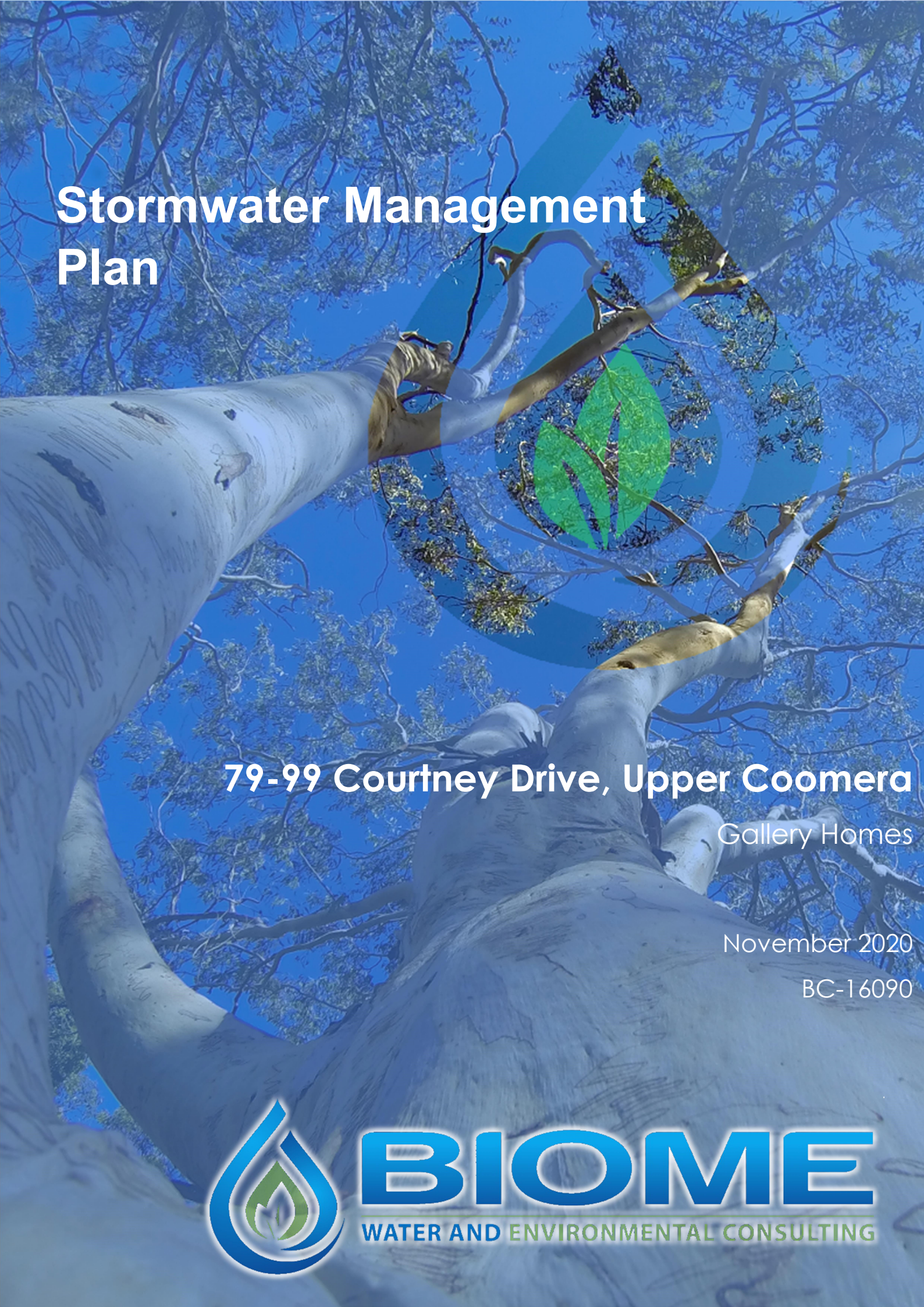
Stormwater Management Plan



79-99 Courtney Drive, Upper Coomera

Gallery Homes

November 2020

BC-16090

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Executive Summary

This conceptual stormwater management plan has been prepared so as to be considered as part of a Development Application for a Reconfiguration of Lot over Lots 188-192 on RP173728, at 79-99 Courtney Drive, Upper Coomera (hereafter referred to as 'the subject site').

The report has been prepared on behalf of Gallery Homes and presents a comprehensive review of available Stormwater Quality Improvement Devices (SQID) and stormwater Best Management Practices (BMP) to ensure that the proposed development adequately addresses the management of stormwater quantity and quality during both the construction and operational phases of the development.

This version of the report (Version 3) has been prepared to respond to layout changes required by City of Gold Coast.

In order to address the management of stormwater quantity and quality, a total of four (4) stormwater treatment basins have been proposed. Three (3) of these basins (Basins A, B and D) will be combined bioretention/detention basin for quality and quantity management. The remaining Basin C will only be required for treatment of pollutant loads, as catchment reductions in the post-development case have resulted in the reduction of peak discharge.

The results of xpstorm modelling indicate that the inclusion of the three (3) proposed detention basins will mitigate the expected increases in peak discharge at PD-A, PD-B, and PD-D during all modelled AEP events.

All internal catchments will be directed to bioretention measure for quality treatment. *MUSIC v6* modelling indicates that reduction target for TSS, TP, TN and gross pollutants of 80%, 60%, 45% and 90% respectively can be achieved for the rainfall data set simulated,

In order to simulate pre- and post-development flooding across the site the 1D - 2D hydrodynamic flood model TUFLOW has been utilised to route stormwater through the upper reaches of Yaun Creek and the other minor streams. Spatial analysis of the peak pre and post-development flooding indicates that the proposed earthworks required to create the subdivision will not cause any flood increase or nuisance to neighbouring properties.

The level of detail provided within this report is suitable for development assessment only and should not be relied upon for construction purposes. A Detailed Stormwater Management Plan containing detailed engineering designs will be needed to finalise the stormwater concepts presented in this report. Detailed design documentation should be prepared in conjunction with civil operational works.

This report has been reviewed by a Registered Professional Engineer of Queensland (RPEQ), and certification has been provided that if the design parameters set out in this report are included within the development:

- there should be no worsening in peak discharge as a result of the proposed development; and
- stormwater pollutant load reductions in accordance with best practice should be achieved.

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1 Introduction

This conceptual stormwater management plan has been prepared so as to be considered as part of a Development Application for a Reconfiguration of Lot over Lots 188-192 on RP173728, at 79-99 Courtney Drive, Upper Coomera (hereafter referred to as 'the subject site').

The report has been prepared on behalf of Gallery Homes and presents a comprehensive review of available Stormwater Quality Improvement Devices (SQID) and stormwater Best Management Practices (BMP) to ensure that the proposed development adequately addresses the management of stormwater quantity and quality during both the construction and operational phases of the development.

This version of the report (Version 3) has been prepared to respond to layout changes required by City of Gold Coast.

It is noted that the assessments relating to the 'existing' and 'pre-development' scenarios have not been altered from the original version (Version 1 dated May 2019) of the report.

The level of detail provided within this report is suitable for development assessment only and should not be relied upon for construction purposes. A Detailed Stormwater Management Plan containing detailed engineering designs will be needed to finalise the stormwater concepts presented in this report. Detailed design documentation should be prepared in conjunction with civil operational works.

1.1 Objectives

The objective of this report is to present practical conceptual stormwater designs which can be integrated into the development proposal so as to ensure that the development minimises potential impacts on downstream receiving environments during the operational phases of development. In addition, the report aims to address the requirements of the City of Gold Coast – Healthy Waters Code (Appendix A).

1.1.1 Operational Phase Objectives

The overarching objective for stormwater management during the operational phase of the development is to ensure that development does not cause an unacceptable impact or nuisance which could result in actionable damage to downstream properties and receiving environments. The following objectives are to be achieved.

Operational Phase Objective (Quantity)	Maintain a Lawful Point of Discharge (LPD) for all site catchments in accordance with the Queensland Urban Drainage Manual (2016).
Operational Phase Objective (Quality)	Stormwater discharged from the site achieves the specified load based reduction targets in accordance Queensland Water Quality Guidelines (2009). For the development site relevant targets are TSS 80%, TP 60%, and TN 45%.

2 Site Location

The subject site is located at 79-99 Courtney Drive, Upper Coomera on Lots 188-192 on RP173728 and is located within the City of Gold Coast Local Government Area. The site comprises of five allotments with a total land area of 18.849 ha. The size of each allotment is included within Table 2.1 with the location and extent of the site shown on Figure 2.1.

Table 2.1 Allotment Details

Street Number	Lot Plan Number	Lot Size (ha)
79	188 RP173728	3.079
89	189 RP173728	3.034
91	190 RP173728	3.095
97	191 RP173728	3.739
99	192 RP173728	5.902



Figure 2.1 Site Location (Google Earth, image date 2016)

3 Existing Case - Hydrological Assessment

xpstorm was utilised to assess the site's existing hydrology and generate hydrographs to represent the stormwater flows expected at the site's existing points of discharge (PD's). Modelling was based on existing catchment areas and surface characteristics. Peak discharge rates for nominated Annual Exceedance Probabilities (AEPs) were derived from the output hydrographs. The following sections detail the input parameters used in the *xpstorm* modelling. Catchments have been delineated on Drawing 201 – Appendix A.

3.1 Rainfall

The hydrologic analysis undertaken in this report will rely on Australian Rainfall and Runoff (ARR) temporal pattern and IFD data obtained for the site from the BOM (Table 3.1).

Table 3.1 Adopted Intensity Frequency Data (mm/hr)

Storm Duration	Annual Exceedance Probability (AEP)					
	0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
5 minute	140.00	176.00	204.00	234.00	276.00	308.00
10 minute	115.00	143.00	164.00	188.00	218.00	240.00
15 minute	97.30	121.00	139.00	158.00	183.00	201.00
20 minute	84.60	105.00	121.00	138.00	160.00	176.00
25 minute	75.20	93.40	108.00	123.00	143.00	158.00
30 minute	67.80	84.40	97.40	112.00	130.00	144.00
45 minute	53.10	66.50	77.10	88.90	105.00	117.00
60 minute	44.20	55.70	64.90	75.20	89.40	100.00
90 minute	34.00	43.20	50.80	59.30	71.30	80.90
120 minute	28.20	36.10	42.70	50.10	60.70	69.30
180 minute	21.70	28.20	33.50	39.70	48.50	55.70

3.2 Existing Case - External Catchments

Two (2) upstream catchments direct flows onto the subject site.

- Ext 1 - discharges flows onto the south western corner of the site as channel flow via a small gully; and
- Ext 2 - directs flows onto the subject site via channel flow. Catchment flows are conveyed across the south eastern corner of the site and to the east.

  NOTE: The conveyance requirements for this catchment through the subject site have been analysis as part of a 2D assessment within Section 7 of this report. As such no further reference is made to this catchment within following 1D hydrological section.

Table 3.2 below presents a summary of the external catchment characteristics.

