Hydrological and Environmental Engineering

Dore Road Development Services Scheme

Functional Design of the Dore Road DSS Wetland/Retarding Basin

Revision A

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Contents

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

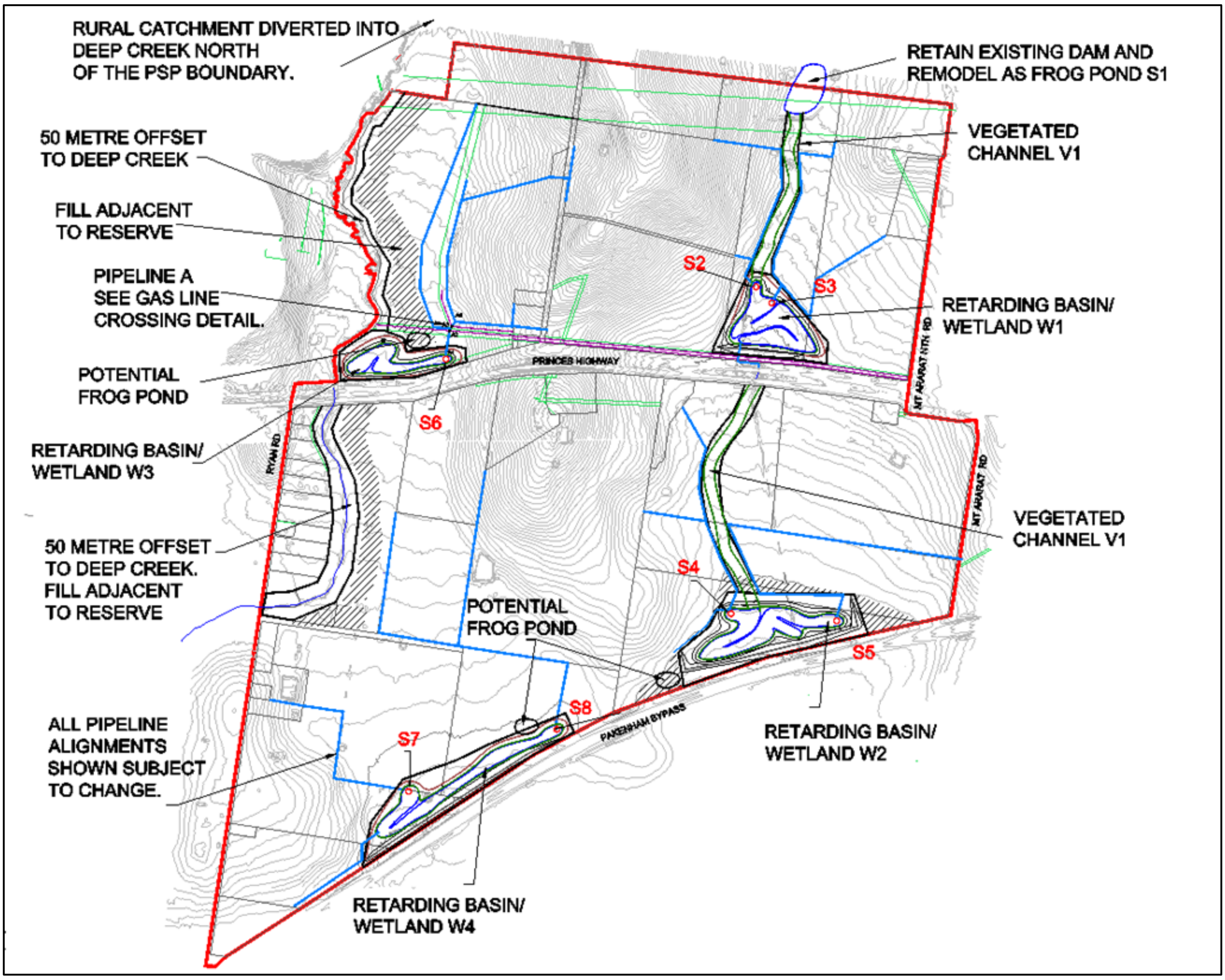

Figure 1 **2013 Preliminary PSP Drainage Strategy**

MWC has subsequently adopted the Dore Road Development Services Scheme (DSS) which covers the north-west portion of the PSP area and the catchment contributing to W3. 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 Basin W3. W3 is referred to as the Dore Road Wetland/Retarding Basin in this report.

The remainder of this report considers the functional design requirements of the wetland systems in line with MWC's "2015 Constructed Wetland Design Manual" (2015 MWC Manual).

2 Design Considerations

2.1 Outfall Considerations – Deep Creek

A major constraint in regard to the Dore Road retarding basin design is the possible backwater affects Deep Creek may have on any outlet arrangement. Tail water levels in Deep Creek have been calculated based on previous modelling described in "Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, SWS, 5 October 2014". Appendix B details the adopted Deep Creek Flood levels used for the design of the wetland/retarding basin system.

It is of important note that in extreme events, the Dore Road retarding basin forms part of the Deep Creek floodplain.

2.2 Gas Line Considerations

The Dore Road DSS is bisected by two major gas lines, the Pakenham - Wollert line and the Langford – Dandendong line as shown in Figure 2. The current PSP zonings is also shown in Figure 2. As can be seen, there are two separate residential areas which are separated by the gas easement running North – South through the DSS catchment.

As crossing the gas mains is costly, SWS and MWC have aimed to minimize the number of crossings within the DSS catchment. As such, runoff from the residential area to the west (Deep Creek side) of the North-South gas line is proposed to be treated and mitigated by a swale located within the 100m offset from deep creek.

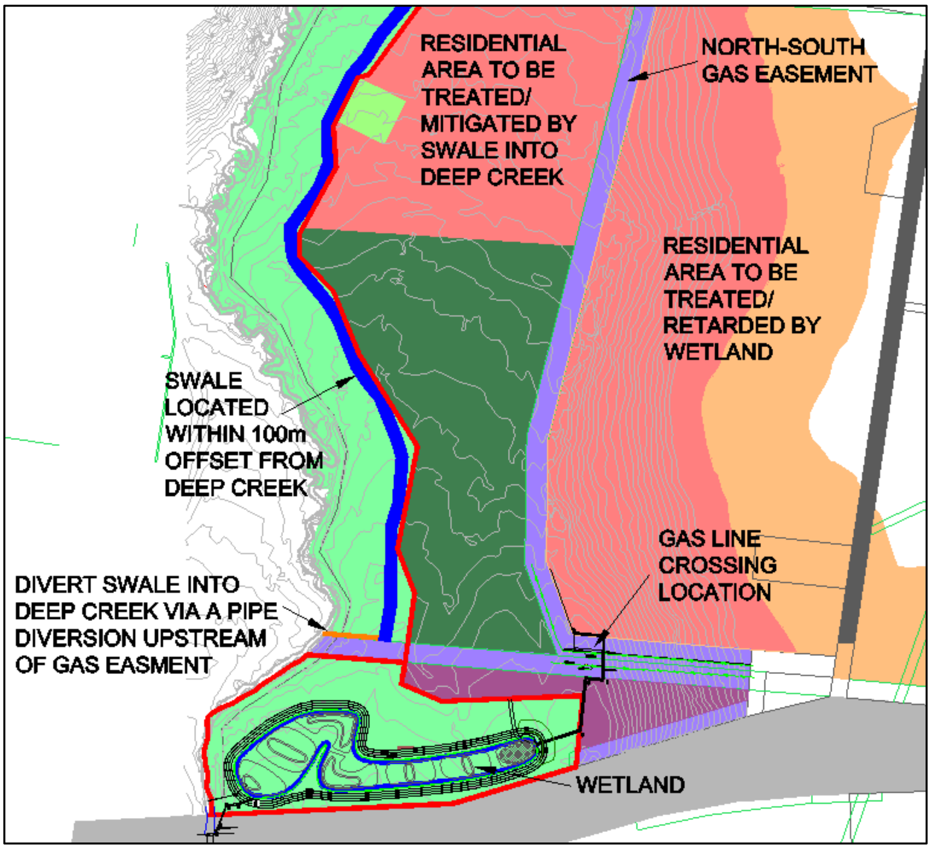

Figure 2 **Dore Road DSS Concept**

All remaining areas are proposed to be treated and retarded within the wetland at the south west of the DSS. This results in only one crossing of the gas lines. Appendix C details the design of the wetland inlet system which meets all of the Origin Energy gas line crossing requirements as detailed below.

APA (who manage the assets on behalf of Origin Energy) have advised that in regards to the gas crossing:

- APA would like all crossings to cross perpendicular to the gas mains,
- APA would prefer that no pipe connections were made between the two major gas mains, the Pakenham – Wollert line and the Langford – Dandendong line,
- When going under the mains, APA requires 500mm-600mm clearance
- When going over the main:
    - 1.2m of cover from the top of the pipe
    - There may be some coating of existing mains required
    - Possible velocity constraints of a channel may be proposed.

Detailed gas line proving has been provided by Melbourne Water at the crossing location. Cardno competed a “Utility Investigation Summary Report, Dore Road, Pakenham, 8 September 2016” which details the location and levels of the mains near the proposed crossing and should be referred to for full detail on the gas line in the vicinity of the crossing. The Utility Investigation Summary Report, Dore Road, Pakenham (8 September 2016) clearly showed that the DSS pipeline is required to go under the gas line at the location shown in Figure 2.

2.3 Water Quality Requirements

The Dore 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).

2.4 Flood Storage Requirements

The wetland/retarding basin is to be designed so that the total 1% AEP flow from the subject site post-development is less than or equal to the 1% AEP flow from the subject site in pre-development conditions.

2.5 Standards & Guidelines

The Dore Road DSS wetland has been designed as close as possible to the current (2016) MWC constructed Wetlands Design Guidelines. Appendix K shows how the wetland and sediment pond meet the deemed to comply checklist for these guidelines.

3 Proposed Functional Design

This section details the functional design of the wetland system which has been designed to meet the objectives described in Section 2. It also details the concept design of the swale system which discharges directly to Deep Creek before crossing the gas line.

3.1 Wetland

Appendix A details the functional design proposal.

The Dore Road wetland has been set with the following characteristics shown in Table 1.

In summary, the NWL has been set to 40.15 m AHD and TED has been set to 40.50 m AHD. It should be noted there is very little room to move in regard to changing the NWL as the design process moves forward. This is due to the outlet being affected by backwater effects from Deep creek (See Section 4.2.3 and Appendix B) and also due to the gas line crossing constraint upstream of the wetland (See Appendix C).

The RORB modelling detailed in Section 4 predicts the design flows originating from the upstream catchment and the retarding basin characteristics of the system.

Appendix K provides the full MWC constructed wetlands deemed to comply checklist.

Table 1 **Dore Road Wetland Characteristics**

Dore Road WETLAND								
Wetland Parameters								
Normal Water Level (NWL) =				40.15	m AHD			
Top of Extended Detention (TED) =				40.5	m AHD			
Wetland Detention Time =				72	hours	(total wetland and Sed Ponds)		
Shallow Marsh		40	to	40.15	m AHD			
Deep Marsh		39.8	to	40	m AHD			
Not Planted - deeper than			39.8	m AHD				
Treatment Areas								
Total Wetland area including Sediment Ponds at NWL =						18600	m^2	
Total Wetland area including Sediment Ponds at TED =						20750	m^2	
	Sediment Pond Areas		Sediment Pond Volumes		Detention time (hrs)			
EAST SED	1260	m^2	915	m^3	4.9	For Input Into MUSIC		
Macrophyte Zone Area at NWL =				17340	m^2			
Macrophyte Zone Area at TED =				20750	m^2			
Volume of water stored for treatment over ED range =						**0.35**	**m**	
						6666	**m^3**	
Over an average ED treatment area of						**19045**	**m^2**	
Detention time in macrophyte area =						**67.1**	**hours**	
					(the rest of the time in sed pond areas)			
Wetland Permanent Pool Volume								
Total wetland including sediment ponds								
Level	Area	Ave. Area	Delta H	Volume	Cumulative Volume			
(m AHD)	(m^2)	(m^2)	(m)	(m^3)	(m^3)			
39.15	2995				0			
39.8	4390	3693	0.65	2400	2400			
40	12990	8690	0.20	1738	4138			
40.15	18600	15795	0.15	2369	6507			
Total wetland including sediment ponds						**6507**	m^3	
Total Macrophyte Volume (exluding sediment ponds)						**5592**	m^3	
Average Macrophyte Zone Depth						**0.32**	m - OK	
						(very shallow system)		
Open Water Check (excluding S2)								
Area of Wetland below 350 mm deep =						3465	m^2	
Area of Wetland =						17340	m^2	
% Macrophyte Zone Area Vegetated (80% min)=						80%	OK	
Planting Check								
Zone:	PP Depth (m)	Minimum Average Plant	50% Plant Height	(Average plant height/2)-PP Depth	WL exceeded 20% of the time (m)	Pass ?		
Shallow Marsh	0.15	1	0.5	0.35	0.165	PASS		
Deep Marsh	0.35	1.5	0.75	0.4	0.165	PASS		

3.2 Swale (Concept)

Critical to the Dore Road wetland design is the design of the swale to convey and treat the small residential development to the west of the north-south gas line so that the only inflow into the wetland system is from the east and not from the north. MWC and Council approved this strategy in early 2016.

Currently, the land to the east of the 100m Deep Creek offset forms the low point and overland flow path for the catchment. Development of land to the east of the swale is proposed to be filled to appropriate levels. The swale will be designed to be the preferred overland flow path for all catchments to the north west of the north-south gas line.

The swale has been sized to convey a peak 1% AEP flow (Section 4.2.4) of 4.6 m^3/s originating from the residential development. General dimensions of the swale are as described below:

- Base Width = 17 m
- Depth = 0.4 m
- Top Width = 23.4 m
- Side Batters = 1V:8H
- Mannings n = 0.055 (i.e. heavily vegetated with sedges and rushes)

The aim of designing a swale of this size is that to the untrained eye it will not look like a constructed feature, mimicking a “natural” depression within the 100m offset to Deep Creek.

Preliminary modelling (Appendix J) indicates that a swale of this size is adequate in providing treatment, flood mitigation and providing adequate outfall invert levels to the development to the west of the north-south gas line. This swale size may be reduced at the functional design stage when more detailed modelling (HecRas and MUSIC in iteration) is completed.

The main advantages of the swale concept are as follows:

- Fewer crossings of gas mains,
- Smaller wetland treatment area needed,
- Smaller wetland flood storage volume needed,
- Small flow retardation benefits,
- Re-aligns overland flow path <u>away</u> from developable land and into 100m offset from Deep Creek, hence maximising developable land,
- Minimises fill requirements on developable land to the east of the swale as the pipes will be aiming at an invert lower than natural surface level (NSL) (at least 400 mm less fill over all land),
- Could be integrated as an effective community asset within the landscape master plan, and
- Supplements the 100 m offset required as part of the Deep Creek Flood plain requirements.

4 Hydrologic Modelling

4.1 Pre-Development Hydrological Model

The RORB Runoff Routing Program – version 6.15, developed at Monash University by E. M. Laurenson and R. G. Mein, was used to determine both the pre and post-development design flows originating from the Dore Road DSS Catchment. RORB is a general runoff and stream flow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall excess and routes this through catchment storage to produce the hydrograph.

4.1.1 RORB Model Parameters

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 of 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 & Runoff (AR&R). The value of Kr is a physical parameter related to the reach type chosen by the modeller which is automatically calculated by RORB.

In this case, flow gauging information was not available. As such the empirical equation recommended by Melbourne Water for South Eastern Melbourne and the Westernport Catchment was used as detailed below.

$$K_c = 1.53 \times A^{0.55} = 1.53 \times 1.279^{0.55} = 1.752$$

$$m = 0.8$$

Other parameters of RORB are the initial loss (IL) and the Pervious Area Runoff Coefficient (C_{Perv}). IL is the amount of rainfall needed before runoff occurs. Again, the Melbourne water regional parameter set was used:

$$IL = 10\ mm$$

The pervious area runoff coefficient adopted for the all events is in line with the Melbourne Water Corporations Guidelines as follows:

$$C_{1\%AEP,perv} = 0.6, C_{9.5\%AEP,perv} = 0.4, C_{18.13\%AEP,perv} = 0.3, C_{1\ EY,perv} = 0.2$$

It should be noted that C_{perv} just relates to the pervious areas of the catchment. The important Fraction impervious parameter, which dominates flow reactions, is detailed in Table 3 below.

Other inputs into the model were that Siriwardena and Weinmann areal reduction factors (A = 0 km^2) were used and a filtered temporal pattern was used.

The 1987 rainfall intensities for the location 38.050 S, 145.525 E were utilised in the model. (Raw Data: 17.91, 4.21, 1.32, 34.4, 8.02,2.45, Skew = 0.37, F2 = 4.26, F50 = 15.01). This location is just to the North of the Dore Road Catchment. The intensities have been compared to the new Australian Rainfall and Runoff (ARR) 2016 Rainfall Intensities (Issued 26 October 2016). Unfortunately, the new intensities cannot currently be entered into RORB, as such a comparison has been completed to show that the 1987 intensities are applicable for use for this design. Table 2 shows the comparison of the two IFD methods. As can be seen, on average the 1987 IFD's produce higher rainfall depths for a given duration. As such the 1987 intensities are deemed appropriate for use in this design.

