61	0.011	0.69	24.42	37.8	0.22
651	15.2	34.0	35.56		35.58	0.012	0.72	22.78	31.2	0.23
569	15.2	33.0	34.60		34.62	0.011	0.69	23.99	31.9	0.22
475	15.2	32.0	33.69		33.71	0.007	0.62	27.17	33.9	0.19
341	15.2	31.0	32.63		32.65	0.009	0.67	24.92	32.5	0.21
236	15.2	30.0	31.66		31.68	0.008	0.64	26.07	33.2	0.20
117	15.2	29.0	30.65		30.67	0.009	0.65	25.73	33.0	0.20
53	15.2	28.5	29.79		29.84	0.039	1.05	15.04	25.7	0.40
0	15.2	28.0	29.75	28.09	29.75	0.000	0.04	396.86	291.8	0.01

File: Hanc_Sth_V3_23Aug16.prj

The modelling shows that the 100 Year ARI flow can be contained to the channel as detailed in drawing HCGS/SWS/ 5.

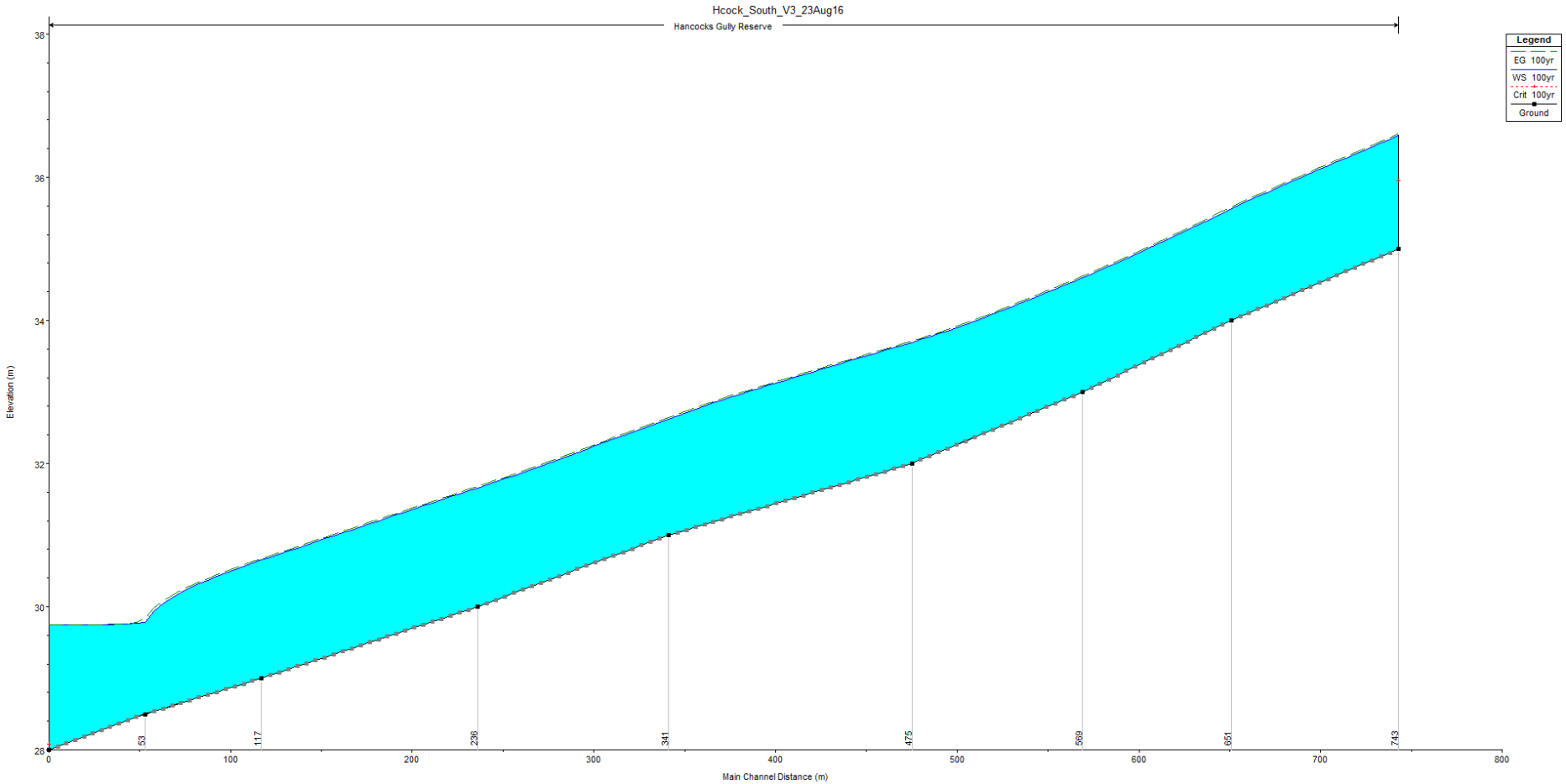


Figure 12 Southern Waterway V2 Longitudinal Profile

Addendum D – RORB Catchment Files

D. 1 Pre-development .cat file

```
1603_Hancocks_Gully_DSS_PREDEV
C RORB MODEL DELVELOPED BY MICHAEL MAG
C Project Engineer, Stormy Water Solutions
C DATE: 10/5/16
C AREA TOTAL = 8.484 km^2
C kc= from DVA formula Kc = 4.959 = 1.53*(AREA)^0.55
C m=0.8
C RoC 100yr = 0.6
C RoC 50yr = 0.55
C RoC 20yr = 0.5
C RoC 10yr = 0.4
C RoC 5yr = 0.30
C RoC 2yr = 0.25
C RoC 1yr = 0.2
C Initial loss = 10mm
C IFD Data location: Pakenham
0
1,1,1.051,-99          1     A
2,1,0.871,-99          2     B
2,1,0.842,-99          3     C
2,1,0.456,-99          4     D
3
1,1,0.540,-99          5     E
3
1,1,0.434,-99          6     F
4
5,1,0.436,-99          7