Table 3.2 Existing Case – External Catchment Characteristic

Catchment ID		Area (ha)	Cover (%)
External	Ext-1	2.26	100 - Dense Forest

3.3 Existing Case - Internal Catchments

The subject site has been divided into four (4) internal catchment areas:

- A – discharges flows primarily as channel flow into Yaun Creek on the western edge of the site;
- B – flows are directed to the north and into Yaun Creek at the site's northern boundary;
- C – discharges catchment flows into the Mackellar Drive road reserve; and
- D – directs flows into a tributary of Yaun Creek at the site south eastern corner.

The characteristic of the site's catchments are detailed in Table 3.3 below and have been delineated on Drawing 201 – Appendix A.

Table 3.3 Existing Case – Catchment Characteristic

Catchment ID		Area (ha)	Cover (%)	Discharge Condition	PD ID
Internal	A	8.6	10 – Dense Forest 90 – Cleared pasture	Channel - Sheet	PD-A
	B	4.4	100 – Cleared pasture with isolated corpses	Sheet	PD-B
	C	2.9	100 – Cleared pasture with isolated corpses	Channel	PD-C
	D	1.94	100 – Cleared pasture with isolated corpses	Sheet – Channel	PD-D

3.4 Existing Case - *xpstorm* Runoff

The "Laurenson" routing method was applied to *xpstorm* for hydrological calculation and hydrograph generation. The contributing catchment was split into pervious (with 0% impervious fraction) and impervious (with 100% impervious fraction) areas. Adopted parameters for the Laurenson routing method include a Manning's roughness coefficient (n) for impervious and pervious areas respectively. Infiltration uniform losses have been applied to the hydrologic model.

The tables below contain the modelling parameters relied upon and present the resulting peak discharges expected for each catchment.

Table 3.4 *xpstorm* Catchment Details – Existing Case

Catchment ID		Impervious Area		Pervious Area	
		Area (ha)	Slope (%)	Area (ha)	Slope (%)
External	Ext-1	-	-	2.26	25
Internal	A	-	-	8.6	8.5
	B	-	-	4.4	11
	C	-	-	2.9	14
	D	-	-	1.94	16

Table 3.5 *xpstorm* Rainfall Multipliers Applied to Temporal Patterns (mm)

Storm Duration	Annual Exceedance Probability (AEP)					
	0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
5 minute	11.67	14.67	17.00	19.50	23.00	25.67
10 minute	19.17	23.83	27.33	31.33	36.33	40.00
15 minute	24.33	30.25	34.75	39.50	45.75	50.25
20 minute	28.20	35.00	40.33	46.00	53.33	58.67
25 minute	31.33	38.92	45.00	51.25	59.58	65.83
30 minute	33.90	42.20	48.70	56.00	65.00	72.00
45 minute	39.83	49.88	57.83	66.68	78.75	87.75
60 minute	44.20	55.70	64.90	75.20	89.40	100.00
90 minute	51.00	64.80	76.20	88.95	106.95	121.35
120 minute	56.40	72.20	85.40	100.20	121.40	138.60
180 minute	65.10	84.60	100.50	119.10	145.50	167.10

Table 3.6 *xpstorm* Adopted Initial and Continuing Losses – Existing Case

Impervious Area			Pervious Area			
IL (mm)	CL (mm/hr)	Manning's (n)	AEP	CL (mm/hr)	IL (mm)	Manning's (n)
0	0	0.014	0.393-0.049	2	15	0.037
			0.020-0.010	2	5	

Table 3.7 *xpstorm* Peak Discharge (m^3/s) – Existing Case

Point of Discharge	Annual Exceedance Probability (AEP)					
	0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
Ext-1	0.60	0.78	0.93	1.09	1.43	1.58
PD-A	1.70	2.55	3.36	4.10	4.53	5.05
PD-B	0.83	1.21	1.53	1.83	2.08	2.30
PD-C	0.65	0.90	1.08	1.28	1.56	1.75
PD-D*	0.51	0.70	0.82	1.01	1.15	1.29

*Peak discharge from Catchment D to PD-D. Does not include flows from Ext-2.

3.5 Existing Case - Model Validation (Rational Method)

For validation purposes, peak discharge values obtained for nominated storm events using xpstorm were compared to those calculated using Rational Method and are shown in Table 3.8 below. Rational calculations were prepared in accordance with QUDM 2013 Section 4, and are detailed within Appendix C.

Table 3.8 Peak Flow Validation *xpstorm* vs Rational – Existing Case

Catchment & Point of Discharge ID			*xpstorm* (m^3/s)	Rational (m^3/s)	Difference (%)
External	PD-A	Ext-1	1.58	1.52	3.8
Internal		A	3.97	4.76	-19.9
	PD-B		2.30	2.62	-13.9
	PD-C		1.75	1.82	-4.0
	PD-D		1.29	1.27	1.55

The peak inflows generated using both hydrological methods are within 20% of each other for the 0.01 AEP. The modelling is therefore considered to be appropriately validated.

4 Proposed Development

The development involves reconfiguring the existing site into 196 residential allotments (Figure 4.1). The development will also include:

- An internal road network with pedestrian footpath;
- Conservation areas (Environmental Reserve, public open space) to protect existing ecologically significant areas;
- A constructed naturalized waterway;
- Open space recreational areas;
- Four (4) stormwater treatment areas and five (5) drainage reserves; and
- Rehabilitated waterways incorporated into public open space

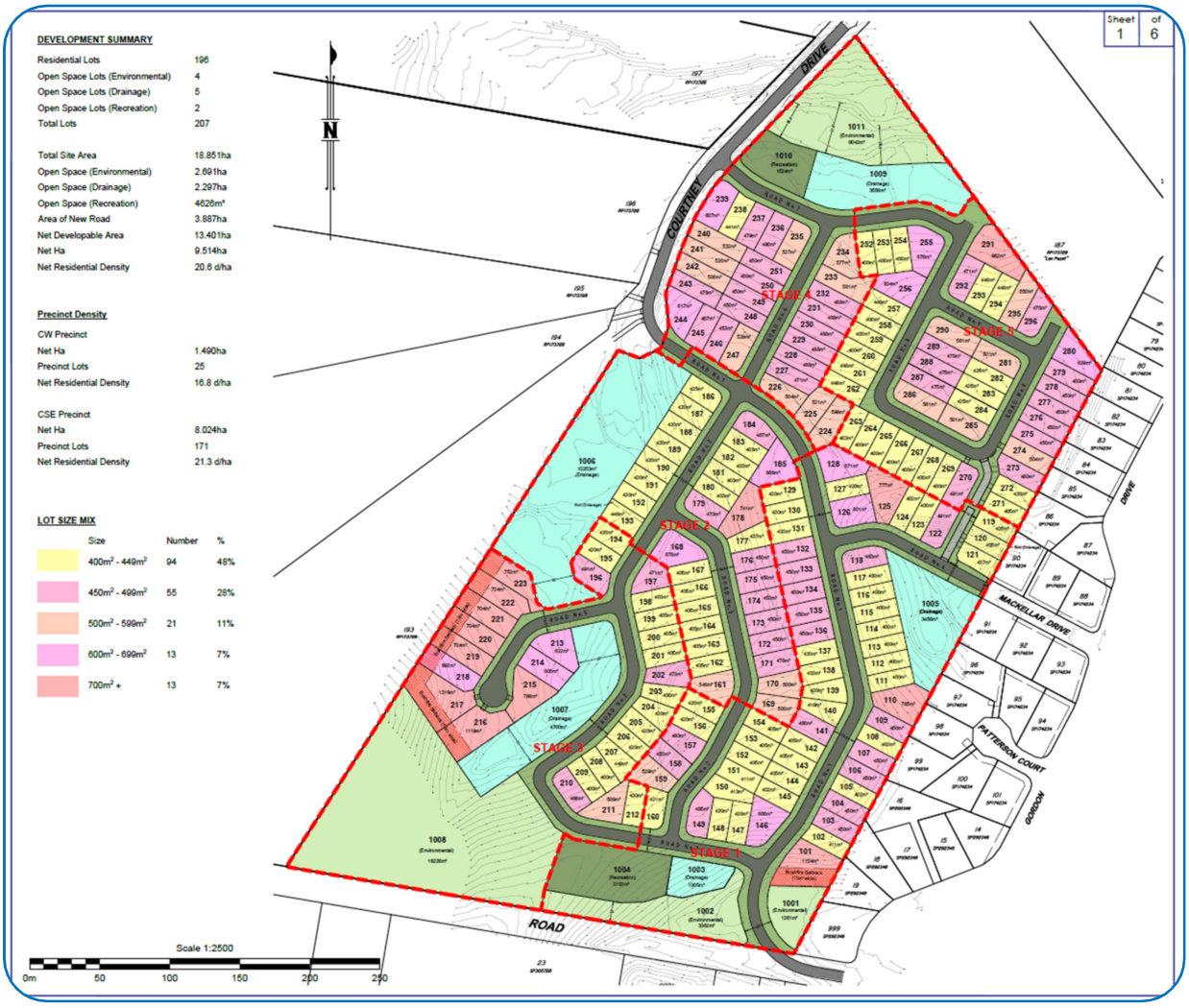


Figure 4.1 Proposed Development Layout (Bennett and Bennett)

5 Developed Case - Hydrological Assessment

xpstorm was utilised to assess the site's developed hydrology and generate hydrographs and peak discharge rates at each point of discharge (PD). The following sections detail the input parameters used in the *xpstorm* modelling. Catchments have been delineated on Drawing - 201 - Appendix A.

5.1 Developed Case - External Catchments

In the developed case the external catchment will remain unchanged (refer to Section 3.1 above). NOTE: The conveyance requirements for Ext-2 have been analysed as part of a 2D assessment within Section 8 of this report.

5.2 Developed Case - Internal Catchments

In the developed case the site will undergo substantial earthworks to profile the development area. Underground piped drainage will be installed to collect minor stormwater flows, with major flows conveyed via an internal road network. Six (6) major post development catchments have been delineated based on the preliminary civil designs by Mortons Urban Solutions. Catchment A has been further subdivided to recognise an area of environmental reserve that will be maintained (A1). The sites post development catchments are described as follows:

- A1 – Environmental Reserve which discharges flows primarily as channel flow into the natural channel/ waterway in A2a;
- A2a & A2b – residential allotments- discharges flows primarily as channel flow into Yaun Creek on the western edge of the site;
- B – residential allotments -flows are directed to the north and into Yaun Creek at the site's northern boundary;
- C – residential allotments -discharges catchment flows into the Mackellar Drive road reserve; and
- D – residential allotments - directs flows into a tributary of Yaun Creek at the site's south eastern corner.

The characteristic of the site's catchments are detailed in Table 5.1 below and have been delineated on Drawing 201 – Appendix A.