Table 2 1987 and 2016 IFD Comparison

Duration	1% AEP Depth (mm)			1 EY Depth (mm)		
	1987 IFD	2016 IFD	Difference	1987 IFD	2016 IFD	Difference
10m	20.9	20.8	-0.1	5.9	6.9	1.0
15m	25.3	25.5	0.2	7.4	8.4	1.0
20m	28.6	28.9	0.3	8.5	9.6	1.1
25m	31.2	31.5	0.3	9.5	10.6	1.1
30m	33.4	33.6	0.2	10.4	11.4	1.1
45m	38.5	38.3	-0.2	12.4	13.4	1.0
1h	42.2	41.7	-0.5	14.0	15.0	1.1
1.5h	50.2	46.9	-3.3	16.6	17.6	1.0
2h	56.6	51.2	-5.4	18.8	19.6	0.8
3h	66.8	58.6	-8.2	22.2	23.0	0.8
4.5h	78.8	68.0	-10.8	26.3	26.9	0.6
6h	88.7	76.4	-12.3	29.6	30.0	0.4
9h	104.7	91.2	-13.5	35.0	35.0	0.0
12h	117.8	104.2	-13.6	39.4	38.9	-0.5
18h	137.2	126.3	-10.9	46.2	44.9	-1.3
24h	152.4	144.5	-7.9	51.7	49.5	-2.2
30h	165.0	159.8	-5.1	56.2	53.1	-3.1
36h	175.5	172.8	-2.7	60.0	56.2	-3.8
48h	192.3	193.3	1.0	66.1	61.1	-5.0
72h	214.5	219.9	5.4	74.4	68.1	-6.3
		Average:	-4.4		Average:	-0.6

Note: "Difference" relates to the 2016 intensities compared to the 1987 intensities.

4.1.2 Pre-development RORB Model Description

Figure 3 details the RORB model for the pre-development conditions model and Table 3 and Table 4 detail the tabulation of the RORB model setup (i.e. catchment area, fraction imperviousness, reach lengths, etc.).

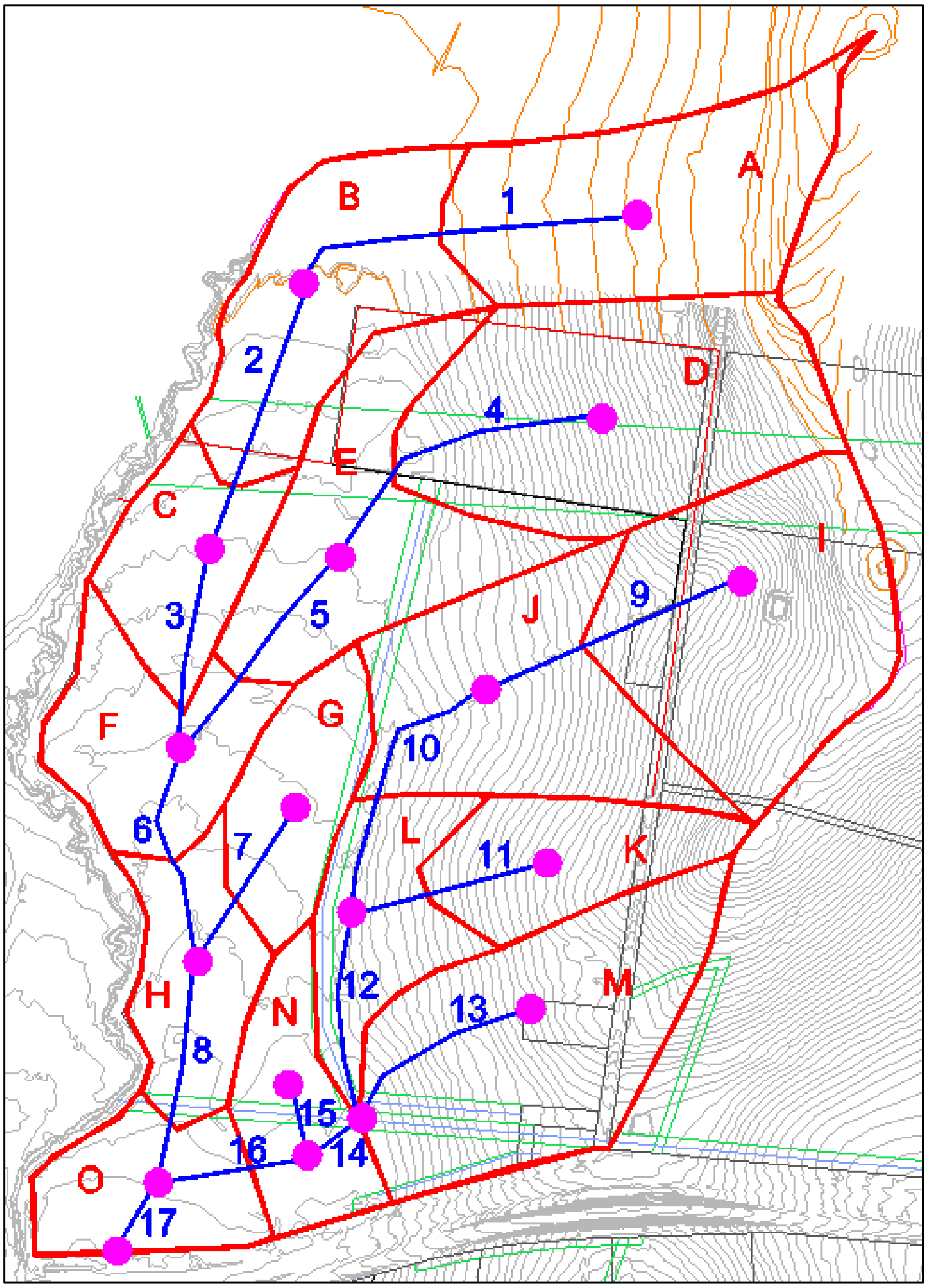

Figure 3 **Pre-development RORB Model**

Table 3 Pre-development Sub Catchment Details

Sub Area	Area (ha)	Area (km^2)	Fraction
			Imperviousness
A	12.0056	0.12006	0.05
B	9.5219	0.09522	0.05
C	5.2729	0.05273	0.05
D	15.2191	0.15219	0.05
E	10.1191	0.10119	0.05
F	6.4447	0.06445	0.05
G	5.1130	0.05113	0.05
H	5.2827	0.05283	0.05
I	13.7454	0.13745	0.05
J	10.7507	0.10751	0.05
K	5.6057	0.05606	0.05
L	5.5137	0.05514	0.05
M	13.2486	0.13249	0.05
N	4.9332	0.04933	0.05
O	5.1146	0.05115	0.05
Total:	**127.89**	**1.279**	**0.05**

Table 4 Pre-development Reach Details

Reach	Length (km)	Reach Type
1	0.476	NATURAL
2	0.385	NATURAL
3	0.265	NATURAL
4	0.432	NATURAL
5	0.341	NATURAL
6	0.298	NATURAL
7	0.245	NATURAL
8	0.305	NATURAL
9	0.377	NATURAL
10	0.376	NATURAL
11	0.263	NATURAL
12	0.291	NATURAL
13	0.287	NATURAL
14	0.092	NATURAL
15	0.100	NATURAL
16	0.213	NATURAL
17	0.111	NATURAL

4.1.3 Model Verification

It is required to check the estimated flows against other flow calculation methods to ensure the RORB model developed is valid for application. To achieve this check design flows are compared against two flow computational methods.

The 100Year ARI flows using the RORB model were compared to:

- Flows estimated by the Rational Method
- Flows obtained from the Rural Flood Regression Curve for Victoria produced by the Department of Conservation and Natural Resources 1994 ($Q_{100} = 4.67 \times A^{0.763}$, A in km^2)

Note: Flow estimates from these methods assume <u>no</u> catchment storage effects and therefore and are <u>not</u> to be used for designing or planning purposes.

The time of concentration for the rational method was calculated assuming an overland flow velocity of 0.5m/s and an initiation time of 5 minutes. This resulted in a time of concentration of 80 minutes. Design Rainfall intensities for Pakenham (38.050S,145.525E) were used (38 mm/hr). A 100-year runoff coefficient of 0.32 was used for existing conditions.

As can be seen from Table 5, the RORB model for the pre-development conditions is close to the comparison methods so was deemed appropriate for use in this study.

Table 5 RORB Model Verification

Method	Q_{100} (m^3/s)
RORB	4.45
Rational	4.27
Regression	5.63

4.1.4 Pre-development Results

Table 6 below details the pre-development RORB results.

Table 6 Pre-development RORB Results

$Q_{1\ EY,Peak}$ (m^3/s)	$Q_{18.13\%AEP,Peak}$ (m^3/s)	$Q_{9.5\%AEP,Peak}$ (m^3/s)	$Q_{1\%AEP,Peak}$ (m^3/s)
0.44 (9-hour)	1.2 (9-hour)	1.8 (9-hour)	4.45 (9-hour)
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Note: brackets detail the critical duration of the peak flow

4.2 Post-Development Hydrological Model

The same RORB parameters as detailed in section 4.1.1 were used for the post-development RORB model.

4.2.1 Post-development RORB Model Description

RORB model catchment boundaries was estimated by 1m contour information and the proposed sub divisional layout.

The RORB model shown Figure 4 was developed to model the post-development conditions at the site.

Table 7 and Table 8 detail the tabulation of the post-development RORB model setup (i.e. catchment area, fraction imperviousness, reach lengths, etc.).

The fraction impervious of the subject site was changed depending on the proposed land use for each sub area.

Table 7 Post-development Sub Catchment Details

Sub Area	Area (ha)	Area (km^2)	Fraction Imperviousness
A	5.0022	0.05002	0.074
B	6.0796	0.06080	0.287
C	5.0457	0.05046	0.359
D	4.2675	0.04268	0.497
E	1.9266	0.01927	0.482
F	3.4949	0.03495	0.750
G	3.0375	0.03038	0.750
H	2.7733	0.02773	0.471
I	3.1131	0.03113	0.750
J	2.6100	0.02610	0.484
K	3.0387	0.03039	0.750
L	4.4894	0.04489	0.750
M	3.4651	0.03465	0.478
N	2.4934	0.02493	0.574
O	0.8263	0.00826	0.050
P	0.6069	0.00607	0.050
Q	1.5906	0.01591	0.470
R	0.8852	0.00885	0.800
S	2.5876	0.02588	0.067
T	12.0419	0.12042	0.077
U	3.3588	0.03359	0.740
V	2.7089	0.02709	0.736
W	2.9329	0.02933	0.641
X	1.9876	0.01988	0.703
Y	8.3051	0.08305	0.050
Z	6.2016	0.06202	0.050
AA	5.3245	0.05325	0.050
AB	5.1401	0.05140	0.050
AC	1.8236	0.01824	0.000
AD	1.0555	0.01056	0.000
AE	2.3603	0.02360	0.000
AF	3.2697	0.03270	0.100
AG	3.4495	0.03450	0.100
AH	2.0222	0.02022	0.000
AI	1.7456	0.01746	0.100
AJ	2.9175	0.02918	0.100
AK	4.0638	0.04064	0.000
Total:	**128.04**	**1.280**	**0.299**

Table 8 Post-development Reach Details

Reach	Length (km)	Slope (%)	Reach Type
1	0.267	8.61%	PIPED
2	0.190	10.02%	PIPED
3	0.277	4.69%	PIPED
4	0.103	4.85%	PIPED
5	0.242	4.96%	PIPED
6	0.179	6.15%	PIPED
7	0.126	3.97%	PIPED
8	0.273	2.56%	PIPED
9	0.088	1.14%	PIPED
10	0.109	12.83%	PIPED
11	0.110	5.45%	PIPED
12	0.128	0.78%	PIPED
13	0.189	7.41%	PIPED
14	0.104	5.77%	PIPED
15	0.269	1.12%	PIPED
16	0.127	6.30%	PIPED
17	0.365	4.94%	PIPED
18	0.100	8.04%	PIPED
19	0.212	6.13%	PIPED
20	0.036	0.69%	PIPED
21	0.170	6.62%	EX/UNLINED
22	0.126	0.33%	EX/UNLINED
23	0.041	0.61%	PIPED
24	0.071	4.93%	PIPED
25	0.121	0.41%	PIPED
26	0.316	0.33%	PIPED
27	0.296		DROWNED
28	0.593	2.28%	PIPED
29	0.145	0.69%	PIPED
30	0.065	1.55%	PIPED
31	0.149	0.34%	PIPED
32	0.158	0.95%	PIPED
33	0.061	1.64%	PIPED
34	0.099	0.33%	PIPED
35	0.267		NATURAL
36	0.268		NATURAL
37	0.248		NATURAL
38	0.220		NATURAL
39	0.164		NATURAL
40	0.076		NATURAL
41	0.188		NATURAL
42	0.120		NATURAL
43	0.150	1.00%	EX/UNLINED
44	0.184	0.82%	EX/UNLINED
45	0.131		NATURAL
46	0.113		NATURAL
47	0.105	0.48%	EX/UNLINED
48	0.182	0.33%	EX/UNLINED
49	0.130	1.15%	PIPED
50	0.010		DROWNED

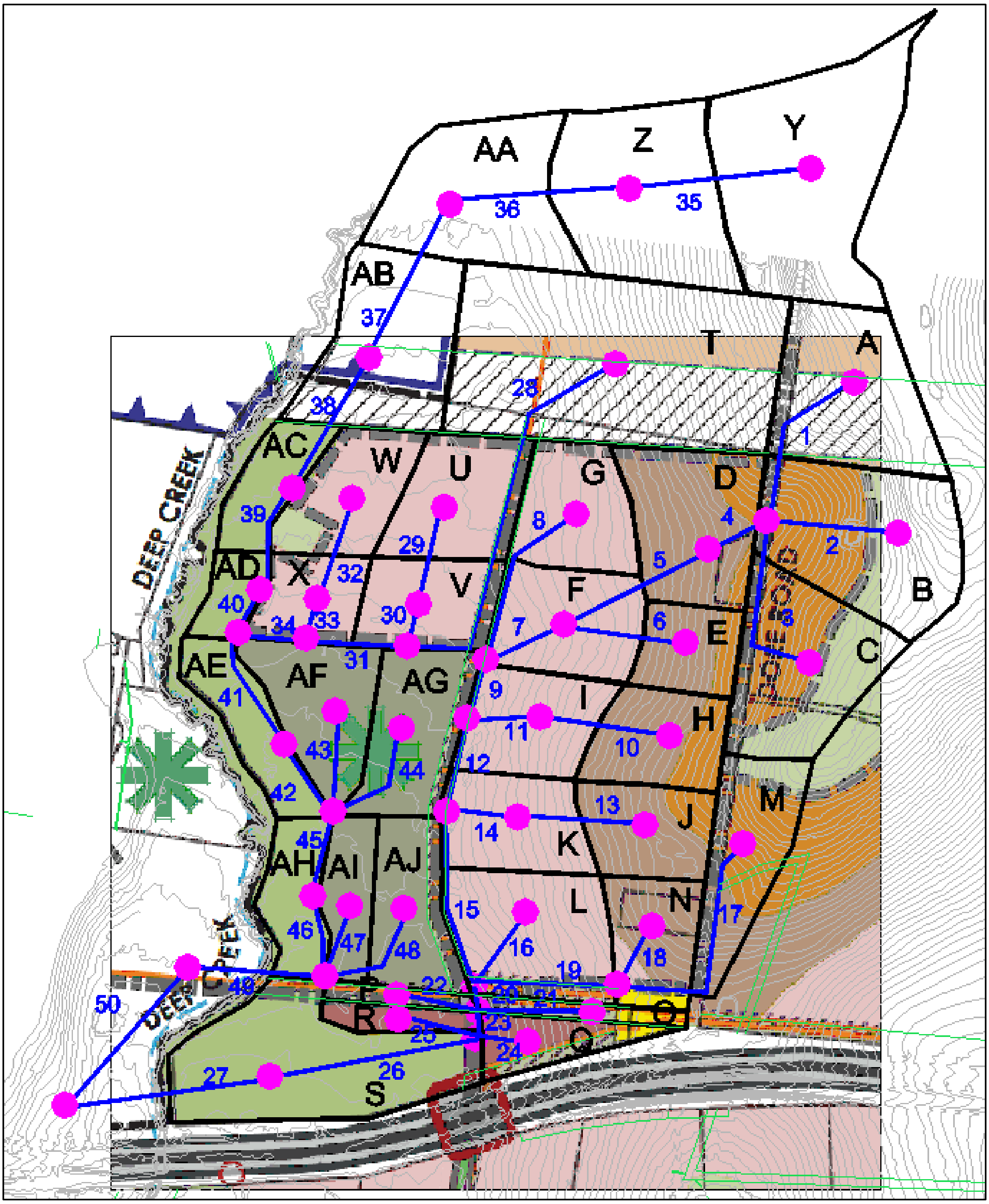


Figure 4 SWS Post-development RORB model of the Dore Road DSS Catchment

Note: Catchment delineation based on 2016 PSP proposals and is subject to change.