2,1,0.553,-99          8     G
4
3
1,1,0.642,-99          9     H
4
7
FLOW FROM NORTH
5,1,0.191,-99          10
3
1,1,0.308,-99          11    I
3
1,1,0.253,-99          12    J
4
4
5,1,0.355,-99          13
3
1,1,0.240,-99          14    K
4
5,1,0.351,-99          15
3
1,1,0.329,-99          16    L
3
1,1,0.524,-99          17    M
2,1,0.549,-99          18    N
4
4
2,1,0.207,-99          19    O
3

1,1,0.482,-99          20    P
3
1,1,0.460,-99          21    Q
4
4
7
NORTH RB
5,3,0.081,1.23,-99     22
3
1,3,0.510,1.27,-99     23    R
3
1,1,0.572,-99          24    S
2,3,0.357,0.56,-99     25    T
4
5,1,0.306,-99          26
2,1,0.509,-99          27    U
3
1,1,0.449,-99          28    V
4
2,1,0.224,-99          29    W
3
1,1,0.315,-99          30    X
2,1,0.400,-99          31    Y
3
1,1,0.995,-99          32    Z
4
4
3
1,1,0.702,-99          33    AA
3
1,1,0.429,-99          34    AB
4
2,1,0.403,-99          35    AC
2,1,0.301,-99          36    AD
4
7
SOUTH RB
0
C SUB-CATCHMENT AREAS (KM^2)
0.6807,0.5836,0.9908,0.6388,0.4958,
0.5595,0.4079,0.3389,0.2009,0.1549,
0.1908,0.1621,0.3763,0.2426,0.1266,
0.1531,0.1110,0.0768,0.2849,0.0547,
0.1515,0.1548,0.1003,0.2045,0.1633,
0.1214,0.1820,0.2100,0.1626,0.2015,-99
C IMPERVIOUS FRACTION
1,0.05,0.05,0.05,0.05,0.05,
0.05,0.05,0.05,0.05,0.05,
0.05,0.05,0.05,0.05,0.05,
0.05,0.05,0.70,0.18,0.70,
0.05,0.05,0.05,0.05,0.05,
0.05,0.05,0.05,0.05,0.05,-99
```