Table 5.1 Developed Case – Catchment Characteristic

Catchment ID		Area (ha)	Cover (%)	Discharge Condition	PD ID
Internal	A1	1.71	100 – Dense Forest	Channel	Into A2a
	A2a	3.78	100 – Res Dev	Weir – Pipe	PD-A
	A2b	3.01	100 – Res Dev	Weir – Pipe	PD-A
	B	5.39	100 – Res Dev	Weir – Pipe	PD-B
	C	2.37	100 – Res Dev	Pipe – Road	PD-C
	D	2.33	100 – Res Dev	Channel	PD-D

5.3 Developed Case - *xpstorm* Runoff

The "Laurenson" routing method was applied to *xpstorm* for hydrological calculation and hydrograph generation. The contributing catchment was split into pervious (with 0% impervious fraction) and impervious (with 100% impervious fraction) areas. Adopted parameters for the Laurenson routing method include a Manning's roughness coefficient (n) for impervious and pervious areas respectively. Infiltration uniform losses have been applied to the hydrologic model.

The tables below contain the modelling parameters relied upon and present the resulting peak discharges expected for each catchment. It is noted that areas with significantly different slopes (such as batter areas) have been modelled as separate sub catchments within the xpstorm hydrological node for the catchment.

Table 5.2 *xpstorm* Catchment Details – Developed Case

Catchment ID		Impervious Area				Pervious Area					
		Area (ha)	Slope (%)	Area (ha)	Slope (%)	Area (ha)	Slope (%)	Area (ha)	Slope (%)	Area (ha)	Slope (%)
External	Ext-1	-	-	-	-	2.26	25	-	-	-	-
Internal	A1	-	-	-	-	1.71	20	-	-	-	-
	A2a	1.83	5	-	-	1.22	5	0.45	25	0.28	8
	A2b	1.81	5	-	-	1.20	5	-	-	-	-
	B	3.04	6	0.02	6	2.02	6	0.31	20	-	-
	C	0.94	4	0.033	10	0.62	4	0.14	10	0.35	33
	D	0.89	4.5	0.15	6.8	0.59	4.5	0.28	15	0.42	17

Table 5.3 *xpstorm* Adopted Initial and Continuing Losses – Developed Case

Impervious Area			Pervious Area			
IL (mm)	CL (mm/hr)	Manning's (n)	AEP	CL (mm/hr)	IL (mm)	Manning's (n)
0	0	0.014	0.393-0.049	2	15	0.037
			0.020-0.010	2	5	

Table 5.4 *xpstorm* Peak Discharge (m^3/s)– Developed Case

Point of Discharge	Annual Exceedance Probability (AEP) – Peak Discharge					
	0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
Ext-1	0.60	0.78	0.93	1.09	1.43	1.58
PD-A	2.74	3.64	4.37	5.14	6.32	7.02
PD-B	1.39	1.80	2.18	2.57	3.01	3.42
PD-C	0.64	0.84	1.00	1.20	1.42	1.66
PD-D	0.62	0.82	0.98	1.15	1.41	1.59

5.3.1 Validation of Flows

Table 5.5 illustrates that the peak discharges generated using both methods are within 20% of each other for the 0.01 AEP. The modelling is therefore considered to be appropriately calibrated.

Table 5.5 Peak Flow Validation *xpstorm* vs Rational – Developed Case

Catchment & Point of Discharge ID			*xpstorm* (m^3/s)	Rational (m^3/s)	Difference (%)
External	PD-A	Ext-1	1.58	1.52	3.8
Internal		A1	1.19	1.16	2.6
		A2a	2.43	2.57	-5.2
		A2b	1.93	1.99	-2.9
	PD-B		3.42	3.70	-7.4
	PD-C		1.66	1.65	0.1
	PD-D		1.59	1.79	-11.0

5.3.2 Peak Discharge Comparison - Existing- vs Developed (Unmitigated)

Table 5.6 presents a comparison of the peak discharges expected at each PD. Results indicate increases in peak discharge are expected a PD-A, PD-B, and PD-D. Decreases in peak discharge to PD-C is expected due to catchment area and slope reductions therefore detention is not proposed to mitigate peak flows from the developed catchment.

Table 5.6 Peak Discharge Comparison (m^3/s) - Unmitigated

PD	Scenario	Contributing Catchment Area (ha)	Annual Exceedance Probability (AEP) – Peak Discharge					
			0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
A	Existing	10.86	1.70	2.55	3.36	4.10	4.53	5.05
	Developed (Unmitigated)	10.76	2.74	3.64	4.37	5.14	6.32	7.02
	Difference		+1.04	+1.09	+1.01	+1.04	+1.80	+1.97
B	Existing	4.40	0.83	1.21	1.53	1.83	2.08	2.30
	Developed (Unmitigated)	5.39	1.39	1.80	2.18	2.57	3.01	3.42
	Difference		+0.56	+0.59	+0.65	+0.74	+0.93	+1.12
C	Existing	2.89	0.65	0.90	1.08	1.28	1.56	1.75
	Developed (Unmitigated)	2.37	0.64	0.84	1.00	1.20	1.42	1.66
	Difference		-0.01	-0.06	-0.08	-0.08	-0.14	-0.09
D*	Existing	1.94	0.51	0.70	0.82	1.01	1.15	1.29
	Developed (Unmitigated)	2.33	0.62	0.82	0.98	1.15	1.41	1.59
	Difference		+0.11	+0.12	+0.16	+0.14	+0.26	+0.30

*Peak discharge from Catchment D to PD-D. Does not include flows from Ext-2.

6 Stormwater Management - Operational Phase

In order to ensure that the operational phase objectives outlined within Section 1.1 of this report can be achieved, a network of stormwater management measures are proposed for inclusion within the development.

To achieve these objectives, both external and internal flows will need to be adequately managed prior to discharge to the site's points of discharge.

6.1 Schematic Design Plan

With consideration given to the existing site characteristics, the proposed development configuration and the range of available stormwater management control measures, a set of conceptual design drawings have been prepared and are included as Appendix B. These drawings illustrate the layout and design requirements of each proposed mitigation measures.

6.1.1 External Drainage

- Ext-1 – Flows from this external catchment will continue to enter the subject site as per the pre-development case and into post-development Catchment A1 (Environmental Reserve). From the environmental reserve flows are to be conveyed through the development via a naturalised channel (located in Catchment A2a). The channel will discharge into a bunded portion of Basin A (denoted as 'by-pass pond' within the stormwater drawings contained in Appendix B) which is separate from the treatment area of the basin. This by-pass pond has been designed as a wet basin maintaining a depth of 1 m prior to discharge via an outlet headwall. Flows from the basin and pond will discharge along the site's western boundary at PD-A. The conveyance design for this catchment through the subject site has been included in Section 6.1.3 below. Additionally, the conveyance design has been further assessed as part of a 2D assessment within Section 8 of this report
- Ext 2 - The conveyance requirements for this catchment through the subject site have been analysed as part of a 2D assessment within Section 8 of this report.

6.1.2 Internal Drainage

The site's internal catchments will be developed for residential allotments which will be graded so as to allow minor stormwater flows to be collected within inter-allotment and road piped drainage systems with higher flows conveyed with road reserves. It is noted that a portion of the high flows above the pipe network design from Catchment A2a will also be directed to the natural channel and conveyed to Basin A prior to discharge at PD-A. Allowances for these flows have been included within the channel design.

Four (4) stormwater treatment basins have been proposed for the management of stormwater. Basins A, B, and D will be combined bioretention/detention basin for quality and quantity management. Basin C will only be required for treatment of pollutant loads, as catchment reductions in the post-development case have resulted a reduction in peak discharge at PD-C.

6.1.3 Natural Waterway Design

As part of discussions held with Council Officers, it has been agreed that due to the degraded nature of the exiting natural drainage line, a fabricated naturalised waterway can be incorporated into the development to convey flows from the Environmental Reserve (denoted as Catchment A1) and the external catchment Ext 1 to Basin A prior to discharge at PD-A. The aim of the naturalised waterway design has been to:

- mimic a natural waterway form and structure to provide opportunities for the establishment of fauna habitat; and
- provide suitable hydraulic conveyance of external and internal flows whilst minimising scour and maximising environmental values.

A preliminary design and assessment of the proposed channel in terms of conveyance capacity has been undertaken below. Further detailed designs of the channel in terms of specific rock size and vegetation lining are to be completed in accordance with the Brisbane City Council Natural Channel Design Guidelines (2003) as part of future operational works application.

Figures 6.1 - 6.4 below presents preliminary sizing of the natural channel at four different sections based on Manning's equation. Layouts and typical sections showing the channel design and peak water depths during the 1% AEP event are contained in DWG - 314 of the stormwater drawing set (Refer to Appendix B).

The base width, batter slopes and rock lining have been designed so as to maintain a d*V of below 0.6 m^2/s.