4.2.2 Model Verification

It is required to check the estimated flows against other flow calculation methods to ensure the RORB model developed is valid for application. To achieve this check design flows are compared against other flow computational methods.

The flows using the 100 Year ARI RORB model was compared to:

- Flows estimated by the Rational Method
- Flows obtained from the Urban Flood Regression Curve for Victoria produced by the Department of Conservation and Natural Resources 1994 ($Q_{100} = 10.29 \times A^{0.71}$, A in km^2)

Note: Flow estimates from these methods assume no catchment storage effects and therefore and are not to be used for designing or planning purposes. Also, for the purpose of the verification, the model was altered so that the storage at the outlet was not present.

As can be seen from Table 9, the RORB model for the post-development conditions is close to the comparison methods so was deemed appropriate for use in this study.

Table 9 RORB Model Verification

Method	Q_{100} (m^3/s)
RORB	12.1
Rational	13.0
Regression	12.3

4.2.3 Retarding Basin Stage/Storage/Discharge

The wetland has been designed to function as a retarding basin. As the retarding basin is located within the declared Deep Creek flood plain, depending on the timing of any storm, the retarding basin may be controlled by the effects of the downstream water level in Deep Creek. As such, two conditions were modelled for each AEP simulation:

Condition 1: The retarding basin outlet can freely outflow into Deep Creek (i.e. the Deep Creek water level is low). This has been modelled as:

- Deep Creek at the pipe outlet obvert (40.20 m AHD) for all events, and

Condition 2: The retarding basin outlet is drowned out by the backwater effects from Deep Creek and can only operate under minimal head for each event.

4.2.3.1 Stage/Storage Relationship

For both conditions, the Stage/Storage relationship was the same. For both conditions, the stage/storage relationship was derived using design contours as detailed in Appendix A. Calculations are shown in Appendix B.

4.2.3.2 Stage/Discharge Relationship

For both conditions, the wetland/retarding basin outlet has been configured to:

- Detain stormwater for wetland treatment between the levels of 40.15 and 40.50 m AHD via the use of weir controls and submerged offtake pits connected with balance pipes,
- Retard the 1% AEP flow so that the combination of the flow entering Deep Creek from the swale system (which discharges directly to Deep Creek) and from the retarding basin system is less than the total pre-development flow into Deep Creek.

Due to the limitations within RORB, a different stage/discharge relationship is required for each flooding condition in Deep Creek and for each AEP simulation. Detailed calculations on the outlet arrangement are as shown in Appendix B. In summary, for the 1% AEP simulations the Stage/Storage/Discharge (SSD) relationships are shown in Table 10 and Table 11 below. It should be noted the wetland ED Outlet has no flood control from Deep Creek (assumed minimal inflow volume from Deep Creek if Deep Creek level is high). The high flow outlet has been designed with a duck bill outlet to restrict large volumes of water from Deep Creek entering the wetland during flood events.

Table 10 1% AEP SSD, Condition 1, Deep Creek Flood Level = 40.20 m AHD (Downstream Obvert)

Level (m)	Storage (m^3)	Outflow (m^3/s)
40.15	0	0.00
40.5	6895	0.025
40.9	13672	0.71
41.3	25920	1.42
41.5	31073	1.55
41.8	39645	1.72
42	45360	1.82

Table 11 1% AEP SSD, Condition 2, Deep Creek Flood Level = 42.30 m AHD (1% AEP Level in Deep Creek)

Level (m)	Storage (m^3)	Outflow (m^3/s)
40.15	0	0
40.5	6895	0.025
40.6	9154	0.026
42.3	54788	0.027
42.4	57930	0.62

Note: Deep Creek 1% flood level as determined in Appendix B.

It should be noted that:

- Condition 1 is more conservative in regard to outflow requirements, and
- Condition 2 is more conservative in regard to site flood levels.

This is why <u>both</u> conditions are required to be analysed.

4.2.4 Post-development Results

4.2.4.1 Condition 1 – Free Outflow to Deep Creek

Location	1% AEP	9.5% AEP	18.13% AEP	1 EY
Upstream of Gas Crossing (Reach 19)	11.9 m^3/s (15 min)	5.0 m^3/s (15 min)	4.0 m^3/s (2 hr)	2.2 m^3/s (2 hr)
Retarding Basin Inflow	11.4 m^3/s (15 min)	4.9 m^3/s (2 hr)	3.9 m^3/s (2 hr)	2.0 m^3/s (2 hr)
Retarding Basin Outflow	1.42 m^3/s (12 hr)	0.77 m^3/s (12 hr)	0.60 m^3/s (12 hr)	0.35 m^3/s (48 hr)
Retarding Basin Storage	26,400 m^3	14,800 m^3	12,600 m^3	10,000 m^3
Retarding Basin Flood Level	41.30 m AHD	40.95 m AHD	40.85 m AHD	40.70 m AHD
Swale Flow from Subdivision (Reach	4.6 m^3/s (15 min)	1.9 m^3/s (25 min)	1.4 m^3/s (25 min)	0.7 m^3/s (2 hr)
Swale Outflow (Reach 50)	2.8 m^3/s (9 hr)	1.3 m^3/s (12 hr)	0.9 m^3/s (9 hr)	0.4 m^3/s (9 hr)
Total Development Outflow	4.2 m^3/s (9 hr)	2.05 m^3/s (9 hr)	1.45 m^3/s (9 hr)	0.55 m^3/s (9 hr)
File:	#1: 1603_Dore_Rd_DSS_PostDev_100Yr_Outlfow_V8_27Oct16.cat			

4.2.4.2 Condition 2 – Deep Creek High

Location	1% AEP #2	9.5% AEP #3	18.13% AEP #4	1 EY #5
Upstream of Gas Crossing (Reach 19)	11.9 m^3/s (15 min)	5.0 m^3/s (15 min)	4.0 m^3/s (2 hr)	2.2 m^3/s (2 hr)
Retarding Basin Inflow	11.4 m^3/s (15 min)	4.9 m^3/s (2 hr)	3.8 m^3/s (2 hr)	2.0 m^3/s (2 hr)
Retarding Basin Outflow	0.40 m^3/s (36 hr)	0.38 m^3/s (72 hr)	0.22 m^3/s (72 hr)	0.03 m^3/s (72 hr)
Retarding Basin Storage	56,300 m^3	32,800 m^3	29,300 m^3	18,100 m^3
Retarding Basin Flood Level	42.45 m AHD	41.55 m AHD	41.45 m AHD	41.10 m AHD
Swale Flow from Subdivision (Reach	4.6 m^3/s (15 min)	1.9 m^3/s (25 min)	1.4 m^3/s (25 min)	0.7 m^3/s (2 hr)
Swale Outflow (Reach 50)	2.8 m^3/s (9 hr)	1.3 m^3/s (9 hr)	0.9 m^3/s (9 hr)	0.4 m^3/s (9 hr)
Total Development Outflow	2.95 m^3/s (9 hr)	1.35 m^3/s (9 hr)	0.95 m^3/s (9 hr)	0.45 m^3/s (9 hr)
Files:	#2: 1603_Dore_Rd_DSS_PostDev_100Yr_FLOOD_LVL_V8_27Oct16.cat			
	#3: 1603_Dore_Rd_DSS_PostDev_10Yr_FLOOD_LVL_V8_27Oct16.cat			
	#4: 1603_Dore_Rd_DSS_PostDev_5Yr_FLOOD_LVL_V8_27Oct16.cat			
	#5: 1603_Dore_Rd_DSS_PostDev_1Yr_FLOOD_LVL_V8_27Oct16.cat			

As detailed:

- Condition 1 results in the higher discharge to Deep Creek ($Q_{1\%AEP\ POST}$ = 4.2 m^3/s), and this is shown to be lower the pre-development flow in the 1% AEP event ($Q_{1\%AEP\ PRE}$ = 4.45 m^3/s).
- Condition 2 results in the higher flood level (42.45 m AHD) in the Dore Road RB. These results should be used to set flood and floor level requirements for the future development of the site.

It should be noted; this retarding basin design assumes flow generated from catchments T – AJ discharges directly to Deep Creek and is not retarded in the retarding basin. Flow from catchments T-AJ are proposed to be mitigated by the use of swales. These swales will convey flows to the Deep Creek via a connection which is to be located to the north of the gas easement. Currently, the swale design has only been completed conceptually and needs to be finalised at the next functional design stage of the design process.

4.2.5 Extreme Flow Considerations

As the retarding basin is located within the existing Deep Creek flood plain, in extreme events the retarding basin is expected to be completely inundated by Deep Creek.

The retarding basin is an almost complete cut construction. Thus, the proposed retarding basin is providing excess flood storage volume to the floodplain in extreme events.

The connection between the retarding basin and Deep Creek is the existing levee between the two systems. It is expected the levee would be overtopped in an extreme event by the water in Deep Creek spilling into the floodplain, not the water in the retarding basin spilling into Deep Creek.

As discussed in Section 4.2.3 above, high levels of Deep Creek have been assumed when setting flood levels within the basin. The basin (if Deep Creek is low) can store events rarer than the 1% AEP event within the volume of the basin.

In essence, in very rare events (1 in 1,000 to 1 in 1,000,000 AEP) the entire site is inundated by Deep Creek flows.

Due to the above, <u>no</u> ANCOLD (or similar assessment) should be required as in an extreme event, the retarding basin forms part of the Deep Creek floodplain.

5 Sediment Pond Design

The wetland designs aim to provide primary treatment to flow conveyed in the incoming pipes via the use of a large sediment ponds upstream of the 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 Appendix E.

In regard to the design of the sediment pond it should be noted that, as described in the functional design drawings, it:

- Has been designed to ensure the sediment build up height in 5 years is lower than 350 mm below the sediment pond NWL,
- Is located online to the DSS pipelines which they are treating water from,
- Captures greater than 95% of course particles ≥ 125 μm diameter for the peak three month ARI (4 EY) flow,
- Provide adequate sediment storage volume to store 5 years of sediment (volume between the base level and 'NWL – 350 mm'),
- Ensures 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).
- Incorporates 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),
- Incorporates 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 from the DSS pipeline is possible by blocking off inlet headwalls and pits and sending water around the treatment system via the bypass pipeline. Drawing DORE/SWS/1 (Appendix A) detail how this can occur.

6 Stormwater Pollutant Modelling

The performance in regard to stormwater pollutant retention of the Dore Road 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 5 Figure 5details 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 Table 1. Furthermore, the proposed swale treating the north-west catchment has been modelled to show that the total DSS treats to best practice before discharging into Deep Creek.

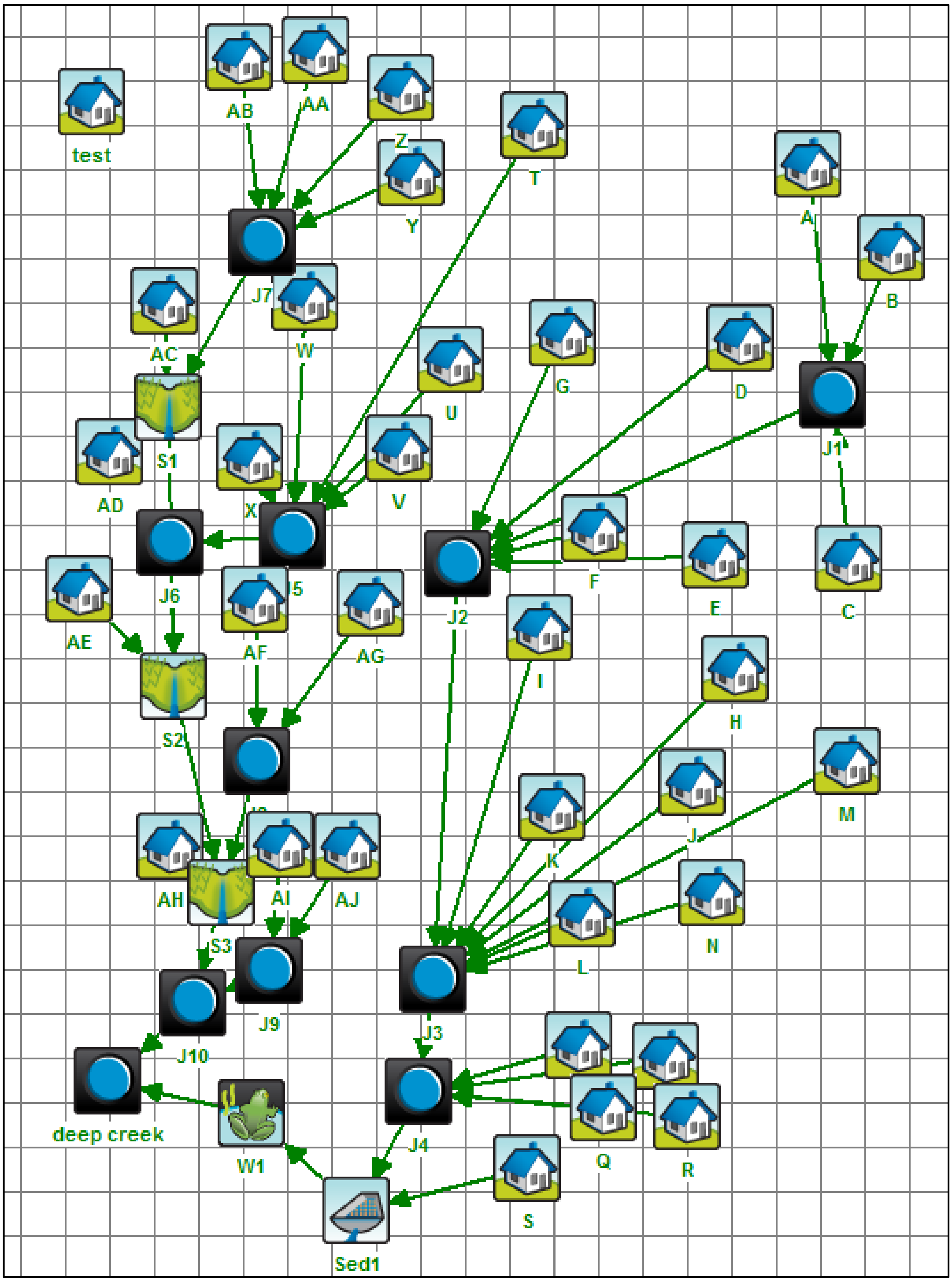


Figure 5 **MUSIC Model Schematic**

Note: Catchment delineation based on 2016 PSP proposals and is subject to change.

Table 12 MUSIC Results at Wetland and DSS Outlet (External Catchments Included)

		Sources	Residual Load	% Reduction
Wetland Outlet	Flow (ML/yr)	306	284	7.4
	Total Suspended Solids (kg/yr)	46000	3450	92.5
	Total Phosphorus (kg/yr)	106	21.7	79.5
	Total Nitrogen (kg/yr)	822	354	57.0
	Gross Pollutants (kg/yr)	9640	0	100.0
Swale Outlet	Flow (ML/yr)	235	235	0.0
	Total Suspended Solids (kg/yr)	20300	4000	80.3
	Total Phosphorus (kg/yr)	61.3	32	47.8
	Total Nitrogen (kg/yr)	573	351	38.5
	Gross Pollutants (kg/yr)	3460	151	95.6
Deep Creek	Flow (ML/yr)	541	519	4.1
	Total Suspended Solids (kg/yr)	66300	7450	88.8
	Total Phosphorus (kg/yr)	167.3	53.7	67.9
	Total Nitrogen (kg/yr)	1395	705	49.5
	Gross Pollutants (kg/yr)	13100	151	98.8

The current best practice requirements of 80% TSS, 45% TP and 45% TN retention can be met by proposed wetland system and at the development outlet. As such, the functional design of the elements meets the requirements of the current Dore Road DSS. It should be noted that the results shown in Table 12 include the pollutants generated by the external catchments to the north. Once the external catchment is removed from the modelling, as shown in Table 13, it is clear that the total treatment train is adequately sized to treat stormwater to well in excess of best practice.