D. 2 Post-development .cat file

```
1603_Hancocks_Gul_DSS_POSTDEV_V5N_V3S
C RORB MODEL DELVELOPED BY MICHAEL MAG
C Project Engineer, Stormy Water Solutions
C DATE: 15/8/16
C AREA TOTAL = 8.472 km^2
C kc= from DVA formula Kc = 4.955 = 1.53*(AREA)^0.55
C m=0.8
C RoC 100yr = 0.6
C RoC 50yr = 0.55
C RoC 20yr = 0.5
C RoC 10yr = 0.4
C RoC 5yr = 0.30
C RoC 2yr = 0.25
C RoC 1yr = 0.2
C Initial loss = 10mm
C IFD Data location: Pakenham
0
1,1,1.051,-99          1     A
2,1,0.871,-99          2     B
2,1,0.842,-99          3     C
2,1,0.456,-99          4     D
3
1,1,0.540,-99          5     E
3
1,1,0.434,-99          6     F
4
5,1,0.436,-99          7
2,1,0.553,-99          8     G
4
3
1,1,0.642,-99          9     H
4
5,1,0.144,-99          10
2,1,0.110,-99          11    I
3
1,1,0.551,-99          12    J
3
1,1,0.436,-99          13    K
4
4
7
INTO PSP BOUNDARY
5,1,0.156,-99          14
2,1,0.245,-99          15    L
2,1,0.279,-99          16    M
7
VEG WATERWAY NORTH
3
1,3,0.139,12.27,-99    17    N
2,3,0.095,12.05,-99    18    O
2,3,0.158,9.17,-99     19    P
2,3,0.244,3.08,-99     20    Q
3
1,3,0.124,10.91,-99    21    R
2,3,0.096,11.99,-99    22    S
2,3,0.140,8.60,-99     23    T
2,3,0.118,8.08,-99     24    U
4
2,3,0.136,1.11,-99     25    V
3
1,3,0.076,13.76,-99    26    W
2,3,0.121,11.96,-99    27    X
2,3,0.115,8.26,-99     28    Y
2,3,0.120,4.99,-99     29    Z
2,3,0.141,4.62,-99     30    AA
3
1,3,0.160,3.76,-99     31    AB
4
3
1,3,0.137,6.95,-99     32    AC
2,3,0.230,2.83,-99     33    AD
2,3,0.149,2.01,-99     34    AE
2,3,0.094,1.06,-99     35    AF
4
5,3,0.124,0.40,-99     36
4
5,1,0.127,-99          37
7
NW SED
4
3
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2,3,0.334,1.95,-99     39    AH
3
1,3,0.179,1.12,-99     40    AI
4
2,3,0.237,1.68,-99     41    AJ
3
1,3,0.333,1.65,-99     42    AK
3
1,3,0.165,0.91,-99     43    AL
4
5,3,0.323,1.08,-99     44
4
3
1,3,0.184,1.09,-99     45    AM
4
3
1,3,0.200,0.75,-99     46    AN
2,3,0.232,0.86,-99     47    AO
4
5,1,0.111,-99          48
7
NE SED
4
2,4,0.146,-99          49    AP
16
NORTH RB
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0,0.00,13252,0.05,19680,1.52,24736,3.21,
29792,5.28,34848,7.67,39904,10.34,44960,11.98,
50681,12.57,56402,13.12,62123,13.66,67844,14.17,
73566,14.67,79744,17.67,92100,28.73,104456,44.27,-99
1,16
39.00,0,39.35,13252,39.50,19680,39.60,24736,
39.70,29792,39.80,34848,39.90,39904,40.00,44960,
40.10,50681,40.20,56402,40.30,62123,40.40,67844,
40.50,73566,40.60,79744,40.80,92100,41.00,104456,-99
5,3,0.081,0.12,-99     50
3
1,3,0.510,1.37,-99     51    AQ
3
1,2,0.572,2.53,-99     52    AR
2,3,0.357,0.56,-99     53    AS
4
4
7
SOUTH PRINCESS HWY
5,1,0.185,-99          54
2,1,0.274,-99          55    AT
2,1,0.232,-99          56    AU
2,1,0.276,-99          57    AV
7
VEG WATERWAY SOUTH
3
1,3,0.191,9.17,-99     58    AW
2,3,0.147,6.46,-99     59    AX
2,3,0.169,1.48,-99     60    AY
2,3,0.182,1.10,-99     61    AZ
2,3,0.115,0.65,-99     62    BA
3
1,3,0.256,5.66,-99     63    BB
3
1,3,0.233,5.58,-99     64    BC
4
5,3,0.159,1.73,-99     65
```