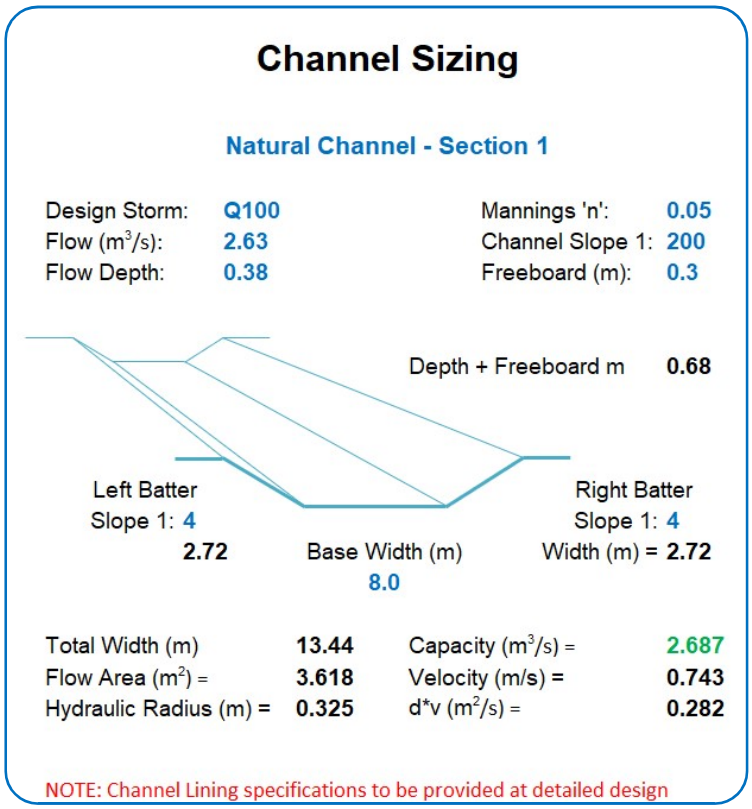


Figure 6.1 Natural Channel Section 1 – Manning's Assessment

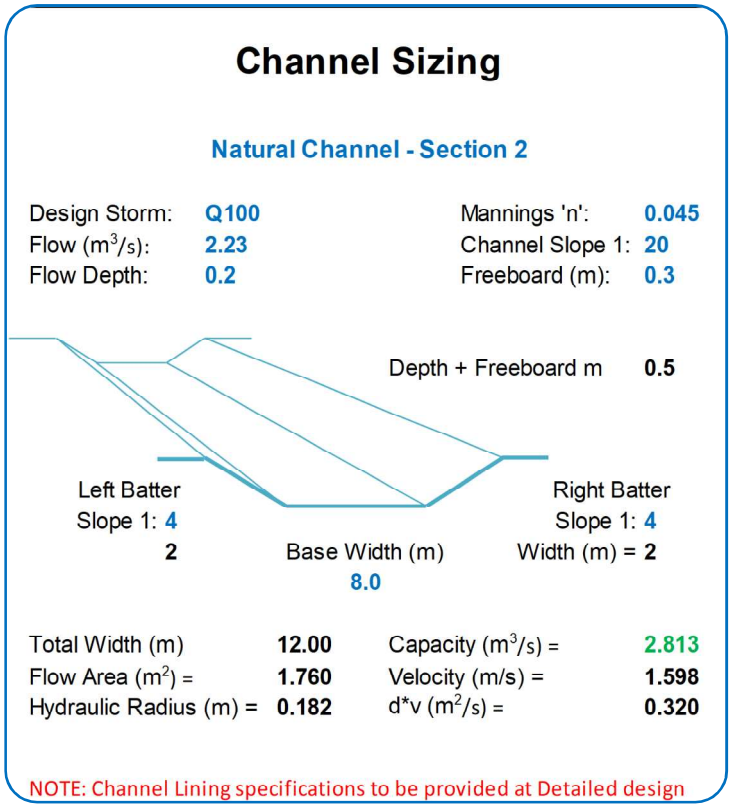


Figure 6.2 Natural Channel Section 2 – Manning's Assessment

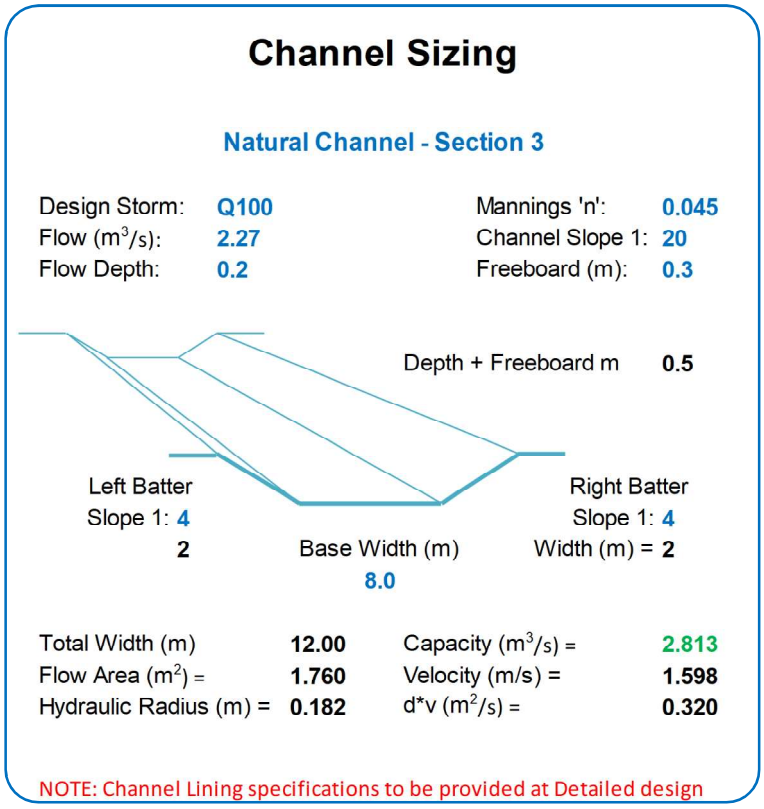


Figure 6.3 Natural Channel Section 3 – Manning's Assessment

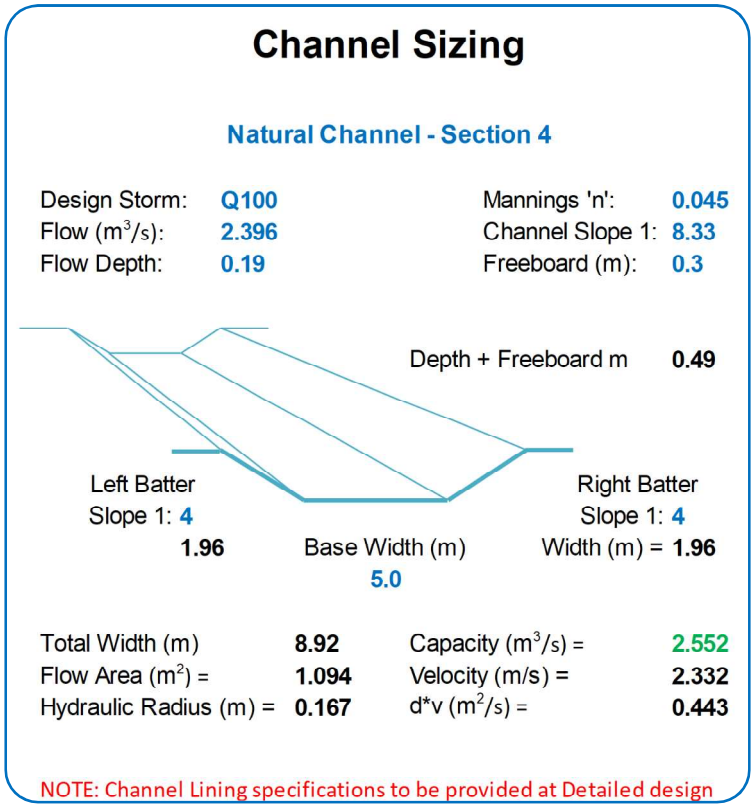


Figure 6.4 Natural Channel Section 4 – Manning's Assessment

6.1.4 Drainage Easements

There are two (2) easements nominated for drainage/conveyance purposes within the subject site. These can be seen on Drawing DWG-300 and DWG-314 (Appendix B) and include the following:

- Lot 193. This easement is over the stormwater pipe which discharges low flows from Catchment A2a to bioretention basin A2a. It is noted that high flows from this catchment will discharge directly to the natural channel; and
- Drainage Easement C which includes pipe and channel over Lots 119 to 121. This easement will convey flows to the existing stormwater infrastructure of Mackellar Drive to the east.

Figures 6.5 below presents preliminary sizing of the proposed channel within Drainage Easement C based on the Manning's equation. Further details of the proposed easements will be completed at detailed design.

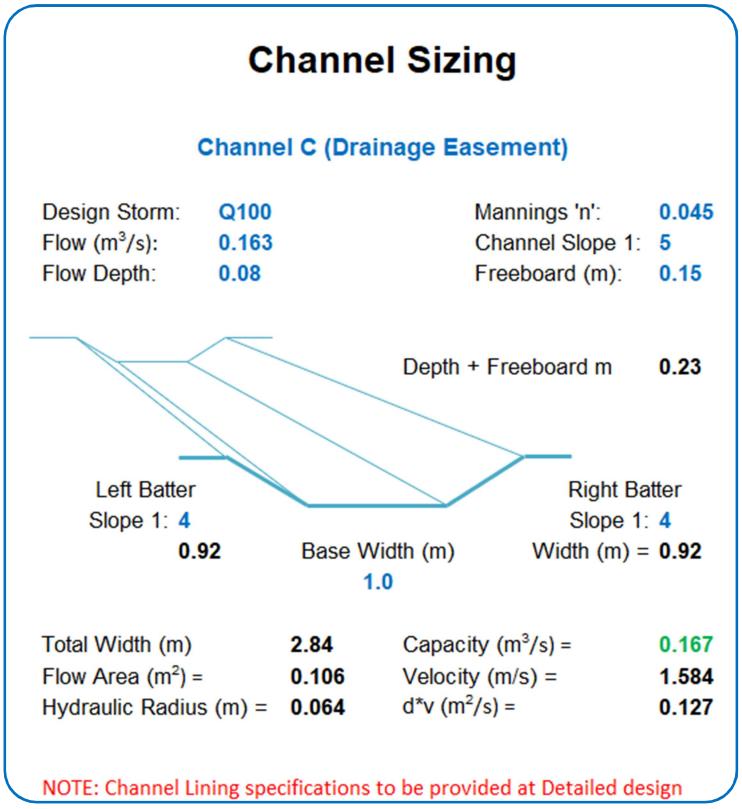


Figure 6.5 Channel C– Manning's Assessment

6.2 Quantity Control – Detention

In order to mitigate the increase in peak discharge expected at PD-A, PD-B and PD-D, detention is proposed within open detention basins. These basins will be located within open space to be dedicated to Council and will function to detain stormwater flows so as to ensure that there is no increase in discharge to downstream properties for all nominated AEP's.

NOTE: No detention is proposed within Catchment C as decreases in peak discharge are expected to PD-C as a result of catchment area and slope reductions in the development case.

6.2.1 *xpstorm* Modelling Paramaters

xpstorm has been relied upon to size the required detention measures and develop appropriate outlet configurations.

xpstorm requires a depth area relationship to be defined when modelling an onsite detention (OSD). An initial depth of 0.3 m has been applied within the modelling to reflect the bioretention extended detention requirements.

A summary of the total depth-area relationship applied to the storage node of the *xpstorm* model in the post-development (mitigated) scenario is contained in Table 6.1. The dimensions and proposed levels of the outlet structures used to mitigate the developed case peak discharge and achieve the required detention volume/s are detailed in Table 6.2.

Table 6.1 Depth Area Relationship

Detention ID	RL (m AHD)	Depth (m)	Surface Area (m^2)
Basin A*	44.50	0.00	1,111
	44.55	0.05	1,171
	44.60	0.10	1,232
	44.65	0.15	1,294
	44.70	0.20	1,356
	44.75	0.25	1,417
	44.80	0.30	1,480
	44.85	0.35	1,542
	44.90	0.40	1,605
	44.95	0.45	1,668
	45.00	0.50	1,731
	45.05	0.55	1,795
	45.10	0.60	1,858
	45.15	0.65	1,922
	45.20	0.70	1,987
	45.25	0.75	2,052
	45.30	0.80	2,117
	45.35	0.85	2,182
	45.40	0.90	2,248
	45.45	0.95	2,313
	45.50	1.00	2,380
	45.55	1.05	2,446
	45.60	1.10	2,513
	45.65	1.15	2,581
	45.70	1.20	2,649

	45.75	1.25	2,717
	45.80	1.30	2,786
	45.85	1.35	2,856
	45.90	1.40	2,927
	45.95	1.45	3,000
	46.00	1.50	3,505
Basin B	36.40	0.00	563
	36.45	0.05	594
	36.50	0.10	626
	36.55	0.15	657
	36.60	0.20	689
	36.65	0.25	721
	36.70	0.30	753
	36.75	0.35	788
	36.80	0.40	823
	36.85	0.45	858
	36.90	0.50	894
	36.95	0.55	930
	37.00	0.60	966
	37.05	0.65	1,002
	37.10	0.70	1,039
	37.15	0.75	1,075
	37.20	0.80	1,112
	37.25	0.85	1,150
	37.30	0.90	1,187
	37.35	0.95	1,225
	37.40	1.00	1,263
	37.45	1.05	1,301
	37.50	1.10	1,340
	37.55	1.15	1,379
	37.60	1.20	1,418
	37.65	1.25	1,457
	37.70	1.30	1,497
	37.75	1.35	1,537
	37.80	1.40	1,577
	37.85	1.45	1,621
	37.90	1.50	1,849
Basin D	57.00	0.00	159
	57.10	0.10	183
	57.20	0.20	208
	57.30	0.30	234
	57.40	0.40	263
	57.50	0.50	293
	57.60	0.60	324
	57.70	0.70	356
	57.80	0.80	389
	57.90	0.90	423
	58.00	1.00	535

*It is noted that for modelling purposes the depth vs area of Basin A has been split over two (2) storage nodes representing the pond area and the bioretention area which is separated via a 600mm bund.