Table 13 MUSIC Results at Wetland and DSS Outlet (External Catchments Excluded)

		Treatment Train Effectivness at Outlet			Pollutants Generated from External Catchments	Pollutants Generated from all Internal Catchments	% Reduction
		Sources	Residual Load	Pollutants Treated			
Wetland Outlet	Flow (ML/yr)	306	284	22.0	81.4	224.6	10%
	Total Suspended Solids (kg/yr)	46000	3450	42550.0	9690	36310	100%
	Total Phosphorus (kg/yr)	106	21.7	84.3	24.7	81.3	100%
	Total Nitrogen (kg/yr)	822	354	468.0	209	613	76%
	Gross Pollutants (kg/yr)	9640	0	9640.0	2190	7450	100%
Swale Outlet	Flow (ML/yr)	235	235	0.0	110	125	0%
	Total Suspended Solids (kg/yr)	20300	4000	16300.0	4880	15420	100%
	Total Phosphorus (kg/yr)	61.3	32	29.3	22.7	38.6	76%
	Total Nitrogen (kg/yr)	573	351	222.0	251	322	69%
	Gross Pollutants (kg/yr)	3460	151	3309.0	469	2991	100%
Deep Creek	Flow (ML/yr)	541	519	22.0	191.4	349.6	6%
	Total Suspended Solids (kg/yr)	66300	7450	58850.0	14570	51730	100%
	Total Phosphorus (kg/yr)	167.3	53.7	113.6	47.4	119.9	95%
	Total Nitrogen (kg/yr)	1395	705	690.0	460	935	74%
	Gross Pollutants (kg/yr)	13100	151	12949.0	2659	10441	100%

File: 1603_Dore_Rd_3Nov16_V8.sqz

7 Further Work Required

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

- As part of the wetland detailed design;
  - Detailed survey of Deep Creek where the wetland outlet is designed to ensure levels are accurate,
  - Confirmation of the design of the inlet system under the gas mains,
  - Soil tests should be performed to confirm the assumption that the existing clayey soil (once compacted) will be suitable for construction of a wetland system without and additional liner,
- Finalisation of the PSP land zonings (if there are any major changes, the wetland functional design may need to be reviewed),
- Ecological and archaeological studies are required to ensure no adverse impacts to existing site values,
- Functional and detailed design of the swale system, including setting invert levels of pipes at major DSS outfall points, and
- Confirmation of the Deep Creek flood level within the proposed swale and subsequent setting of required adjacent development fill levels.

Council and MWC may wish to consider reducing the retarding basin site sight allocation slightly via reducing the northern boundary as detailed in drawing DORE/SWS/1. Reducing the land allocation as per drawing DORE/SWS/1 provides an approximate extra 1.4 ha of developable land.

APA group and Origin Energy should also be kept informed of all major updates relating to works in the vicinity of their easements.

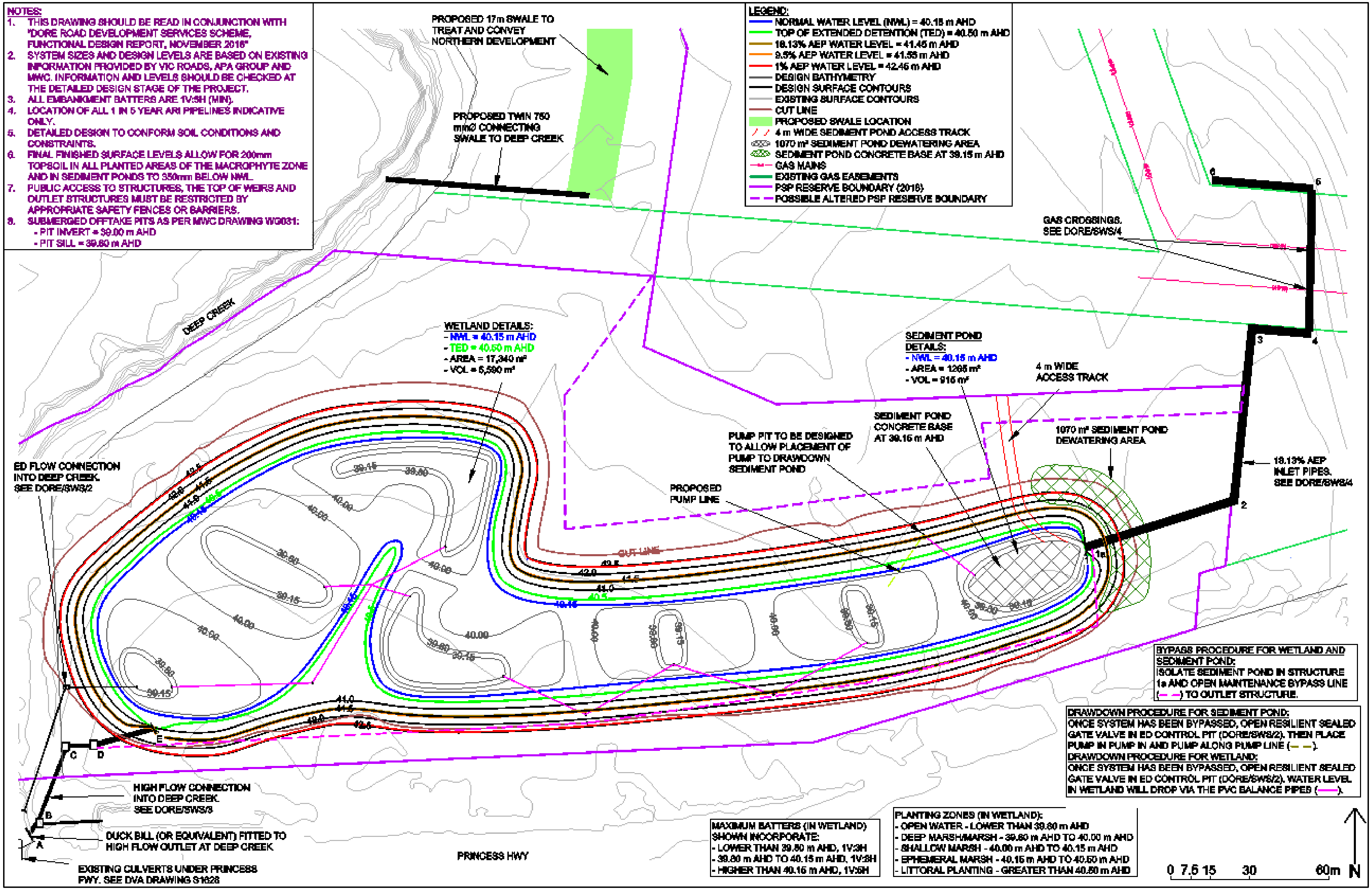
8 Abbreviations, Descriptions and Definitions

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

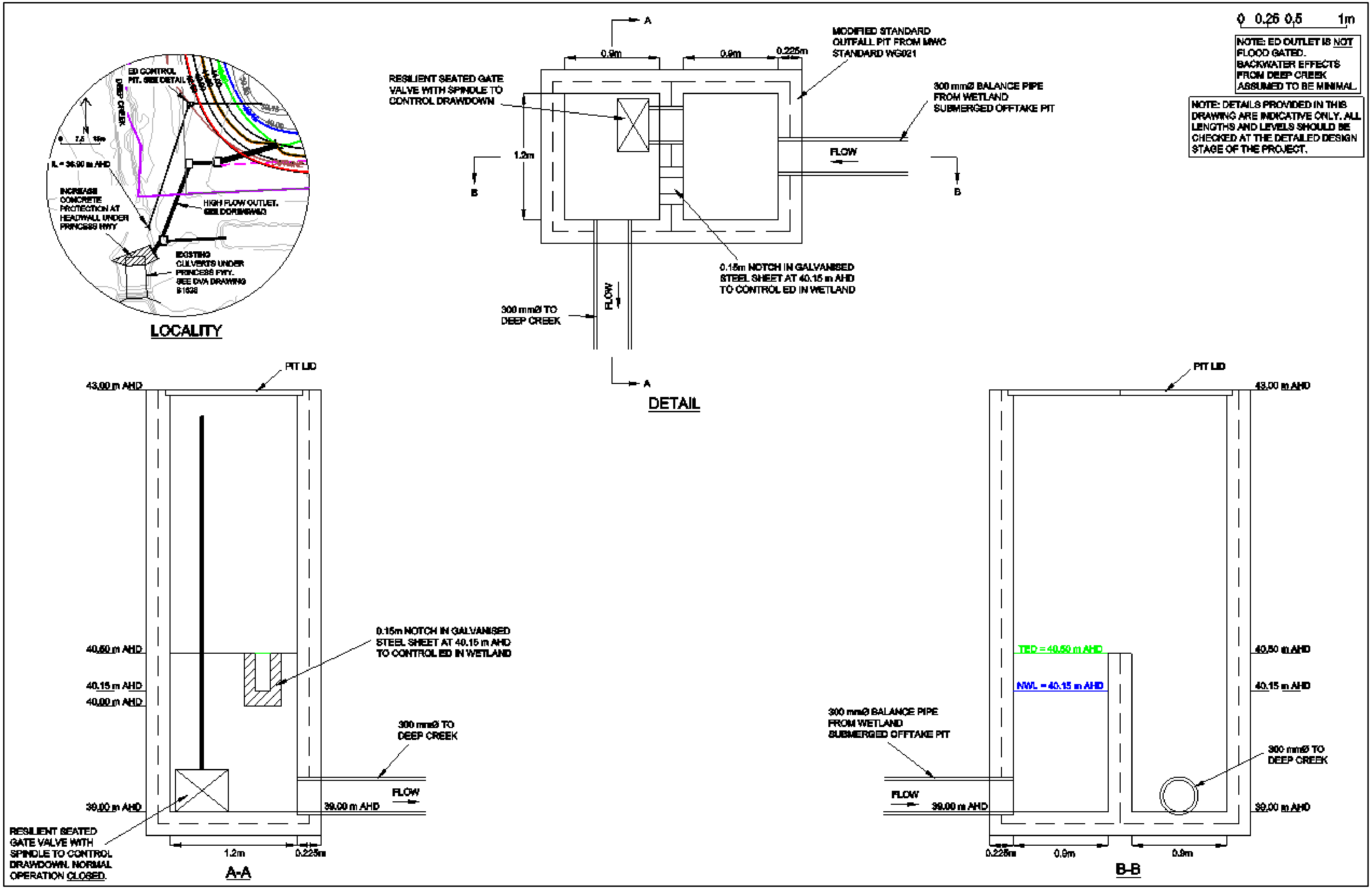
Abbreviation Descriptions	Definition
AHD - Australian Height Datum	Common base for all survey levels in Australia. Height in metres above mean sea level.
AEP – Annual Exceedance Probability	The probability of an event being exceeded per year. i.e. 1% AEP = 100 Year ARI event.
ARI - Average Recurrence Interval.	The average length of time in years between two floods of at least a given size. A 100 Year ARI event has a 1% chance of occurring in any one year.
DSS	Development Services Scheme – catchment drainage strategies developed, implemented and run by MWC.
EY – Exceedances per Year	Magnitude of event that is expected to be exceeded X times per year. i.e. 4 EY = 3-month ARI event.
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 (NWL)	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.
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.

Appendix A – Functional Design Drawings

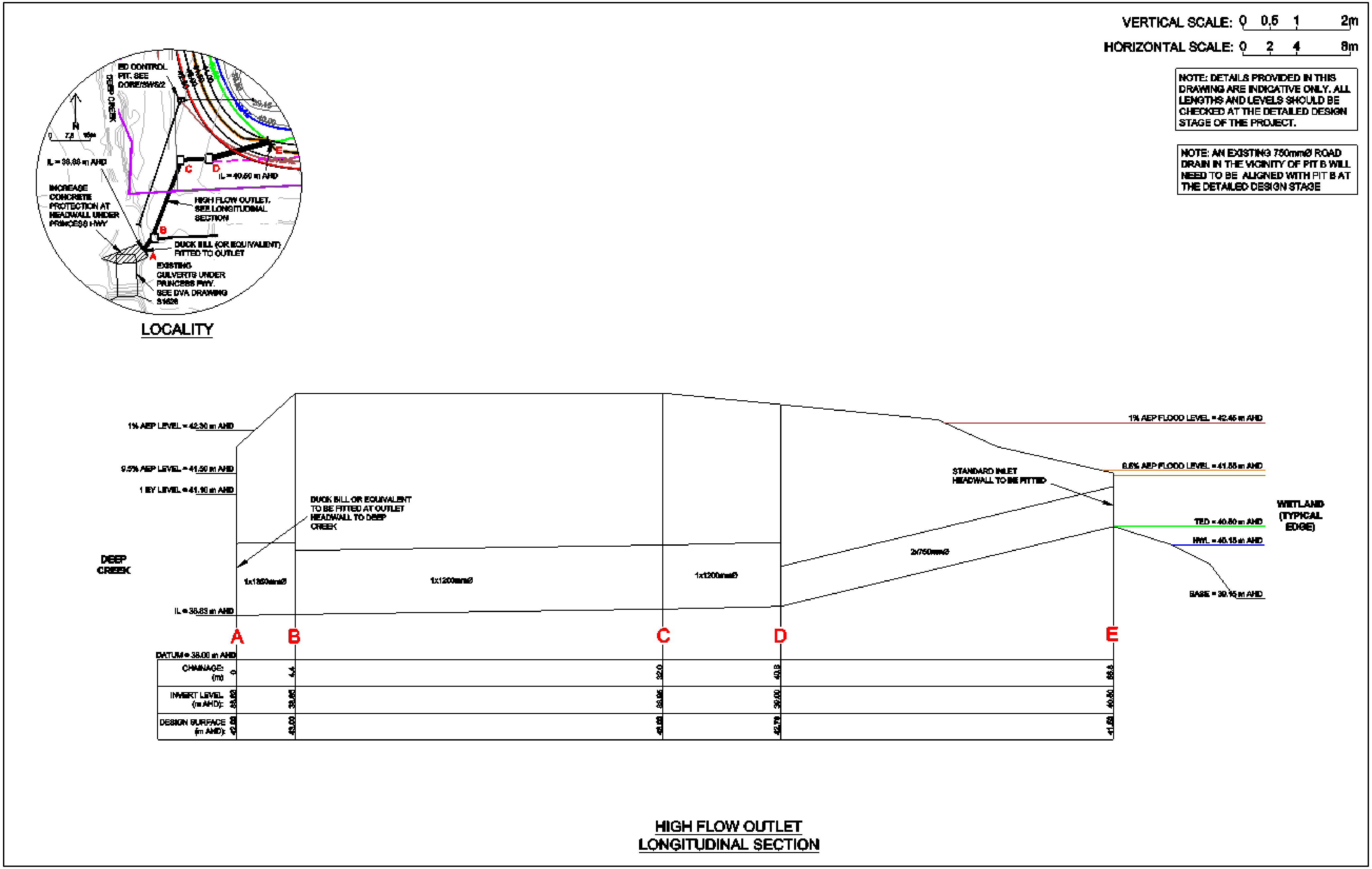
Note that the drawings detailed have been modified from the AutoCAD set. The AutoCAD drawings should be referred to for the full design detail.



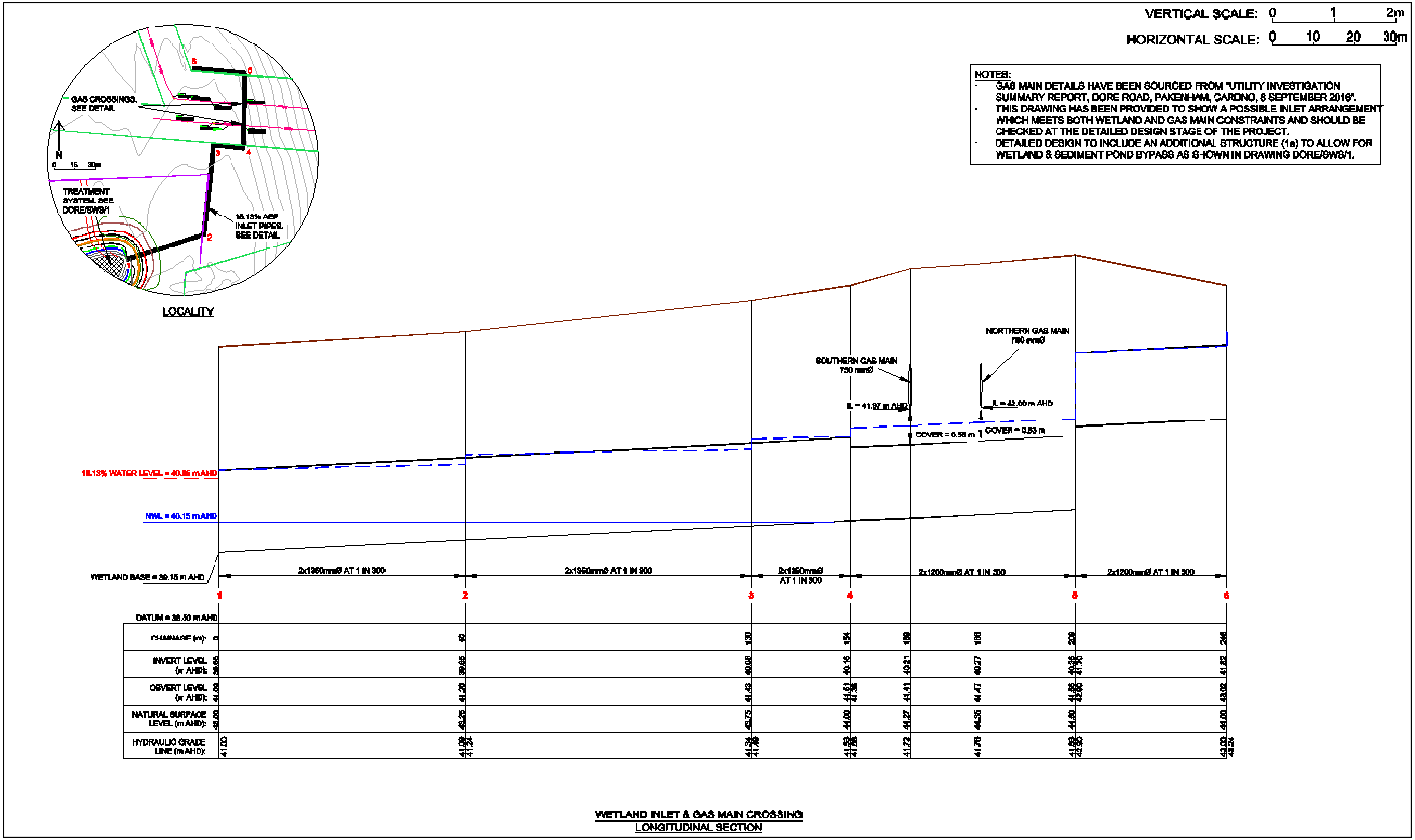
DORE/SWS/1 Wetland/Retarding Basin Detail