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4
5,3,0.175,1.00,-99          66
2,3,0.210,0.71,-99          67      BD
3
1,3,0.337,3.42,-99          68      BE
3
1,3,0.192,1.04,-99          69      BF
4
5,3,0.213,0.24,-99          70
3
1,3,0.188,2.12,-99          71      BG
4
4
5,1,0.123,-99               72
7
SW SED
4
3
1,3,0.437,1.26,-99          73      BH
3
1,3,0.148,1.02,-99          74      BI
4
5,3,0.340,1.03,-99          75
3
1,3,0.515,0.78,-99          76      BJ
4
3
1,3,0.268,0.75,-99          77      BK
4
3
1,3,0.263,1.14,-99          78      BL
2,3,0.136,0.74,-99          79      BM
4
5,3,0.228,0.66,-99          80
3
1,3,0.428,0.70,-99          81      BN
3
1,3,0.175,0.71,-99          82      BO
4
4
5,3,0.058,0.86,-99          83
3
1,2,0.597,2.68,-99          84      BP
2,3,0.367,0.14,-99          85      BQ
4
5,1,0.147,-99               86
7
SE SED
4
2,4,0.163,-99               87      BR
16
SOUTH RB
1,0,14
0,0.00,16942,0.06,33923,0.38,51871,0.69,
61728,1.06,68593,1.24,75458,1.40,82324,3.43,
89189,7.03,97419,11.62,105648,17.04,113878,23.15,
122108,29.87,130338,37.17,-99
1,14
28.0,0,28.4,16942,28.7,33923,29.0,51871,
29.1,61728,29.2,68593,29.3,75458,29.4,82324,
29.5,89189,29.6,97419,29.7,105648,29.8,113878,
29.9,130338,30.0,130338,-99
0
C SUB-CATCHMENT AREAS (KM^2)
0.6807,0.5836,0.9908,0.6388,0.4958,
0.5595,0.4079,0.3389,0.1183,0.0848,
0.0825,0.0223,0.0143,0.0280,0.0302,
0.0256,0.0510,0.0197,0.0215,0.0133,
0.0157,0.0246,0.0091,0.0368,0.0350,
0.0344,0.0190,0.0243,0.0185,0.0279,
0.0441,0.0126,0.3759,0.0605,0.0535,
0.0470,0.0940,0.0329,0.0417,0.0605,
0.0313,0.0996,0.0765,0.2846,0.0544,
0.0188,0.0206,0.0221,0.0482,0.0564,
0.0178,0.0954,0.0747,0.0984,0.0522,
0.1107,0.0832,0.0645,0.0533,0.0757,
0.0703,0.0793,0.0718,0.0739,0.0466,
0.0400,0.0605,0.1210,0.0455,0.1486,-99
C IMPERVIOUS FRACTION
1,0.05,0.05,0.05,0.05,0.05,
0.05,0.05,0.05,0.10,0.10,
0.10,0.10,0.10,0.10,0.40,
0.50,0.75,0.10,0.40,0.50,
0.75,0.75,0.10,0.40,0.50,
0.75,0.75,0.75,0.50,0.75,
0.75,0.80,0.05,0.75,0.59,
0.75,0.75,0.75,0.75,0.80,
0.80,0.50,0.70,0.05,0.70,
0.10,0.10,0.10,0.75,0.75,
0.80,0.90,0.74,0.75,0.75,
0.75,0.75,0.69,0.75,0.80,
0.80,0.75,0.75,0.80,0.80,
0.75,0.75,0.05,0.75,0.50,-99
```