Table 6.2 Outlet Structures

Detention ID	Pond Outlet	Outlet Riser & Pipe(s)	Weir
Basin A	2 x 525 mm dia RCP @ RL 44.5 m AHD	Pit A1: 1 x 600 x 900 mm inlet pit Crest @ RL 45.3 m AHD Outlet 1 x 525mm dia RCP @ RL 43.4 m AHD Pit A2: 1 x 600 x 900 mm inlet pit Crest @ RL 44.8 m AHD Outlet 1 x 375 mm dia RCP @ RL 43.4 m AHD	1 x 15 m Wide weir Crest @ RL 45.6 m AHD
Basin B	-	Pit B1: 1 x 600 x 900 mm inlet pit Crest @ RL 37.1 m AHD Outlet 1 x 450 mm dia RCP @ RL 35.3 m AHD Pit B2: 1 x 600 x 900 mm inlet pit Crest @ RL 36.7 m AHD Outlet 1 x 450 mm dia RCP @ RL 35.3 m AHD	1 x 12 m Wide weir, crest @ RL 37.45 m AHD
Basin D	-	Pit D: 1 x 600 x 900 mm inlet pit Crest @ RL 57.3 m AHD Outlet 1 x 375 mm dia RCP @ RL 56.0 m AHD	1 x 7 m Wide weir, crest @ RL 57.6 m AHD

6.2.2 Modelling Results

The results of xpstorm modelling indicate that the inclusion of the three (3) proposed detention basins will mitigate the expected increases in peak discharge at PD-A, PD-B, and PD-D during all modelled AEP events.

Table 6.3 presents a comparison between the expected peak discharge during nominated AEP events for the pre- and post-development mitigated cases. Figure 6.7, 6.8 and 6.9 illustrate a comparison of the outlet hydrographs at PD-A, PD-B and PD-D.

Table 6.3 Peak Discharge Comparison (m^3/s) – Mitigated

PD	Scenario	Contributing Catchment Area (ha)	Annual Exceedance Probability (AEP) – Peak Discharge					
			0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
A	Existing	10.86	1.70	2.55	3.36	4.10	4.53	5.05
	Developed (Mitigated)	10.78	1.64	2.02	2.46	2.98	3.70	4.41
	Difference		**-0.08**	**-0.23**	**-0.62**	**-0.94**	**-0.99**	**-1.27**
B	Existing	4.40	0.83	1.21	1.53	1.83	2.08	2.30
	Developed (Mitigated)	5.20	0.72	1.00	1.23	1.49	1.72	2.11
	Difference		**-0.11**	**-0.21**	**-0.31**	**-0.34**	**-0.35**	**-0.19**
D*	Existing	1.94	0.51	0.70	0.82	1.01	1.15	1.29
	Developed (Mitigated)	1.83	0.49	0.62	0.72	0.81	1.12	1.29
	Difference		**-0.03**	**-0.08**	**-0.10**	**-0.20**	**-0.03**	**0.00**