DORE/SWS/2 Wetland/Retarding ED Outlet Detail



DORE/SWS/3 Wetland/Retarding High Flow Outlet Detail



DORE/SWS/4 Wetland/Retarding Inlet and Gas Crossing Detail

Appendix B – Wetland Outlet Calculations

B.1 Calculation of tail water levels in Deep Creek at RB Outlet

As the retarding basin is located within the declared Deep Creek Flood Plain, for any given event the outlet arrangement to the retarding basin may be effected by the downstream water level in Deep Creek. Previous modelling completed by SWS (See report titled: “Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014”) has indicated that the flows and flood levels within Deep Creek, downstream of Princess Hwy, are as follows:

Event	Q (m^3/s)	WL (m AHD)
1% AEP	44	41.7
9.5% AEP	18	41.3
18.13% AEP	14	41.2
1 EY	6.8	41.0

Using the above information, and the details provided on the culverts underneath Princess Hwy, SWS has completed road culvert calculations on the system as shown below:

Deep Creek at Princes Highway						
Based on Vic Roads Drawings						
Assume all culverts flowing under outlet control						
Melbourne Bound carriageway culvert system						
Width =		3.9	m			
Depth =		3.7	m			
Number of culverts =		2				
Culvert Length =		18.3	m			
Upstream IL =		38.69	m AHD			
Upstream obvert level =		42.39	m AHD			
Downstream IL =		38.62	m AHD			
Downstream obvert level =		42.32	m AHD			
Upstream headwall =		43.29	m AHD			
100 Year Flood Level Downstream =			41.70	m AHD		3.08
Note : Culvert contriction control of Level in 100yr analysis, TWL for other Events						
Culverts not operating under pressure - as confirmed by Vic Roads design drawings						
using mannings formula for a recatngular concrete channel.						
Water Depth in culvert =		3.24	m			
Culvert Base width		3.9	m	Note: Iterating HGL to make		
Longitudinal Slope		0.00040		depth of flow in culvert		
side slope of batters		1 in	0	(close to drowned)		
Flow Area (A)		12.636	m^2			
ss length		3.24	m			
Wetted Perimeter (P)		10.38	m			
Hydraulic Radius (R)		1.22	m			
mannings n		0.013				
Capacity (Q)		44.3	m^3/s - twin culverts - Ok approx 44 m3/s)			
Velocity (V)		1.75	m/s			
Exit Loss coefficient =		1				
Exist loss = $K_{ex}V^2/2g$ =		0.16	m			
Depth in culvert = TWL + exist loss =			3.24		OK, matches above	
Inlet Loss coefficient =		0.5				
Inlet loss = $K_eV^2/2g$ =		0.08	m			
HWL =	42.01	.= tail water level of Warrigal bound culvert				

Warrigal Bound carriageway culvert system					
Width =		3.9	m		
Depth =		3.7	m		
Number of culverts =		2			
Culvert Length =		13.1	m		
Upstream IL =		38.83	m AHD		
Upstream obvert level =		42.53	m AHD		
Downstream IL =		38.8	m AHD		
Downstream obvert level =		42.5	m AHD		
Upstream headwall =		43.15	m AHD		
100 Year Flood Level Downstream =			42.01	m AHD	
Culverts not operating under pressure - as confirmed by Vic Roads design drawings					
Water Depth in culvert =		3.35	m		
Culvert Base width		3.9	m	Note: Iterating HGL to make	
Longitudinal Slope		0.00036		depth of flow in culvert	
side slope of batters		1 in	0	(close to drowned)	
Flow Area (A)		13.065	m^2		
ss length		3.35	m		
Wetted Perimeter (P)		10.60	m		
Hydraulic Radius (R)		1.23	m		
mannings n		0.013			
Capacity (Q)		43.8	m^3/s - twin culverts - Ok approx 44 m3/s)		
Velocity (V)		1.68	m/s		
Exit Loss coefficient =		1			
Exist loss = $K_{ex}V^2/2g$ =		0.14	m		
Depth in culvert = TWL + existing gloss =			3.35		OK, matches above
Inlet Loss coefficient =		0.5			
Inlet loss = $K_eV^2/2g$ =		0.07	m		
HWL =	42.25	.= tail water level of Warrigal bound culvert			
		Confirmed , culverts not running under pressure.			

Calculations have only been shown for the 1% AEP Event. Full calculations are provided in the file: *Deep Creek at Princes Highway - culvert analysis_2Nov16.xlsx*

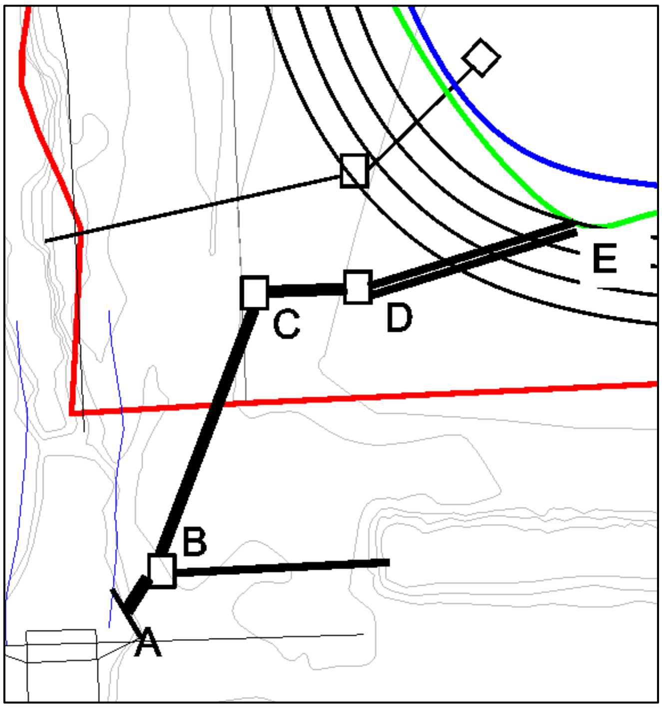
The results from the analysis for the different events is shown below (at Point A):

Event	WL At RB Outlet (m AHD)
1% AEP	42.3
9.5% AEP	41.5
18.13% AEP	41.4
1 EY	41.1

As such, the results shown above have then formed the tail water level's (TWL's) used as part of the design of the RB outlet system for both conditions as the Dore Road RB outlet system outfalls just upstream of the princess highway culvert system.

B.2 Retarding Basin Outlet Arrangement

The outlet arrangement to the RB is as shown in Drawings DORE/SWS/2 & 3. The outlet arrangement is shown conceptually below.



The standard MWC notch weir controls extended detention from 40.15 m AHD to 40.50 m AHD as detailed below:

Wetland rectangular notch ED Control betweeen NWL & TED							
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 Len	0.15	m					
Area at NWL =	18600	m2					
Area at TED =	20800	m2	(conservatively same as TED)				
Volume of water stored for treatment over Ed range						0.35	m
				.=		6895	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 Volume	ED Detention Time (hrs)		
40.15	0	0.15	0.000	0			
40.5	0.35	0.08	0.028	6895	68		

For RB levels above TED, a hydraulic grade line analysis has been conducted on the outlet arrangement. The analysis is shown below for the 1% AEP, Condition 2 (Deep Creek at 42.30 m AHD).

File: 1603_Dore_Rd_DSS_RORB_SSD_V8_28Oct16.xlsm

Design Flow =	0.616	m^3/s	
Structure A - Outlet into Deep Creek			
Deep Creek Level =	42.3		
Culvert B to A			
Culvert Size =	1.35	mØ	
Number of Culverts =	1		
Downstream IL =	38.83	m AHD	
Downstream Obvert =	40.18	m AHD	
Upstream IL =	38.85	m AHD	
Upstream Obvert =	40.2	m AHD	
Length =	4.4	m	
TWL =	42.30	m AHD	
Simplified Colebrook-White Formula			
head loss = S x L			
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$			
pipe dia =	1.35	m	
RCP pipe radius =	0.675	m	
Design flow =	0.924	m^3/s	ADDITIONAL 50% FLOW FROM 750 ASSUMED
Wetted perimeter =	4.24	m	
Area =	1.43	m^2	
Hyd radius =	0.3375	m	
V =	0.65	m/s	
k =	0.0015		
S =	0.000317		
L=	4.4	m	
Head loss = HL =	0.00	m	
Head Water Level = TWL +	42.30	m AHD	
Structure B - Connection with assumed 750 mm Ø			
Pit Invert =	38.85	m AHD	
TWL =	42.30	m AHD	
V_{out} =	0.65	m/s	
k_{pit} =	1.5		assume 750 mmØ from road contributes equal flow to Downstream Pipe
h_{pit} =	0.03	m	
Head Water Level = TWL +	42.33	m AHD	

Culvert C to B		
Culvert Size =	1.2	mØ
Number of Culverts =	1	
Downstream IL =	38.85	m AHD
Downstream Obvert =	40.05	m AHD
Upstream IL =	38.95	m AHD
Upstream Obvert =	40.15	m AHD
Length =	27.6	m
TWL =	42.33	m AHD
Simplified Colebrook-White Formula		
head loss = S x L		
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$		
pipe dia =	1.2	m
RCP pipe radius =	0.6	m
Design flow =	0.616	m^3/s
Wetted perimeter =	3.77	m
Area =	1.13	m^2
Hyd radius =	0.3	m
V =	0.54	m/s
k =	0.0015	
S =	0.000261	
L=	27.6	m
Head loss = HL =	0.01	m
Head Water Level = TWL +	42.34	m AHD
Structure C - Junction Pit "L" Bend		
Pit Invert =	38.95	m AHD
TWL =	42.34	m AHD
V_{out} =	0.54	m/s
k_{pit} =	1.3	Junction Pit on "L" Bend
h_{pit} =	0.02	m
Head Water Level = TWL +	42.36	m AHD

Culvert D to C			
Culvert Size =	1.2	mØ	
Number of Culverts =	1		
Downstream IL =	38.95	m AHD	
Downstream Obvert =	40.15	m AHD	
Upstream IL =	39	m AHD	Constraint so Wetland Can be drawndown
Upstream Obvert =	40.2	m AHD	
Length =	8.8	m	
TWL =	42.36	m AHD	
Simplified Colebrook-White Formula			
head loss = S x L			
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$			
pipe dia =	1.2	m	
RCP pipe radius =	0.6	m	
Design flow =	0.616	m^3/s	
Wetted perimeter =	3.77	m	
Area =	1.13	m^2	
Hyd radius =	0.3	m	
V =	0.54	m/s	
k =	0.0015		
S =	0.000261		
L=	8.8	m	
Head loss = HL =	0.00	m	
Head Water Level = TWL +	42.36	m AHD	
Structure D - Junction Pit with ED on Lateral			
Pit Invert =	39	m AHD	
TWL =	42.36	m AHD	
V_{out} =	0.54	m/s	
k_{pit} =	0.5		Junction Pit on THROUGH PIPE WITH SOME LATERAL (ed)
h_{pit} =	0.01	m	
Head Water Level = TWL +	42.37	m AHD	

Culvert E to D		
Culvert Size =	0.75	mØ
Number of Culverts =	2	
Downstream IL =	39	m AHD
Downstream Obvert =	39.75	m AHD
Upstream IL =	40.5	m AHD
Upstream Obvert =	41.25	m AHD
Length =	25	m
TWL =	42.37	m AHD
Simplified Colebrook-White Formula		
head loss = S x L		
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$		
pipe dia =	0.75	m
RCP pipe radius =	0.375	m
Design flow =	0.308	m^3/s
Wetted perimeter =	2.36	m
Area =	0.44	m^2
Hyd radius =	0.1875	m
V =	0.70	m/s
k =	0.0015	
S =	0.000773	
L=	23	m
Head loss = HL =	0.02	m
Head Water Level = TWL +	42.39	m AHD
Structure E - Inlet Headwall		
Pit Invert =	40.5	m AHD
TWL =	42.39	m AHD
V_{out} =	0.70	m/s
k_{Entry} =	0.5	
$h_{structure}$ =	0.01	m
Head Water Level = TWL +	42.40	m AHD

The advantage of the hydraulic grade line analysis is that it easily allowed SWS to compute the RB outflow and RB level for any given Deep Creek TWL level as shown below:

		Water Level in RB																		
		41.2	41.3	41.4	41.5	41.6	41.7	41.8	41.9	42.0	42.1	42.2	42.3	42.4	42.5	42.6	42.7	42.8	42.9	43.0
Deep Creek Level	41.1	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133	2.22	2.304	2.385	2.463	2.539	2.612	2.684
	41.2	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133	2.22	2.304	2.385	2.463	2.539	2.612
	41.3	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133	2.22	2.304	2.385	2.463	2.539
	41.4	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133	2.22	2.304	2.385	2.463
	41.5	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133	2.22	2.304	2.385
	41.6	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133	2.22	2.304
	41.7	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133	2.22
	41.8	0	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042	2.133
	41.9	0	0	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947	2.042
	42.0	0	0	0	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847	1.947
	42.1	0	0	0	0	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742	1.847
	42.2	0	0	0	0	0	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629	1.742
	42.3	0	0	0	0	0	0	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508	1.629
	42.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0.616	0.871	1.067	1.232	1.377	1.508

Using the above table, SWS was able to Formulate SSD's for each event (1%, 9.5%, 18.13% AEP, 1 EY) and for each condition. It should be noted SWS has assumed that the although the RB is affected by the TWL in Deep Creek, a small flow 0.026 to 0.027 m^3/s, is able to be conveyed out into Deep Creek, even when the flood level in Deep Creek is high.

B.3 Stage/Storage Discharge Tables

B.3.1 Condition 1

The retarding basin outlet can freely outflow into Deep Creek (i.e. Deep Creek water level low, at outlet obvert). High exit loss (K_{ex} = 5) on Duck bill flood gate.

- Assumed WL in Deep Creek = 41.10 m AHD
- SSD as shown below.

Level (m)	Storage (m^3)	Flow (m^3/s)
40.15	0	0.00
40.5	6895	0.025
40.9	13672	0.71
41.3	25920	1.42
41.5	31073	1.55
41.8	39645	1.72
42	45360	1.82

B.3.2 Condition 2

The retarding basin outlet is drowned out by the backwater effects from Deep Creek and can only operate under minimal head.

1% AEP Simulation:

- Deep Creek Level = 42.30 m AHD
- SSD shown below:

Level (m)	Storage (m^3)	Flow (m^3/s)
40.15	0	0
40.5	6895	0.025
40.6	9154	0.026
42.3	54788	0.027
42.4	57930	0.62

9.5% AEP Simulation:

- Deep Creek Level = 41.50 m AHD
- SSD Shown Below:

Level (m)	Storage (m^3)	Flow (m^3/s)
40.15	0	0
40.5	6895	0.025
40.6	9154	0.026
41.5	31073	0.027
41.6	33930	0.62
41.8	39645	1.07
42.0	45360	1.38

18.13% AEP Simulation:

- Deep Creek Level = 41.40 m AHD
- SSD Shown Below:

Level (m)	Storage (m^3)	Flow (m^3/s)
40.15	0	0
40.5	6895	0.025
40.6	9154	0.026
41.4	28496	0.027
41.5	31073	0.62
41.8	39645	1.23
42.0	45360	1.51

1 EY Simulation:

- Deep Creek Level = 41.10 m AHD
- SSD shown below:

Level (m)	Storage (m^3)	Flow (m^3/s)
40.15	0	0
40.5	6895	0.025
40.6	9154	0.026
41.1	20767	0.027
41.2	23343	0.62
41.5	31073	1.23
42.0	45360	1.85

Appendix C – Wetland Inlet & Gas Crossing Calculations

18.13% AEP flows into the Dore Road wetland/RB from the large residential zoning land to the north east of the catchment are proposed to be conveyed within a 18.13% AEP pipe system. It is assumed events larger than this will be conveyed into the wetland/retarding basin via appropriate overland flow path provisions across the gas mains (to be designed once development layout is known).

Two 750 mmØ gas mains separate this large residential development from the wetland/retarding basin. The design of the 18.13% AEP system to upstream of the gas mains has been completed to accommodate this constraint. Sizing this system upfront ensures that the NWL of the wetland is set correctly to allow for appropriate slope (1 in 300) on the 18.13% AEP inlet system while maintaining Origin Energy's conditions in regards to the crossing of the Gas Mains.

APA (who manage the assets on behalf of Origin Energy) have advised that in regards to the gas crossing:

- APA would like all crossings to cross perpendicular to the gas mains,
- APA would prefer that no pipe connections were made between the two major gas mains, the Pakenham – Wollert line and the Langford – Dandendong line,
- When going under the mains, APA requires 500mm-600mm clearance
- When going over the main:
    - 1.2m of cover from the top of the pipe
    - There may be some coating of existing mains required
    - Possible velocity constraints of a channel may be proposed.

APA gas main contacts for design going forward should be:

- Peter Dawson (03) 9797 5149
- Daniel Tucci (03) 9797 5330 daniel.tucci@apa.com.au
- Ken Adamson (03) 5976 0314 ken.adamson@apa.com.au

Note: Ken Adamson should be the first contact if available

As part of the functional design process, the gas mains have been surveyed in the vicinity of the crossing. Cardno competed a "Utility Investigation Summary Report, Dore Road, Pakenham, 8 September 2016" which details the location and levels of the mains near the proposed crossing. Analysis of the findings is that the gas main is approximately 1m higher at the eastern end of the survey area compared to the western end. As such, SWS has designed the pipe alignment to cross as far east as known, thus maximising the pipe size allowed in the design while keeping the system under appropriate head.

Assuming the NWL of the wetland is 40.15 m AHD as detailed in this functional design, SWS has completed a HGL analysis of the 18.13% AEP inlet system to ensure that all of the above Origin requirements are met and that the pipes have appropriate slope.

Figure 6 below shows conceptually the pipe layout and labels key structures (junction/inlet pits) in relation to the gas mains (survey location shown), easement and wetland/retarding basin.

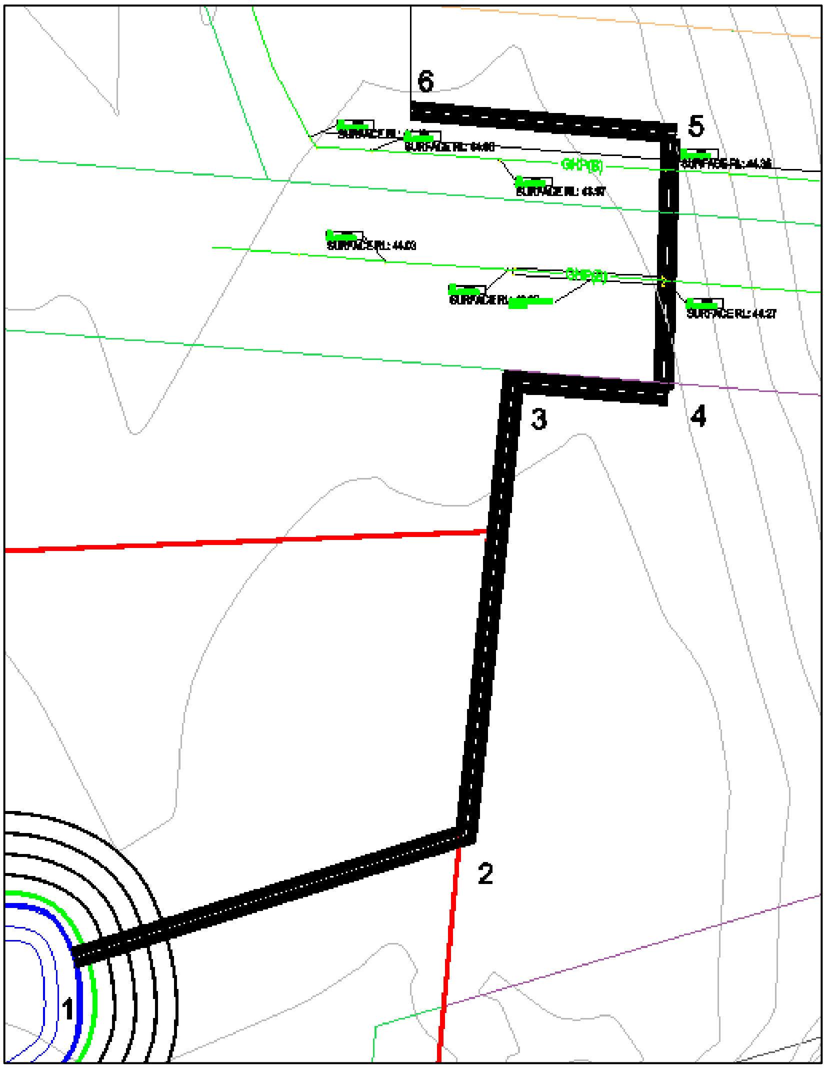


Figure 6 Conceptual layout of 18.13% AEP Inlet Pipe System

The hydraulic grade line (HGL) analysis performed by SWS below assumes:

- Inlet pipes into the wetland/retarding basin can be drowned (i.e. 37% drowned 1350 mmØ inlet pipes),
- NWL of the wetland/retarding basin is at 40.15 m AHD,
- The downstream condition on the HGL is the peak 18.13% AEP flood level in the basin when the basin is able to freely outflow to Deep Creek. i.e. 18.13% AEP level = 40.85 m AHD,
- Where any downstream conditions were calculated at lower than the downstream obvert of any pipe, the downstream obvert level was set for the analysis, and
- The maximum cover below gas mains = 0.5 m.

The HGL analysis is provided in detail below:

Note: Overland Flows to be directed into the Sediment Pond through appropriate provisions.

File: 1603_Dore_Rd_Gas_Crossing_16Nov16.xlsx

Q_5		4	m^3/s
Wetland Flood Level =		40.85	m AHD
Point 1			
NSL =	43		
V_{out} =	1.40	m/s	
k_{out}=	1		
h_{exit} =	0.10	m	
Culvert 1-2			
Culvert Size =		1.35	mØ
Number of Culverts =		2	
Downstream IL =		39.65	m AHD
Downstream Obvert =		41	m AHD
Slope =		300	
Upstream IL =		39.85	m AHD
Upstream Obvert =		41.2	m AHD
Length =		60	m
TWL =		41.00	m AHD
	(max of obvert or flood level + Exit Loss)		
Simplified Colebrook-White Formula			
head loss = S x L			
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$			
pipe dia =		1.35	m
RCP pipe radius =		0.675	m
Design flow =		2.0	m^3/s
Wetted perimeter =		4.24	m
Area =		1.43	m^2
Hyd radius =		0.337456	m
V =		1.40	m/s
k =		0.0015	
S =		0.001485	
L=		60	m
Head loss = HL =		0.09	m
Head Water Level = TWL + H		41.09	m AHD

Structure 2			
NSL =		43.25	m AHD
Pit Invert =		39.85	m AHD
TWL =		41.09	m AHD
V_{out} =	1.40	m/s	
k_{pit} =	1.5		L Bend Junction Pit
h_{pit} =	0.15	m	
Head Water Level = TWL + H		41.24	m AHD
Culvert 2-3			
Culvert Size =		1.35	mØ
Number of Culverts =		2	
Downstream IL =		39.85	m AHD
Downstream Obvert =		41.2	m AHD
Slope =		300	
Upstream IL =		40.08	m AHD
Upstream Obvert =		41.43	m AHD
Length =		70	m
TWL =		41.24	m AHD
Simplified Colebrook-White Formula			
head loss = S x L			
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$			
pipe dia =		1.35	m
RCP pipe radius =		0.675	m
Design flow =		2.0	m^3/s
Wetted perimeter =		4.24	m
Area =		1.43	m^2
Hyd radius =		0.337456	m
V =		1.40	m/s
k =		0.0015	
S =		0.001485	
L=		70	m
Head loss = HL =		0.10	m
Head Water Level = TWL + H		41.34	m AHD

Structure 3				
NSL =		43.75	m AHD	
Pit Invert =		40.08	m AHD	
TWL =		41.34	m AHD	
V_{out} =	1.40	m/s		
k_{pit} =	1.5		L Bend Junction Pit	
h_{pit} =	0.15	m		
Head Water Level = TWL + I		41.49	m AHD	
Culvert 3-4				
Culvert Size =		1.35	mØ	
Number of Culverts =		2		
Downstream IL =		40.08	m AHD	
Downstream Obvert =		41.43	m AHD	
Slope =		300		
Upstream IL =		40.16	m AHD	
Upstream Obvert =		41.51	m AHD	
Length =		24	m	
TWL =		41.49	m AHD	
Simplified Colebrook-White Formula				
head loss = S x L				
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$				
pipe dia =		1.35	m	
RCP pipe radius =		0.675	m	
Design flow =		2.0	m^3/s	
Wetted perimeter =		4.24	m	
Area =		1.43	m^2	
Hyd radius =		0.337456	m	
V =		1.40	m/s	
k =		0.0015		
S =		0.001485		
L=		24	m	
Head loss = HL =		0.04	m	
Head Water Level = TWL + I		41.53	m AHD	

Structure 4				
NSL =		44	m AHD	
Pit Invert =		40.16	m AHD	
TWL =		41.53	m AHD	
V_{out} =	1.40	m/s		
k_{pit} =	1.5		L Bend Junction Pit	
h_{pit} =	0.15	m		
Head Water Level = TWL +		41.68	m AHD	
Culvert 4-5 (Under Gas Mains)				
Culvert Size =		1.2	mØ	
Number of Culverts =		2		
Downstream IL =		40.16	m AHD	
Downstream Obvert =		41.36	m AHD	
Slope =		300		
Upstream IL =		40.35	m AHD	
Upstream Obvert =		41.55	m AHD	
Length =		55	m	
TWL =		41.68	m AHD	
Simplified Colebrook-White Formula				
head loss = S x L				
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$				
pipe dia =		1.2	m	
RCP pipe radius =		0.6	m	
Design flow =		2.0	m^3/s	
Wetted perimeter =		3.77	m	
Area =		1.13	m^2	
Hyd radius =		0.299961	m	
V =		1.77	m/s	
k =		0.0015		
S =		0.002756		
L=		55	m	
Head loss = HL =		0.15	m	
Head Water Level = TWL +		41.83	m AHD	

Gas Details:			
Southern Gas Pipe		Northern Gas Pipe	
Size =	0.75	Size =	0.75
NSL =	44.27	NSL =	44.35
Obvert =	42.72	Obvert =	42.75
Invert =	41.97	Invert =	42.00
Cover to SW Pipe =	0.50	Cover to SW Pipe =	0.50
SW Pipe Max Obvert =	41.47	SW Pipe Max Obvert =	41.50

Southern Gas Crossing			
Length from Structure 4 =	14.7	m	
SW Culvert Size =	1.2	m	
IL =	40.21	m AHD	
Obvert =	41.41	m AHD	
Cover below gas =	0.56	m	>0.5 m OK
Northern Gas Crossing			
Length from Structure 4 =	32	m	
SW Culvert Size =	1.2	m	
IL =	40.27	m AHD	
Obvert =	41.47	m AHD	
Cover below gas =	0.53	m	>0.5 m OK
Structure 5			
NSL =	44.5	m AHD	
Pit Invert =	40.35	m AHD	
US Pipe IL =	41.00		
TWL =	41.83	m AHD	
V_{out} =	1.77	m/s	
k_{pit} =	2.5		Drop pit >45°
			Change of Direction
h_{pit} =	0.40	m	
Head Water Level = TWL +	42.23	m AHD	

Note: Structure 5 is designed as a drop pit upstream of the gas mains. In this way, Deep pipes are only required downstream of this point. System sizes upstream of this pit indicative only and subject to change given final developer layouts and constraints.

Culvert 5-6			
Culvert Size =	1.2	mØ	
Number of Culverts =	2		
Downstream IL =	41.70	m AHD	
Downstream Obvert =	42.90	m AHD	
Slope =	300		
Upstream IL =	41.82	m AHD	
Upstream Obvert =	43.02	m AHD	
Length =	37	m	
TWL =	42.90	m AHD	
(Max of Obvert or TWL as drop pit)			
Simplified Colebrook-White Formula			
head loss = S x L			
$S = V^2/(\log_{10}(14.8R/k))^2/(32gR)$			
pipe dia =	1.2	m	
RCP pipe radius =	0.6	m	
Design flow =	2.0	m^3/s	
Wetted perimeter =	3.77	m	
Area =	1.13	m^2	
Hyd radius =	0.299961	m	
V =	1.77	m/s	
k =	0.0015		
S =	0.002756		
L=	38	m	
Head loss = HL =	0.10	m	
Head Water Level = TWL +	43.00	m AHD	
Structure 6			
NSL =	44	m AHD	
Pit Invert =	41.82	m AHD	
TWL =	43.00	m AHD	
V_{out} =	1.77	m/s	
k_{pit} =	1.5		L Bend Junction Pit
h_{pit} =	0.24	m	
Head Water Level = TWL +	43.24	m AHD	

The HGL analysis is shown in drawing DORE/SWS/4 (Appendix A).

Appendix D – Wetland Maintenance Bypass Calculations

Wetland systems are required to allow bypass of low flows during maintenance periods. The wetland/sediment pond maintenance bypass system has been sized by calculating the 4 EY flow from MUSIC (Section 6) (at 3-hour time intervals) and then sizing an appropriate pipe based on Mannings formula. This analysis is shown below:

Bypass provisions for maintenance - Dore Road Sediment Pond & Wetland					
Bypass provisions from Upstream to Outlet					
Music File:	1603_Dore_Rd_Inundation_3Nov16_V8.sqz				
Flux File:	Dore_Rd_Sed_3h_3Nov16.csv				
Flow Frequency Analysis, 3hour inflow data		Maximum flow calculated =	0.9	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%	0.002	Assume sediment pond to be isolated for maintenance.			
70.0%	0.005				
80.0%	0.008	**Bypass flow for maintenance activity > flow which occurs 98.17% of the time =**			
98.2%	0.109			**0.11**	**m^3/s**
99.5%	0.222	*Note: 98.17% of the time = 98.17% AEP = 3 Month ARI Flow*			
Mannings to Size Bypass	$Q = \frac{A(\frac{A}{WP})^{2/3}\sqrt{S}}{n}$				
Diameter	0.375	m			
Radius	0.1875	m			
Area	0.11	m^2			
WP	1.18	m			
Hydraulic Radius	0.09375	m			
HGL slope	0.003	m/m			
n	0.013	$s/m^{1/3}$			
Capacity	0.10	m^3/s	OK		
Velocity	0.92	m/s			

File: 1603_Dore_Rd_Maintenance_Bypass_Pipe_Sizing_V8_2Nov16.xlsx

Appendix E – Sediment Pond Design Calculations

File: 1603_Dore_Rd_Sed_Pond_Sizing_V8_17Nov16.xlsx

General

Fill in the following given the concept/functional design drawing parameters

NWL =	40.15	m AHD
TED =	40.5	m AHD
Safety batters at 1 in	8	to 350 below NW
Base level =	39.15	m AHD
Area at NWL =	1265.00	m^2
Total Volume =	915	m^3
Sump Volume (volume below350 mm depth) =	531	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

Fill in the following given the concept/functional design drawing parameters

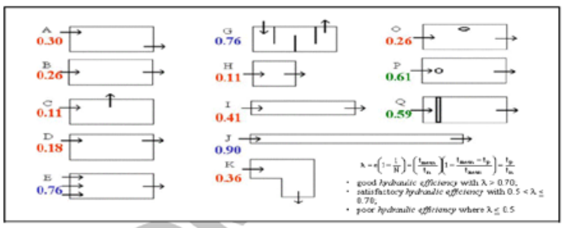


Figure 10.5 Hydraulic Efficiency – A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds; Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment [Source: Australian Runoff Quality, 2003]

Fair and Geyer Equation

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

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

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

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

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

Target = very fine sand

Vs =	0.011	m/s				
d_e =	0.35	m				
d_p =	1.0	m				
d* =	1.0	m				
(d_e+d_p) = / (d_e+d^*)	1.0					
Q_{4EY} =	0.76	m^3/s	(RULE OF T	$Q_{18.13\%AEP}$ =	3.8	m^3/s (RORB)
A =	1265.00	m^2				
V_s = / Q/A	18.31					
λ =	0.26	pond shape assumption				
n =	1.35					

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 =	54.75	ha	Areas A-R	
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 =	531	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=	533	m^3
Required Dewatering area =	1066	m^2

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

Dewatering Provision for Maintenance

Isolate shown in Drawing DORE/SWS/1 (Appendix A)

Sediment Pond Flood Flow Velocity Check

See Appendix F

Appendix F – Wetland Velocity Checks

File: 1603_Dore_Rd_DSS_Velocity_Check_V8_17Nov16.xlsx

DORE Rd Wetland Flow Velocity Checks

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

Hydraulic analysis of flow velocities, Manual calculation

Initial Velocity Checks

$Q_{1\%AEP}$ =	11.4	m^3/s (RORB)
$Q_{9.5\%AEP}$ =	4.9	m^3/s (RORB)
$Q_{18.13\%AEP}$ =	3.8	m^3/s (RORB)
Q_{4EY} =	0.8	m^3/s (Rule of Thumb given 18.13%AEP Flow)

Wetland Normal Water Level (NWL) =	40.15	m AHD
Wetland Top of Extended Detention (TED) =	40.5	m AHD
Maximum Base level at wetland narrowest width =	40.00	m AHD