*Peak discharge from Catchment D to PD-D. Does not include flows from Ext-2

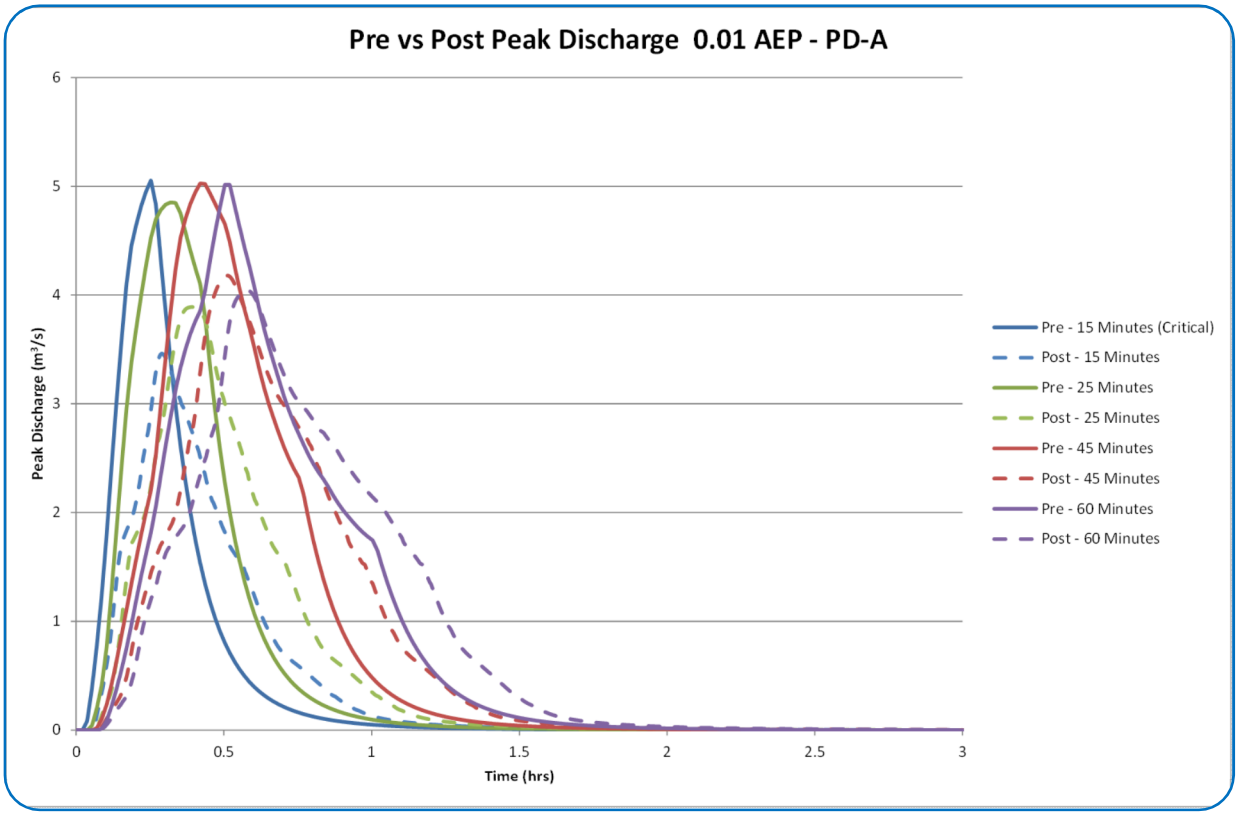


Figure 6.6 Comparison 0.01 AEP Hydrography PD-A

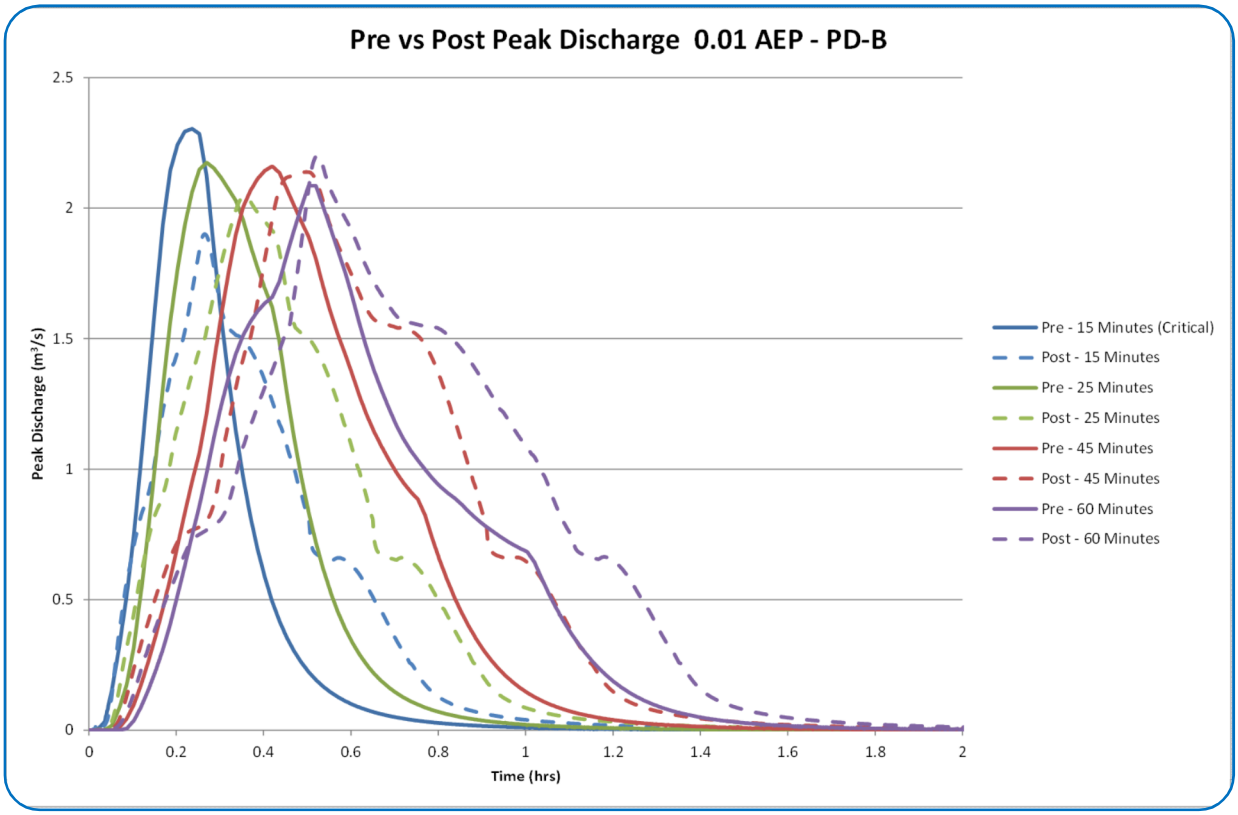


Figure 6.7 Comparison 0.01 AEP Hydrography PD-B

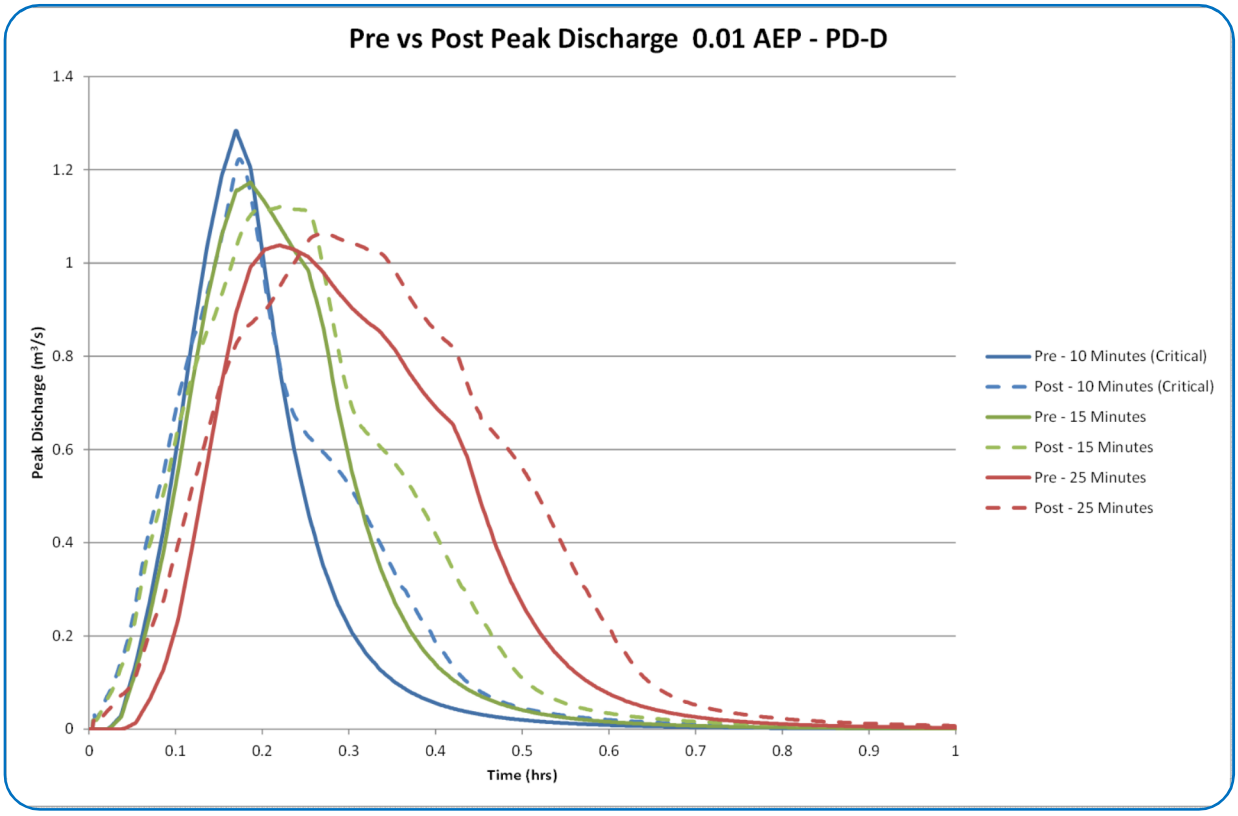


Figure 6.8 Comparison 0.01 AEP Hydrography PD-D

6.2.3 QUDM Minimum Design Safety Standards – Detention Depth

In accordance with the QUDM the depth of water in a detention basin is not to exceed 1.2 m during the 0.049 AEP and is to be limited to less than 1.5 m for the 0.01 AEP. A freeboard of 300 mm should also be achieved, along with a weir flow depth of no greater than 300 mm.

Table 6.4 presents the peak water levels, depths and volumes expected within the detention basins during both the 0.049 AEP and 0.01 AEP. Based on the *xpstorm* modelling of the proposed post-development scenario, the basin designs will not result in water depths outside the recommended design criteria with.

Table 6.4 Detention Storage Details, Depths and Volumes

	Basin A		Basin B		Basin D	
	0.049 (20 yr ARI)	0.01 (100 yr ARI)	0.049 (20 yr ARI)	0.01 (100 yr ARI)	0.049 (20 yr ARI)	0.01 (100 yr ARI)
Base Level (m AHD)	44.5	44.5	36.40	36.40	57.00	57.00
Top of Bund Level (m AHD)	46.0	46.0	37.90	37.90	58.00	58.00
Weir Level (m AHD)	45.6	45.6	37.45	37.45	57.60	57.60
Peak Water Surface (m AHD)	45.6	45.7	37.42	37.54	57.62	57.67
Peak Depth of Water (m)	1.1	1.2	1.02	1.14	0.62	0.67
Peak Volume (m^3)	1,654	1,966	923	1,079	78.5	104
Freeboard Achieved (m)	0.4	0.3	0.48	0.36	0.38	0.33
Depth of Weir Flow (m)	-	0.1	-	0.09	-	0.07

6.2.4 Sensitivity Analysis

In accordance with the requirements of QUDM Section 5.08.2, consideration has been given to the consequences of fully blocked outlets within each basin. Table 6.5 illustrates the expected peak water levels within each basin, should 50% blockage occur. In this event, water levels would increase and flows would be discharged over the high flow weir structures.

Table 6.5 Sensitivity Analysis Results for 0.01 AEP Event

Basin ID	Peak Water Level (m AHD)	Top of Bund (m AHD)	Depth of Weir Flow (m)
Basin A	45.85	46.0	0.25
Basin B	37.65	37.9	0.20
Basin D	57.79	58.0	0.19

Based on the proposed basin and weir configurations, peak water levels would not be expected to exceed the Top of Bund during the 0.01 AEP event for all three (3) basins. The expected maximum depth of flow over each weir has been modelled at less than 0.3 m.

6.2.5 QUDM Design Safety Standards – Inlet Grates

In accordance with Section 12.3 of the Queensland Urban Drainage Manual an initial safety risk assessment has been undertaken to determine the need for mitigation measures to reduce risks for any of the proposed inlet headwalls. These include the following structures:

- The culverts (4 x 300 mm x 1200 mm box culverts) underneath the internal road within Catchment A2a that convey flows from the natural channel underneath the internal road; and
- The culverts 2 x 525 mm which outlet through the Basin A bund.

This risk assessment has identified a cumulative score of 8 and 11 for the proposed box culverts and 2 x 525 mm stormwater structures respectively. In both scenarios Case 3 mitigation options are applicable in accordance with table 12.3.7 of QUDM.

Case 3 mitigation measures are described as:

> *Appropriate inlet screens should ideally be placed on stormwater inlets with openings that exceed a diameter of 300 mm, or a slot width of 125 mm wherever practical. However, the appropriate management of debris blockage issues and associated flooding risks could prevent the use of inlet screens in some cases. In such cases, consider the use of fencing (i.e. exclusion fencing or barrier fencing) to restrict access to the upstream channel.*
>
> *The placement of inlet screens on culverts and large stormwater pipes should generally only be considered as a last option if access to the upstream channel cannot be severely restricted and/or the safety risks associated with the culvert/pipe cannot be otherwise mitigated.*
>
> *Warning signs should be used liberally.*

Given the potential safety risk associated with the above conduits inlet screens have been recommended, and are to be designed in accordance with Section 12.5 of QUDM. Additionally, warning signages have been specified within the by-pass pond of Basin A (Refer to DWG-340 of the stormwater drawing set contained in Appendix B). Further assessment of the before mentioned structures and features to reduce safety risks will be provided as part of detailed design.

6.3 Lawful Point of Discharge

The criteria for determining a Lawful Point of Discharge (LPD) as specified within the Queensland Urban Drainage Manual (2016) is as follows:

(i) Will the proposed development alter the site's stormwater discharge characteristics in a manner that may substantially damage a third party property?

- If not, then no further steps are required to obtain tenure for a lawful point of discharge (assuming any previous circumstances and changes were lawful).
- If there is a reasonable risk of such damage then consider (ii) or (iii).

(ii) Is the location of the discharge from the development site under the lawful control of the local government or other statutory authority from whom permission to discharge has been received? This will include a park, watercourse, drainage or road reserve,

stormwater registered drainage easement, or land held by local government (including freehold land).

- If so, then no further steps are required to obtain tenure for a lawful point of discharge
- If not, then consider issue (iii). A land owner or regulator may require that the developer obtain an authority to discharge as described in (iii) in order for the stormwater to ultimately flow to a location described in (ii).

(iii) An authority to discharge over affected properties will be necessary. In descending order of certainty, an authority may be in the form of:

- Dedication of a drainage reserve or park;
- A registered easement for stormwater discharge/works; or
- Written approval.

The locations of the proposed points of discharge for each stormwater outlet are shown on Drawing DWG-200 – Appendix B. All the points of discharge have been aligned and designed so as to not result in any impacts that would substantially damage a third party property.

6.4 Quality Control – Pollutant Reduction

In accordance with the State Planning Policy for Healthy Waters and the Queensland Water Quality Guidelines (2009), treatment measures have been included to achieve the minimum mean annual load based reductions of 80% for Suspended Sediment; 60% for Total Phosphorus, 45% for Total Nitrogen; and 90% for Gross Pollutants.
The Model for Urban Stormwater Improvement Conceptualisation (*MUSIC v6*) has been used to estimate the potential pollutant loads generated by the development and to size the proposed treatment measures. The following sections outline the parameters relied upon within the *MUSIC v6* modelling.

6.4.1 Rainfall and Evapotranspiration Data

Rainfall and evapotranspiration data has been obtained from the Bureau of Meteorology (BOM) rainfall station and is included within Table 6.6 below.

Table 6.6 Meteorological and Rainfall Runoff Data Reporting

Station	40584 Hinze Dam
Period	1/01/1976 to 31/12/1985 (10 years)
Time step	6 minute
Mean annual rainfall (mm)	1371
Evapotranspiration	1539

6.4.2 Developed Case - Catchment Parameters

The developed site has been modelled as an Urban Residential Land use. Each internal catchment has been split into roof, road and ground level source nodes. Table 6.7 summarises the sub-catchment areas and Drawing DWG-210 – Appendix B presents the sub-catchment delineation. The pollutant export and runoff parameters for each sub catchment adopted is based on the data from the Water by Design *MUSIC v6* Modelling Guidelines (2010), as summarised in Table 6.8 and 6.9.

Table 6.7 *MUSIC v6* Sub-Catchment Areas

Catchment ID	Land Use	Area (ha)	Total Impervious (%)
Catchment A2a	Urban Residential - Road	0.72	60
	Urban Residential - Roof	0.65	100
	Urban Residential - Ground Level	1.25	20
Catchment A2b	Urban Residential - Road	0.67	60
	Urban Residential - Roof	0.88	100
	Urban Residential - Ground Level	1.03	20
Catchment B	Urban Residential - Road	1.51	60
	Urban Residential - Roof	1.46	100
	Urban Residential - Ground Level	1.80	20
Catchment C	Urban Residential - Road	0.50	60
	Urban Residential - Roof	0.82	100
	Urban Residential - Ground Level	0.84	20
Catchment D	Urban Residential - Road	0.48	60
	Urban Residential - Roof	0.49	100
	Urban Residential - Ground Level	0.60	20

Table 6.8 Rainfall Runoff Parameters

Parameter	All Nodes
Land use	Residential
Rainfall threshold (mm)	1
Soil storage capacity (mm)	500
Initial storage (% capacity)	10
Field capacity (mm)	200
Infiltration capacity coefficient a	211
Infiltration capacity exponent b	5.0
Initial depth (mm)	50
Daily recharge rate (%)	28
Daily baseflow rate (%)	27
Daily deep seepage rate (%)	0

Table 6.9 Pollutant Export Parameters

Flow Type	Surface Type	Total Suspended Solids (log mg/L)		Total Phosphorous (log mg/L)		Total Nitrogen (log mg/L)	
		Mean	Std Dev.	Mean	Std Dev.	Mean	Std Dev.
Urban Residential							
Base Flow	Roof	N/A	N/A	N/A	N/A	N/A	N/A
	Road	1.00	0.34	-0.97	0.31	0.20	0.20
	Ground	1.00	0.34	-0.97	0.31	0.20	0.20
Storm Flow	Roof	1.30	0.39	-0.89	0.31	0.26	0.23
	Road	2.43	0.39	-0.30	0.31	0.26	0.23
	Ground	2.18	0.39	-0.47	0.31	0.26	0.23

6.4.3 Treatment Measures

In order to determine the design requirements for the necessary stormwater treatment measures, key "Treatment Nodes" were added to the *MUSIC v6* model. The following sections outline the modelling parameters relied upon for each "Treatment Node".

Buffers & Grassed Channel

It is noted that there are two (2) minor areas within Catchment A2a and C (identified as 'untreated area' in the MUSIC Catchment Plan – DWG 210, Appendix B) which are unable to be directed to the proposed bioretention measures. These areas include a grassed batter within A2a, and the rear of four (4) lots within Catchment C. In order to treat these areas treatment has been proposed via grassed buffers (the batter and ground area from the four (4) lots in Catchment C) and a grassed swale that will form a drainage easement.

The buffer strip will reduce sediment loads by slowing stormwater flow and causing coarse sediments to fall out of suspension.

Key buffer design parameters include:

- Even distribution of flow onto the buffer;
- Avoiding rilling or channelled flows;
- Maintaining flow depths less than vegetation heights; and
- Minimising the slope of the buffer to below 5%.

Table 6.10 presents the buffer parameters relied upon for the purpose of MUSIC modelling.

Table 6.10 Buffer Parameters

Parameter	Parameter Settings
Upstream area buffered (%).	50
Buffer area (% upstream impervious area)	5
Exfiltration rate (mm/hr)	0

The drainage easement that runs along the rear of Lots 119-121 has been designed as a grassed channel to convey flows from a portion of these lots to Mackellar Drive. The channel will be vegetated (grassed) and therefore help to reduce sediment loads by slowing stormwater and causing sediments to fall out of suspension and as such has been modelled

as a grassed swale. Table 6.11 summaries the parameters used within the *MUSIC v6* modelling undertaken for the subject site.

Table 6.11 Grass Channel Parameters

Grass Swale Parameters	Parameter Settings
Length (m)	20
Bed Slope	1:5
Base Width (m)	0.3.
Top Width (m)	3
Depth (m)	0.30
Vegetation Height (m)	0.25
Exfiltration Rate (mm/hr)	0

Bioretention

It is proposed that bioretention measures be incorporated into the development layout to provide the necessary load-based reductions. Bioretention systems operate by capturing and retaining water in an extended ponding area (no more than 400 mm deep for a maximum of four (4) days to prevent anaerobic conditions, plant death and insect breeding), before filtering through a soil media. The devices remove pollutants via the following physical processes:

- Sedimentation in the extended detention storage;
- Filtration by filter media;
- Nutrient uptake by biofilms;
- Nutrient adsorption and pollutant decomposition by soil bacteria; and
- Adsorption of metals and nutrients by filter particles (Somes & Crosby, 2007).

In accordance with Water by Design Guidelines, Table 6.12 summarises the treatment node parameters used in the *MUSIC v6* modelling whilst Figure 6.10 provides typical design parameters for the proposed bioretention devices.

Table 6.12 Bioretention Parameters

Parameter	Bioretention A	Bioretention B	Bioretention C	Bioretention D
Surface area (m^2)	2 x 320	470	205	165
Extended detention depth (m)	0.3	0.3	0.3	0.3
Filter area (m^2)	2 x 290	420	175	140
Unlined filter media perimeter (m)	0.01	0.01	0.01	0.01
Saturated hydraulic conductivity (mm/hr)	200	200	200	200
Filter depth (m)	0.7	0.7	0.5	0.5
TN content of filter media (mg/kg)	400*	400*	400*	400*
Proportion of organic material in filter (%)	< 5	< 5	< 5	< 5
Orthophosphate content of filter media (mg/kg)	30*	30*	30*	30*
Is the base lined?	Yes	Yes	Yes	Yes
Effectiveness of plant TN removal	Effective	Effective	Effective	Effective
Overflow weir width (m)	Surface Area/10	Surface Area/10	Surface Area/10	Surface Area/10
Exfiltration rate (mm/hr)	0.00	0.00	0.00	0.00

If an exfiltration rate has been used, have node water balance losses been used in calculation of treatment train effectiveness?	N/A	N/A	N/A	N/A
If exfiltration rate has been used, is the exfiltration rate justified?	N/A	N/A	N/A	N/A
Underdrain present?	Yes	Yes	Yes	Yes
Submerged zone with carbon present?	No	No	No	No
Depth of submerged zone (m)	N/A	N/A	N/A	N/A
Confirmation that K and C* remain default?	Yes	Yes	Yes	Yes
Sediment Forebay Required (>2.5 ha contributing catchment)	Yes	Yes	No	No

*As per Healthy Land and Water Recommendations for *MUSIC v6* modelling

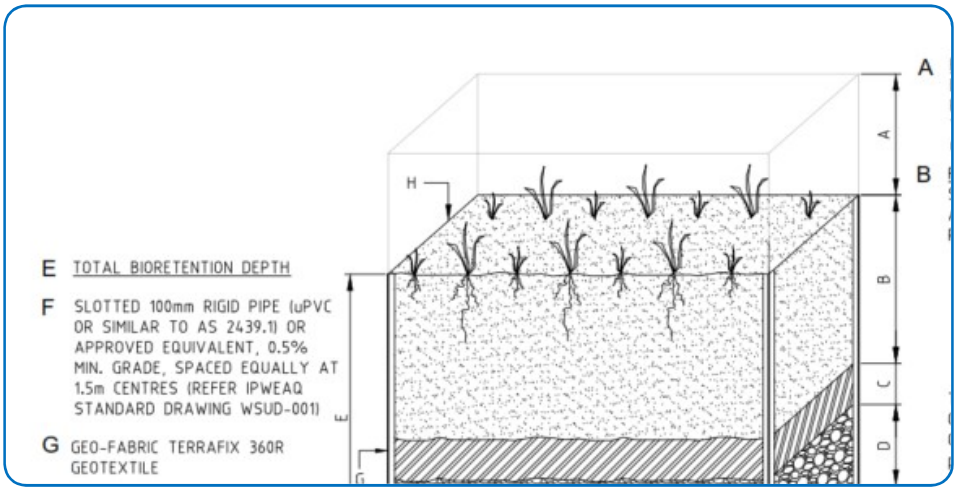


Figure 6.9 Bioretention specifications

Coarse Sediment Forebay

As the contributing catchment to bioretention basins A2a, A2b, B and C will greater than 2 ha a coarse sediment forebay will be installed at each piped discharge point into the bioretention basin. The forebay will limit sediment accumulation over the bioretention filter surface and reduce the potential for clogging. This will also help to reduce the frequency of maintenance required for the bioretention systems. Table 6.13 provides a summary of the sediment forebay calculations and requirements. The sediment forebays have been excluded from the *MUSIC v6* modelling. It is noted that basin A has two (2) inlet points.

Table 6.13 Sediment Forebay Parameters

Forebay ID	Contributing Catchment Area (ha)	Q_{3month} Inflow Rate (m^3/s)	Settling Velocity (m/s)	Area (m^2)	Depth (m)	Capture Efficiency (%)
A2a	2.62	0.33	0.1	40	0.15	80
A2b	2.58	0.31	0.1	36	0.15	80
B	4.77	0.51	0.1	58	0.15	80
C	2.16	0.25	0.1	30	0.15	80

6.4.4 Bioretention Basin Staged Construction and Establishment

In accordance with the Water by Design Construction and Establishment Guidelines: Swales Bioretention Systems and Wetlands (C & E Guidelines), inflow of sediment-laden runoff during

the building stage is a major risk to the successful and long-term functioning of bioretention systems.

During the building phase of developments, sediment can seal the surface of the filter media, move into and clog the filter media and accumulate in the under-drainage. Within the C & E Guidelines, there are four (4) options recommended to overcome the challenges associated with delivering bioretention systems. These generally include the following options:

- Option 1: Surface Protection
- Option 2: Bypass flows and early establishment of vegetation;
- Option 3: Sediment basin and bioretention function; and
- Option 4: Leave as sediment basin.

Any of the above options will ensure adequate protection of the bioretention during the building phase and are suitable for implementation.

It is proposed that the Option 4 method for staged construction and establishment be implemented for the proposed development. This method will generally include the following:

- Earthworks and bulking out of basin;
- Install the hydraulic structures;
- Allow system to operate as a sediment basin during the building phase; and
- When 80-90% of building in the catchment is complete, the bioretention system can be cleaned and profiled, the under-drainage and bioretention media installed, and the vegetation planted and established.

For details of the procedure and order of construction for Option 4 method please refer to Section 3.10 (Steps 1-41) of the C& E Guidelines. If an alternative option is found to be more suitable for the site, construction methods and steps should be referenced from the C & E Guidelines.

It is noted that the methodologies of construction listed above are for the sole purpose of protection of the bioretention measures during the construction phase. It is expected that all active building/construction sites will be subject to independent Erosion and Sediment Control measures and plans.

6.4.5 Modelling Results

Results of the *MUSIC v6* modelling for the treatment train effectiveness are summarised in Table 6.14. The results indicate the 80%, 60%, 45% and 90% reduction target for TSS, TP, TN and gross pollutants respectively are achieved for the rainfall data set simulated at both assessment points. Screen captures of the *MUSIC v6* modelling results are included as Figures 6.11 to 6.13.

Table 6.14 Treatment Train Effectiveness

Catchment ID	Pollutant	Inflows (kg/yr)	Outflows (kg/yr)	Reduction (kg/yr)	Reduction Achieved (%)	Water Quality Objective (%)
Northern	TSS	15,500	2,760	12,740	**82.2**	80
	TP	31.4	7.92	23.48	**74.8**	60
	TN	182	87.2	94.8	**52**	45
Southern	TSS	5,320	985	4,335	**81.5**	80
	TP	11.4	3.15	8.25	**72.5**	60
	TN	70.1	34.7	35.4	**50.5**	45
Combined	TSS	20,800	3,750	17,050	**82**	80
	TP	42.9	11.1	31.8	**74.2**	60
	TN	252	122	130	**51.6**	45

NOTE: All simulations have been run with pollutant export estimation set to “stochastic generation”.