1a	Peak 9.5% AEP flow through sediment pond =	4.9	m^3/s (RORB)	
	Peak 1% AEP flow through sediment pond =	11.4	m^3/s (RORB)	
1b	Bypass around Macrophyte zone =	0.0	m^3/s	
	Macrophyte zone inlet capacity =	0.0	m^3/s	
	Peak 4 EY flow through macrophyte zone =	0.8	m^3/s	
	Peak 9.5% AEP flow through macrophyte zone =	4.9	m^3/s (accounts for bypass)	
	Peak 1% AEP flow through macrophyte zone =	11.4	m^3/s (accounts for bypass)	

Initial Sediment Pond Velocity Check

2	9.5% AEP WL =	40.95	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 =	30	m		
3b	Width at 9.5% AEP WL =	34.2	m	(conservatly same as TED)	
4	9.5% AEP WL - NWL =	0.80	m		
	Average width =	32.1	m		
	Cross section flow area =	25.68	m^2		
5	1% AEP Flow velocity =	0.44	m/s < 0.5 m/s		OK

Initial Macrophyte zone Velocity Check

6	9.5% AEP WL =	40.95	m AHD		
	At narrowest part of the macrophyte zone:				
7a	NWL width =	30	m		
7b	TED width =	34.2	m		
7c	Width at 9.5% AEP WL =	34.2	m	(conservatly same as TED)	
8a	TED - base level at narrowest width =	0.50	m		
	Average width =	32.1	m		
	Cross section flow area =	16.05	m^2		
8b	9.5% AEP WL - NWL =	0.8	m		
	Average width =	34.2	m		
	Cross section flow area =	27.36	m^2		
9	4 EY Flow velocity =	0.05	m/s < 0.05 m/s		OK
10	1% AEP Flow velocity =	0.42	m/s < 0.5 m/s		OK

Appendix G – 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 DORE/SWS/2 & 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:

- 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 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 5592 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 Dore_Rd_Turn_3Nov16.csv
FILE IS UPLOADED

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

Clear Selection

Name	Average plant height (m)	Shallow marsh plants	Deep marsh plants	Suitability
Sea Club-rush *Bolboschoenus caldwellii*	1	☑		Shallow Only
Water Ribbons *Triglochin procerum*	1	☑	☐	
Jointed Club-rush *Baumea articulata*	1.8	☑	☐	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		☑	
Common Spike-rush *Eleocharis acuta*	0.5	☐		Unsuitable
+ Add user defined plant				

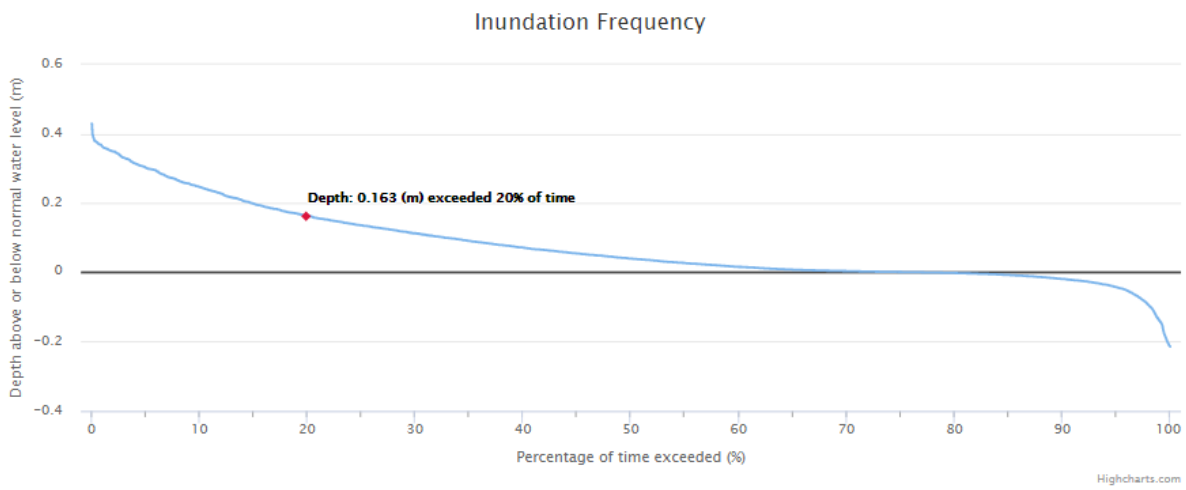
Report

File: Dore_Rd_Turn_3Nov16.csv
Shallow marsh zone meets deemed to comply criteria

Deep marsh zone meets deemed to comply criteria

Water level exceeded for 20% of time: 0.163 m

90th Percentile Residence Time: 3 days



Wetland Plant Health Requirements Check - Dore Road Wetland

Critical Plant Depth = Plant Height relative to NWL (m) — 10 Year NWN data 1984 - 1993

Extended Detention = 0.35 m

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

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

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

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

Wetland specifications

	Max Permanent Pool Depth	Average Height of Plant in wetland plant list given any exclusions	50% Average Plant Height	Check 1 - Critical Plant Height Relative to NWL (Average plant height/2)-PP
Shallow Marsh	0.15	1.0	0.50	0.35
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 = 165.0 mm

Note: 3 Hour Data used, Hence Slightly diffenrt results from MUSIC Auditor which uses daily

Percentile	WL Relative to NWL - (mm)
0.5%	-183
1.0%	-140
5.0%	-43
15.0%	-8
20.0%	-2
50.0%	37
70.0%	112
80.0%	165
97.0%	340
99.5%	398

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

YES

Check1 met

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

YES

Check1 met

Files: *MUSIC:* *1603_Dore_Rd_Inundation_3Nov16_V8.sqz*

MUSIC AUDITOR: *Dore_Rd_Turn_3Nov16.csv*

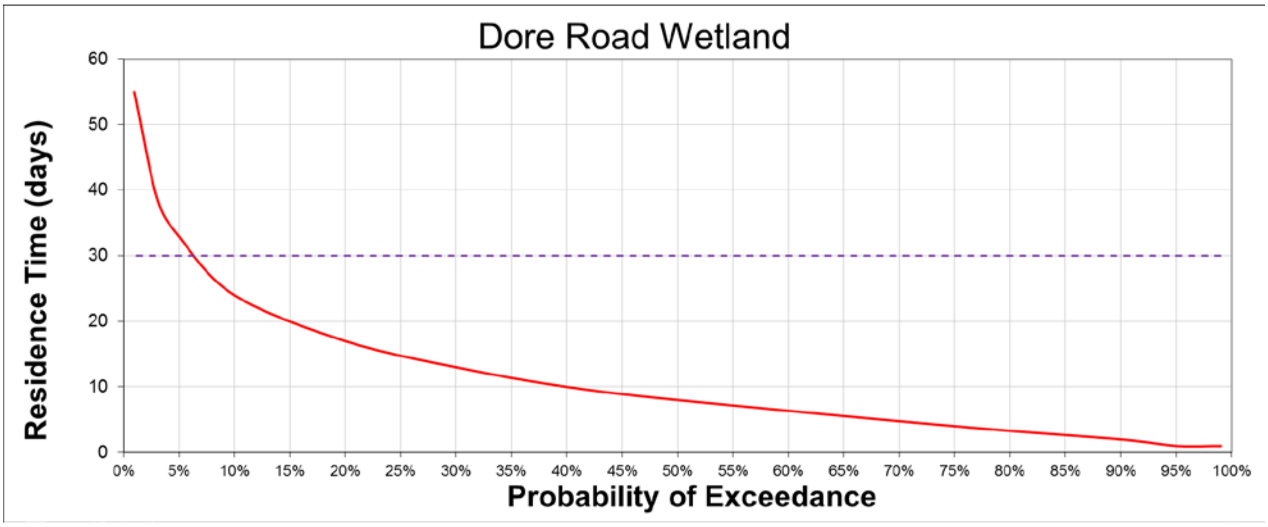
SWS CHECK: *1603_Dore_Rd_INNUNDATION_Frequency_V8_3Nov16.xlsx & Dore_Rd_Turn_3h_3Nov16.csv*

Appendix H – Wetland Turnover Analysis

It is also prudent to conduct a Turnover Analysis on the wetland system to ensure that in summer, the wetland turnover or “flushes” itself enough so as to not encourage algal growth in the water body.

WSUD Engineering Procedures: Stormwater (2005) recommends for this region of Victoria (20°C summer water temperature) that the 20th percentile residence time should not exceed 30 days. i.e. the water body flushes itself quicker than 30 days 80% (or more) of the time.

Using the MUSIC model described in Section 6 and Appendix G, SWS has completed a turnover analysis and obtained the following results.



File: 1603_Dore_Rd_Turnover_V8_3Nov16.xlxm

Clearly, the Dore Road Wetland system meets the criteria as its 20th percentile residence time is 17 days and it has a 30 days’ residence time around only 7% of the time.

Appendix I – 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 a previous 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 Dore Road 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.

Appendix J – Swale Concept Design

The calculations of the swale proposed to treat and convey the flows originating from the residential development to the west of the north-south gas line is shown below.

File: 1603_Dore_Rd_Swale_Calcs_3Nov16.xlsx

Swale Capacity: Mannings

Capacity of a trapozoid (>4.6 m^3/s)			
Slope =		0.004	m/m
n =		0.055	
x =		8	(1V in xH)
base width=		17	m
height=		0.4	m
Top Width =		23.4	m
Wetted perimeter =		23.45	m
Area =		8.08	m^2
Hyd radius =		0.344566	m
Q =		4.6	m^3/s
V =		0.6	m/s
		OK	
Note: Slope is at the residential outfall			

The swale alignment and location is as shown in Figure 7 below. Flows from the modelling described in Section 4.2 have been generated at three locations along the swale as detailed below:

Identifier	Chainage (m)	1% AEP Flow (m^3/s)		
FP1	793.8	1.0		
FP2	538	4.6		
FP3	248.2	2.8		
Swale Outlet	30.3	2.8		
File: 1603_Dore_Rd_DSS_PostDev_100Yr_Outflow_V8_27Oct16.cat				

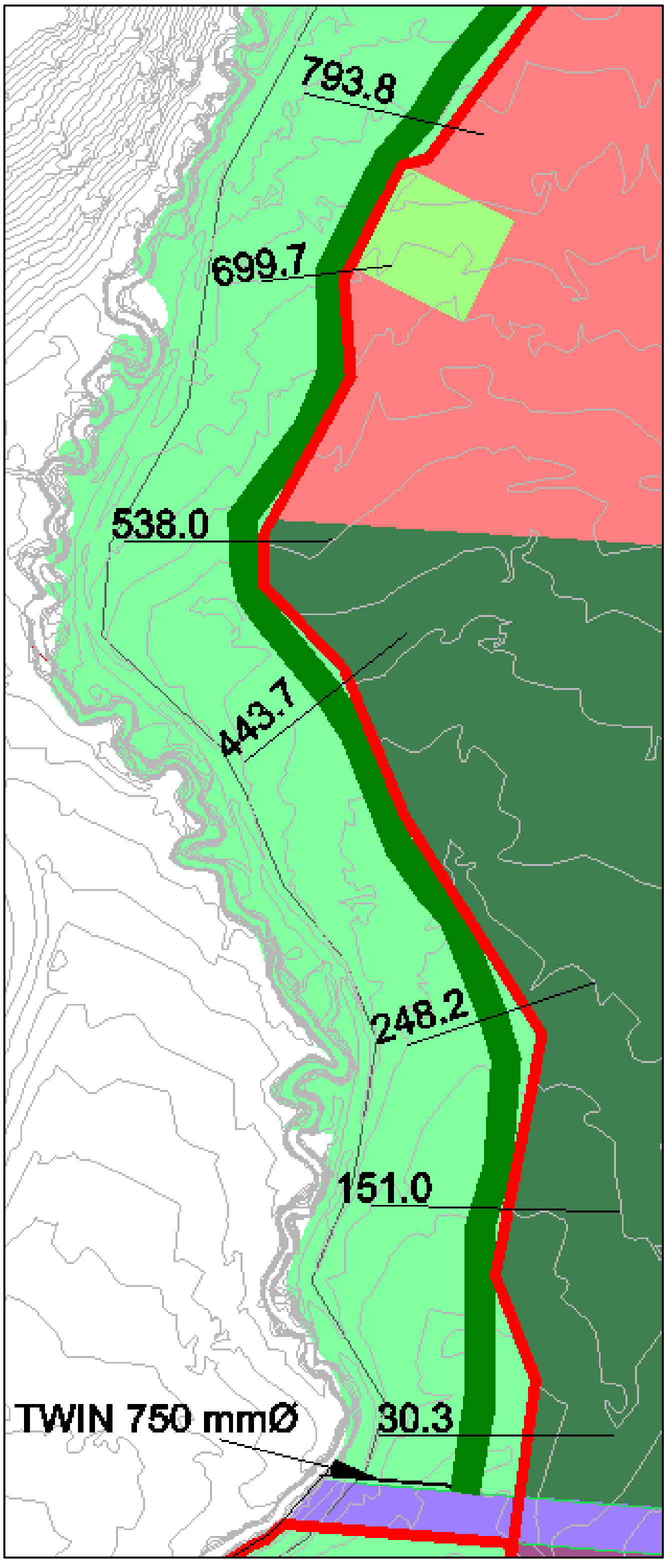


Figure 7 **Swale Alignment and Location**

In order to obtain appropriate cover on top the incoming pipes from the residential development (FP2), the swale has a varying slope along its length. Figure 8 below shows the design longitudinal section of the swale. The depth at FP2 has been assumed at 0.975 m (0.6m cover + pipe size allowance). It is assumed a small amount of fill may be required on the residential development to ensure that appropriate cover is met.

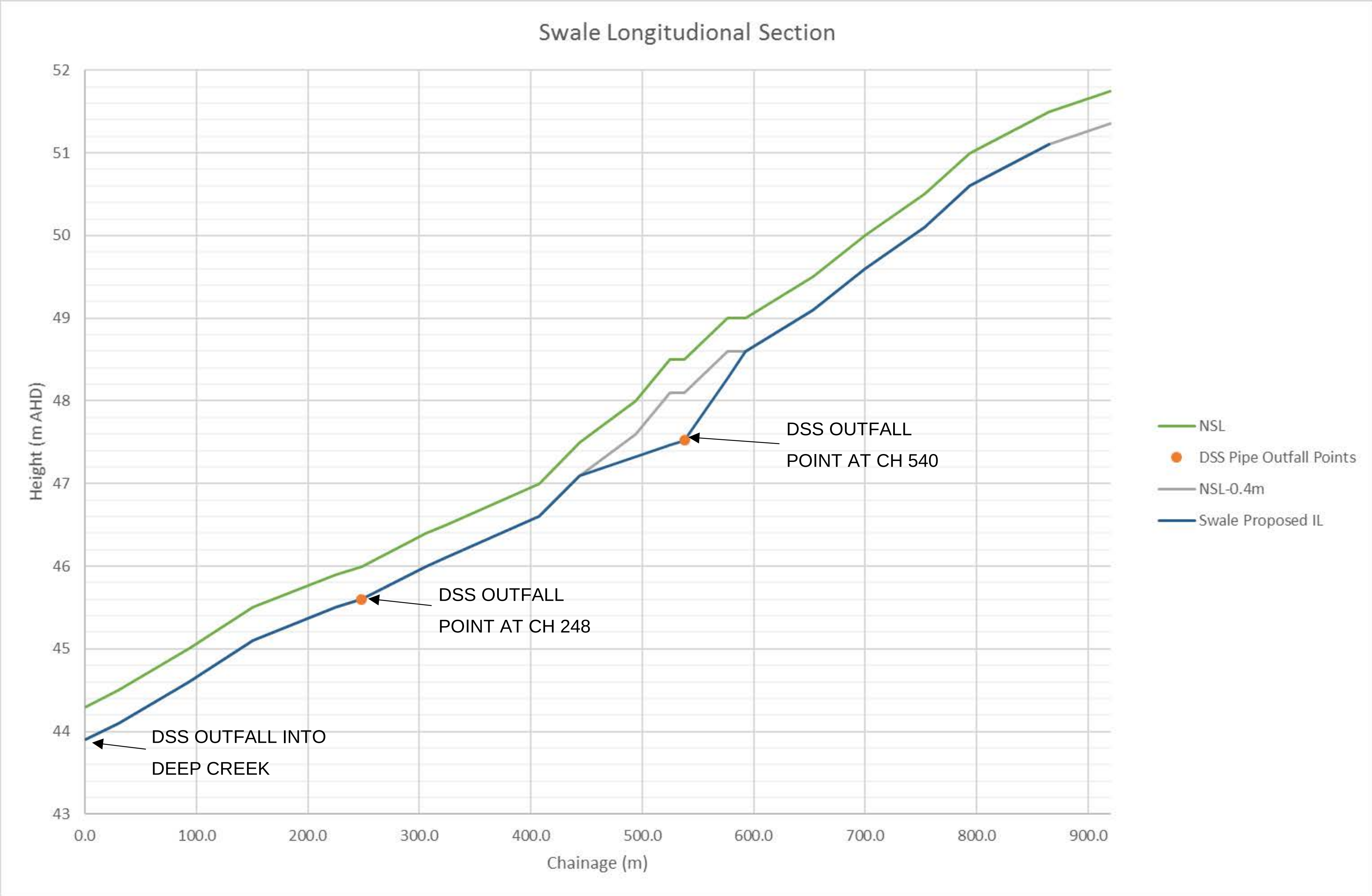


Figure 8 **Swale Longitudinal Section**

The key advantage of the swale is that is minimises fill on the residential development as shown in Figure 9 and Figure 10. The swale provides and outfall point for point for the development while also re-aligning the overland flow path within the 100m offset from Deep Creek, hence maximising the developable land while also reducing fill requirements.

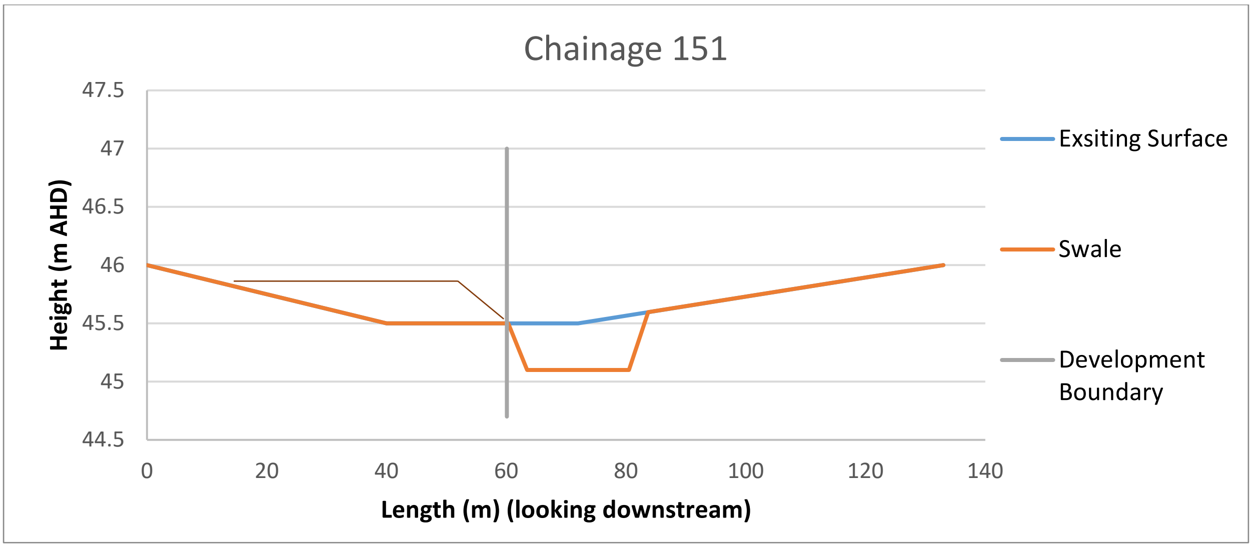


Figure 9 Cross Section at Chainage 151

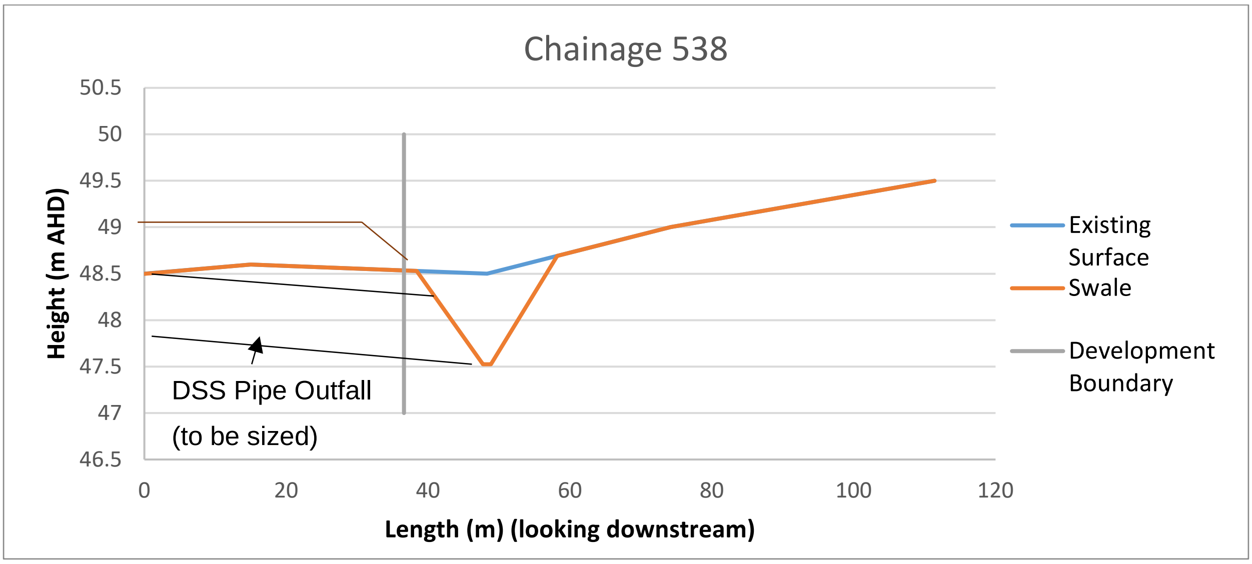


Figure 10 Cross Section at Chainage 538 (Development Inlet)

It should be noted that at chainage 538, the swale shape has been altered, cutting down from NSL at 1 in 8 batters to reach the designed obvert (0.975 m below NSL). This revised shape still has adequate capacity to contain the 1% AEP flows.

Initial sizing's indicate twin 750mmØ culverts are adequate to connect the swale to Deep Creek upstream of the gas easement.

Development to be filled 600 mm above the applicable Deep Creek flood level. The functional design of the swale system is to clearly define Deep Creek flood levels.

Appendix K – Constructed Wetlands Design Manual, Part A.2: Checklist

General				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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.	Functional Design Report: Section 6	☑	
GN2	The meteorological data in the conceptual modelling data or software (i.e. MUSIC) must be: • 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	Functional Design Report: Section 6	☑	
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.	Functional Design Report: Section 6	☑	
GN4	Peak design flows must be estimated in accordance with methods in Australian Rainfall and Runoff.	Functional Design Report: Section 4	☑	

Liner and Topsoil				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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.	No Exfiltration modelled. See Appendix I for discussion	☑	
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 DORE/SWS/1	☑	
Landscape Design Structures				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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.	Not in Scope of Works but 10Yr ARI Water Level Clearly Shown for Future Works	☑	
LDS3	Boardwalks or viewing platforms are not permitted over sediment ponds.	Not in Scope of Works	☑	
Edge Treatment				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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).	Functional Design Drawing: DORE/SWS/1	☑	
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.	Functional Design Drawing: DORE/SWS/1	☑	
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.	Functional Design Drawing: DORE/SWS/1	☑	

Maintenance Provisions				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
MN1	Sediment ponds must be able to be drained whilst maintaining the macrophyte zone water level at normal water level.	Design is Based on MWC Standard Drawing WG010(B)		☑
MN2	All parts of the base of a sediment pond must be accessible: • 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	Shown in Functional Design Drawing: DORE/SWS/1	☑	
MN5	Maintenance access ramps are required on all sediment ponds that cannot be 'edge cleaned'. The maintenance access ramp into a sediment pond must: • 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).	Shown in Functional Design Drawing: DORE/SWS/1	☑	
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: • 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.	Shown in Functional Design Drawing: DORE/SWS/1	☑	
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.) **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)	Adequate Space Provided in Functional Design Drawings: DORE/SWS/1	☑	
MN9	Dedicated sediment dewatering areas must be provided and: • 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.	Shown in Functional Design Drawings: DORE/SWS/1	☑	
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).	Adequate Space Provided in Functional Design Drawings: DORE/SWS/1	☑	

Sediment Pond				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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.	Design is Based on MWC Standard Drawing WG010(B)		☑
SP2	Sediment ponds must be located at each point stormwater enters the "wetland system" unless: • 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	Only one inlet to treatment system	☑	
SP3	Sediment ponds must be sized to: • 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).	Functional Design Report: Section 5, Appendix E	☑	
SP4	The sediment pond EDD must be ≤ 350 mm.	Functional Design Report: Section 5	☑	

Bypass				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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. **Note:** Refer to Melbourne Water Waterways Manual for channel design specifications when designing bypass routes.	Design is Online, Based on MWC Standard Drawing WG010(B)		☑

Inlets and Outlets				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
IO4	The connection between the sediment pond and macrophyte zone must be sized such that: • 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	Design is Based on MWC Standard Drawing WG010(B)		☑
IO6	The macrophyte zone controlled outlet must be configured so that: • 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	Functional Design Drawing: DORE/SWS/2	☑	
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).	Functional Design Drawing: DORE/SWS/1	☑	

Macrophyte Zone				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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).	Functional Design Report: Section 3, Functional Design Drawings: DORE/SWS/1	☑	
MZ2	The macrophyte zone EDD must be ≤ 350 mm.	Functional Design Report: Section 3	☑	
MZ3	Macrophyte zones must be located offline from all waterways and drains (i.e. there must be a bypass route around the macrophyte zone).	Functional Design Drawing: DORE/SWS/1	☑	
MZ4	The length of the macrophyte zone must be ≥ four times the average width of the macrophyte zone.	Functional Design Drawing: DORE/SWS/1	☑	
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.	Functional Design Drawing: DORE/SWS/1	☑	
MZ6	The macrophyte zone outlet must be located at the opposite end of the macrophyte zone to the inlet(s).	Functional Design Drawing: DORE/SWS/1	☑	
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.	Functional Design Drawing: DORE/SWS/1	☑	
MZ8	Inlet and outlet pools must be ≤ 1.5 m depth.	Functional Design Report: Section 3, Functional Design Drawings: DORE/SWS/1	☑	
MZ9	Intermediate pools (between the inlet and outlet pool) must be ≤ 1.2 m deep.	Functional Design Report: Section 3, Functional Design Drawings: DORE/SWS/1	☑	
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.	Appendix F shows calculations	☑	
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.	Functional Design Report: Appendix G	☑	
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.	Functional Design Drawing: DORE/SWS/1	☑	

Vegetation				
Criteria	*Description*	*Reference*	*Deemed To Comply Criteria Met*	*Alternative Design Approach Met*
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.	Functional Design Report: Section 3	☑	
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.	N.A. No Open Water Area in excess of 20%. Functional Design Report: Section 3	☑	
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 DORE/SWS/1. Species to be Specified at Detailed Design Stage from Appendix G	☑	
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 DORE/SWS/1. Species to be Specified at Detailed Design Stage from Appendix G	☑	
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 DORE/SWS/1. Species to be Specified at Detailed Design Stage from Appendix G	☑	
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 Functional Design Report: Appendix E. Species to be Specified at Detailed Design Stage	☑	
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 DORE/SWS/1. Species to be Specified at Detailed Design Stage from Appendix G	☑	
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).	N.A. No Harvesting	☑	

Appendix L - RORB .cat Files

L.1 Pre-development .cat file

*File: 1603_Dore_Rd_DSS_PreDev_7Oct16.*cat

```
1603_Dore_Rd_DSS_PREDEV
C RORB MODEL DELVELOPED BY MICHAEL MAG
C Project Engineer, Stormy Water Solutions
C DATE: 7/10/16
C AREA TOTAL = 1.279 km^2
C kc= from DVA formula Kc = 1.752 = 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: Dore Rd
0
1,1,0.476,-99          1     A
2,1,0.385,-99          2     B
2,1,0.265,-99          3     C
3
1,1,0.432,-99          4     D
2,1,0.341,-99          5     E
4
2,1,0.298,-99          6     F
3
1,1,0.245,-99          7     G
4
2,1,0.305,-99          8     H
3
1,1,0.377,-99          9     I
2,1,0.376,-99          10    J
3
1,1,0.263,-99          11    K
4
2,1,0.291,-99          12    L
3
1,1,0.287,-99          13    M
4
5,1,0.092,-99          14
3
1,1,0.100,-99          15    N
4
5,1,0.213,-99          16
4
2,1,0.111,-99          17    O
7
OUTLET
0
C SUB-CATCHMENT AREAS (KM^2)
0.12006,0.09522,0.05273,0.15219,0.10119,
0.06445,0.05113,0.05283,0.13745,0.10751,
0.05606,0.05514,0.13249,0.04933,0.05115,-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,-99
```

L.2 Post-development .cat file

File:

1603_Dore_Rd_DSS_PostDev_100Yr_FLOOD_LV

*L_V8_27Oct16.*cat

```
1603_Dore_Rd_DSS_POSTDEV_FLOOD_LEVEL_SSD_
V8_100YR
C RORB MODEL DELVELOPED BY MICHAEL MAG
C Project Engineer, Stormy Water Solutions
C DATE: 27/10/16
C RB DIFFERNT FOR ALL EVENTS DUE TO TWL IN
DEEP CREEK
C THIS MODEL SIMULATES DEEP CREEK HIGH (42.3m
AHD)
C I.E. CRITICAL FLOOD LEVEL IN BASIN
C AREA TOTAL = 1.280 km^2
C kc= from DVA formula Kc = 1.753 = 1.53*(AREA)^0.55
C m=0.8
C RoC 100yr = 0.60
C RoC 50yr = 0.55
C RoC 20yr = 0.50
C RoC 10yr = 0.40
C RoC 5yr = 0.30
C RoC 2yr = 0.25
C RoC 1yr = 0.20
C Initial loss = 10mm
C IFD Data location: Dore Rd
0
1,3,0.267,8.61,-99          1     A
3
1,3,0.190,10.02,-99         2     B
3
1,3,0.277,4.69,-99          3     C
4
4
5,3,0.103,4.85,-99          4
2,3,0.242,4.96,-99          5     D
3
1,3,0.179,6.15,-99          6     E
4
2,3,0.126,3.97,-99          7     F
3
1,3,0.273,2.56,-99          8     G
4
5,3,0.088,1.14,-99          9
3
1,3,0.109,12.83,-99         10    H
2,3,0.110,5.45,-99          11    I
4
5,3,0.128,0.78,-99          12
3
1,3,0.189,7.41,-99          13    J
2,3,0.104,5.77,-99          14    K
4
5,3,0.269,1.12,-99          15
3
1,3,0.127,6.30,-99          16    L
4
3
1,3,0.365,4.94,-99          17    M
3
1,3,0.100,8.04,-99          18    N
4
5,3,0.212,6.13,-99          19
4
7
GAS EASEMENT
5,3,0.036,0.69,-99          20
3
1,2,0.170,6.62,-99          21    O
```

```
4
3
1,2,0.126,0.33,-99          22      P
4
5,3,0.041,0.61,-99          23
3
1,3,0.071,4.93,-99          24      Q
3
1,3,0.121,0.41,-99          25      R
4
4
5,3,0.316,0.33,-99          26
2,4,0.235,-99               27      S
16
100YR DEEP CRK LVL = 42.3 RB
1,0,5
0,0,6895,0.025,9154,0.10,
54788,0.20,57930,0.62,-99
1,5
40.15,0,40.5,6895,40.6,9154,
42.4,54788,42.5,57930,-99
3
1,3,0.593,2.28,-99          28      T
3
1,3,0.145,0.69,-99          29      U
2,3,0.065,1.55,-99          30      V
4
5,3,0.149,0.34,-99          31
3
1,3,0.158,0.95,-99          32      W
2,3,0.061,1.64,-99          33      X
4
5,3,0.099,0.33,-99          34
3
1,1,0.267,-99               35      Y
2,1,0.268,-99               36      Z
2,1,0.248,-99               37      AA
2,1,0.220,-99               38      AB
7
FP1
2,1,0.164,-99               39      AC
2,1,0.076,-99               40      AD
4
7
FP2
5,1,0.188,-99               41
2,1,0.120,-99               42      AE
3
1,2,0.150,1.00,-99          43      AF
3
1,2,0.184,0.82,-99          44      AG
4
4
7
FP3
5,1,0.131,-99               45
2,1,0.113,-99               46      AH
3
1,2,0.105,0.48,-99          47      AI
3
1,2,0.182,0.33,-99          48      AJ
4
4
5,3,0.130,1.15,-99          49
5,4,0.010,-99               50
7
FROM SWALE
4
7
OUTFLOW
0
C SUB-CATCHMENT AREAS (KM^2)
0.05002,0.06080,0.05046,0.04268,0.01927,
0.03495,0.03038,0.02773,0.03113,0.02610,
0.03039,0.04489,0.03465,0.02493,0.00826,
0.00607,0.01591,0.00885,0.06646,0.12042,
0.03359,0.02709,0.02933,0.01988,0.08305,
0.06202,0.05325,0.05140,0.01824,0.01056,
0.02360,0.03270,0.03450,0.02022,0.01746,
0.02918,-99
C IMPERVIOUS FRACTION
1,0.074,0.287,0.359,0.497,0.482,
0.750,0.750,0.471,0.750,0.484,
0.750,0.750,0.478,0.574,0.050,
0.050,0.470,0.800,0.026,0.077,
0.740,0.736,0.641,0.703,0.050,
0.050,0.050,0.050,0.000,0.000,
0.000,0.100,0.100,0.000,0.100,
0.100,-99
```