Figure 6.10 Screen Capture of MUSIC Modelling Results (Northern)



Figure 6.11 Screen Capture of MUSIC Modelling Results (Southern)

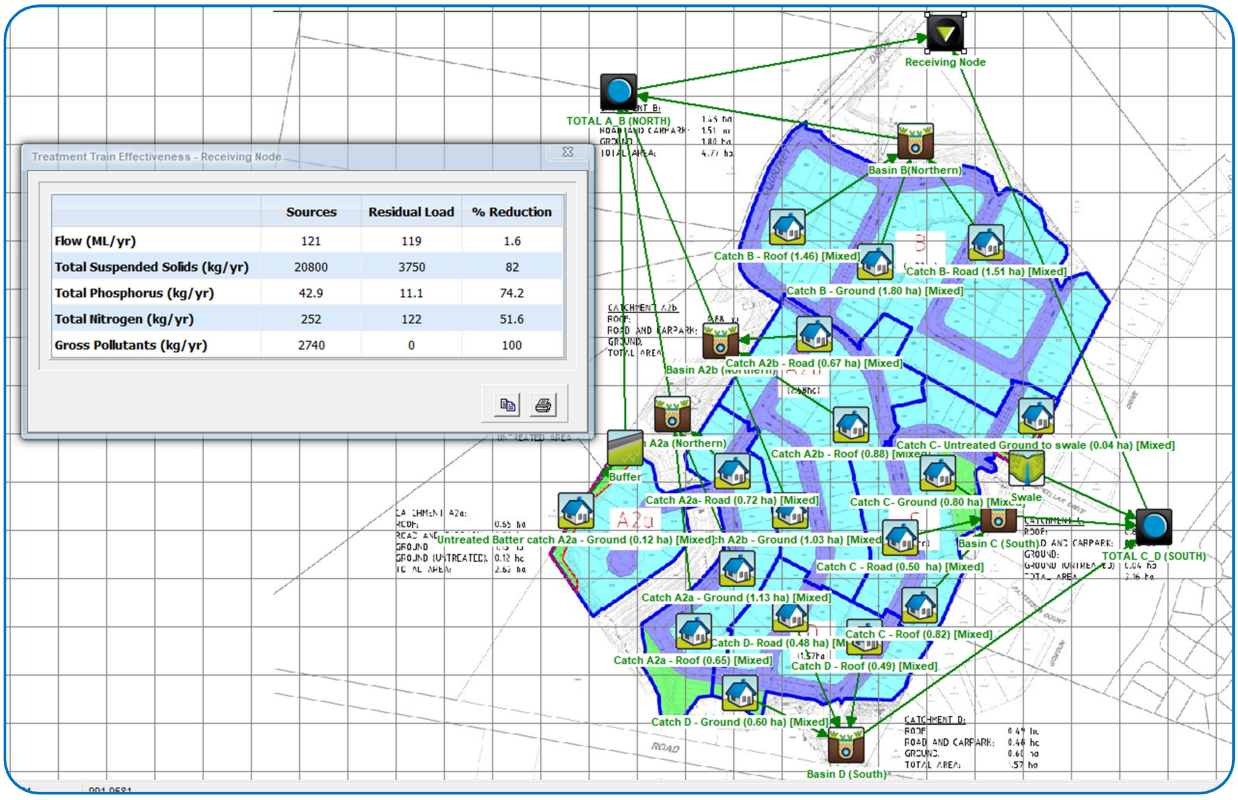


Figure 6.12 Screen Capture of MUSIC Modelling Results (Combined)

6.5 Water Quality Monitoring and Maintenance

In order to ensure that the stormwater management measures detailed within this management plan function correctly in the long term and to ensure that impacts to downstream receiving environments are mitigated, appropriate operational phase water maintenance and monitoring is to be undertaken. The following sections detail the minimum requirements for each specific control device.

6.5.1 Operational Phase Water Quality Monitoring

Monitoring during the operational phase will be undertaken to determine the impact of activities on the receiving waters. Surface water quality monitoring is to be undertaken at discharge points from the site. Samples should be collected for total suspended solids (TSS), pH, dissolved oxygen (DO), TP, TN and hydrocarbons. Sampling is to be performed in accordance with procedures set out in the Environmental Protection Authority's Water Quality Sampling Manual. A NATA registered laboratory is to be used to analyse the collected samples.

Table 6.15 specifies the parameters and sampling frequencies required. Results of the monitoring program are to be compiled monthly into an ongoing Water Quality Monitoring Report. A copy of the report and monitoring data is to be maintained at all times.

Table 6.15 Operational Phase Water Quality Parameters and Sampling Frequencies

Sampling Parameter	Sampling Frequency
Total Suspended Solids (TSS)	Water quality monitoring will be completed following a rainfall event of 25 mm or greater in any 24 hour period monthly for a minimum period of 12 months, or as specified by the Local Authority conditions of approval for the development.
pH	
Dissolved Oxygen (DO)	
TN	
TP	
Hydrocarbons	

Table 6.16 sets the water quality criteria for discharge water realised from the development site.

Table 6.16 Operational Phase Water Quality Discharge Criteria

Water Quality Parameter	Discharge Criteria
Total Suspended Solids (TSS)	No net deterioration of the downstream receiving environment as a result of discharge from the development.
Turbidity (NTU)	
pH	
DO	
TN	
TP	
Hydrocarbons	

6.6 Operational Phase Device Maintenance

In order for each of the proposed stormwater treatment devices to achieve the necessary pollutant removal efficiencies regular maintenance is necessary. Poorly maintained devices will result in under performance and in some instances may cause leaching of pollutants to downstream receiving environments. Based on the proposed treatment train Table 6.17 details appropriate maintenance regimes for each treatment device.

Table 6.17 Operational Phase Device Maintenance Requirements

Treatment Device	Maintenance Action
Grassed Swales	Routine inspection to identify obvious increased sediment deposition. Remove sediment when flow within the swale is impeded or smothering of vegetation occurs. If necessary reprofile swale and revegetate.
	Regular watering of vegetation until plants are established.
	Mowing of grass or slashing of vegetation to preserve the optimal design height.
	Removal and management of invasive weeds.
	Removal and replacement of dead and dying vegetation.
	Regular inspection and removal of litter and gross pollutants.
Bioretention	Routine inspection to identify obvious increased sediment deposition. Remove sediment when flow within the bioretention is impeded or smothering of vegetation occurs.
	Routine inspection of inlet and outlet pit to identify any areas of scour, litter build up or blockages.
	Tilling of the bioretention surface if there is evidence of clogging.
	Regular watering of vegetation until plants are established.
	Removal and management of invasive weeds.
	Removal and replacement of dead and dying vegetation.
	Regular inspection and removal of litter and gross pollutants.
	Resetting (i.e. complete reconstruction) of the bioretention will be required if the flow within the bioretention system is reduced by 25% due to an accumulation of sediment.
Detention Basin Area	Routine inspection to identify obvious increased sediment deposition. Remove sediment if there is any sign of smothering of vegetation. If necessary reprofile base and revegetate.
	Routine inspection of inlet and outlet structures to identify any areas of scour, litter build up or blockages.
	Regular watering of vegetation until plants are established.
	Removal and management of invasive weeds.
	Removal and replacement of dead and dying vegetation.
	Regular inspection and removal of litter and gross pollutants.
Inlet/Bypass Pond	Routine inspection to identify obvious increased sediment deposition. Remove sediment accumulation at inflow points. If sediment accumulation occurs within pond to a depth . 50% removal and reprofile base if necessary.
	Routine inspection of inlet and outlet structures to identify any areas of scour, litter build up or blockages.
	Regular watering of vegetation until plants are established.
	Removal and management of invasive weeds.
	Removal and replacement of dead and dying vegetation.
	Regular inspection and removal of litter and gross pollutants.

All material removed during maintenance, whether solid or liquid, is to be disposed of in a manner that does not cause ongoing soil erosion or environmental harm.

7 Hydraulic Assessment

The 1D - 2D hydrodynamic flood model TUFLOW has been utilised to route stormwater through the upper reaches of Yaun Creek and the other minor streams which cross the site. This modelling will confirm:

- The extent of flooding from Yaun Creek;
- The effect of the proposed earthworks on this flooding; and
- Sizes of infrastructure required to route flood flows through the site.

7.1.1 Modelling Procedure

To simulate the peak flow behaviour within the catchment and quantify peak flood depths, discharges, and flow velocities, hydraulic modelling using the 1D/2D hydrodynamic model *TUFLOW* was undertaken. This involved the insertion of inflow hydrographs into the model upstream of the site. The hydrographs have been calculated using *WBNM* software as described in the following sections. Pre-development and post-development scenarios have been modelled using LiDAR and proposed earthworks data respectively.

7.2 Hydrology – *WBNM*

WBNM hydrological software has been utilised to generate hydrographs to represent the contributing catchments. WBNM is a software package which calculates flood runoff from rainfall hyetographs, utilising sub-catchments and flow routings to estimate flood flows at various points within the catchment.

Hydrographs were generated for nominated Annual Exceedance Probability's (AEP)'s with peak discharges derived from the output hydrographs. The following sections detail the catchment parameters used in the *WBMN* modelling.

7.2.1 Design Rainfall

Rainfall data for this report has been based on site specific Intensity-Frequency-Duration (IFD) data acquired from the Bureau of Meteorology summarised in Figure 7.1.

Intensity-Frequency-Duration Ta

Location: 27.875S 153.275E Issued: 21/6/2

Rainfall intensity in mm/h for various durations and Average F

	Average Recurrence Interval				
Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	2
5Mins	125	158	193	212	
6Mins	117	148	181	199	
10Mins	96.0	122	149	164	
20Mins	70.4	89.3	110	122	
30Mins	57.4	72.9	90.1	99.8	
1Hr	38.9	49.5	61.7	68.6	

Figure 7.1 Adopted Intensity Frequency Data (mm/hr)

7.2.2 Temporal Patterns

Temporal patterns applied to the rainfall data have been extracted from Australian Rainfall and Runoff (1987) for Zone 3. Storm durations of 25, 60, 90, 120, 180, 360, 540, 720, 1440, and 1800 minutes have been analysed for each AEP.

7.2.3 Design Storm Events

This report has analysed a range of storm events which simulate the rainfall from hypothetical storms based on an Average Exceedance Probability (AEP). Table 7.1 lists each modelled AEP storm event and the corresponding Average Occurrence Interval (ARI) and description.

Table 7.1 Design Storm Events

Annual Exceedance Probability (AEP)	Average currence Interval (ARI)	Storm Description
0.632	1 year	Storm rainfall total has a 63.2% probability of being exceeded in any one year
0.393	2 years	Storm rainfall total has a 39.3% probability of being exceeded in any one year
0.181	5 years	Storm rainfall total has a 18.1% probability of being exceeded in any one year
0.095	10 years	Storm rainfall total has a 9.5% probability of being exceeded in any one year
0.049	20 years	Storm rainfall total has a 4.9% probability of being exceeded in any one year
0.02	50 years	Storm rainfall total has a 2% probability of being exceeded in any one year
0.01	100 years	Storm rainfall total has a 1% probability of being exceeded in any one year

7.2.4 Land Uses

The land uses within each sub-catchment have been used to determine the impervious fraction relied upon in hydrological modelling. The land use parameters for this study are based upon the City of Gold Coast planning scheme assuming fully developed catchments. Table 7.2 below summarises the percentage impervious adopted for each land use type.

Table 7.2 Fraction Impervious

Land Use Type	Fraction Impervious (%)
Open Space	0
Low Density Urban	40
Rural	20

7.2.5 Infiltration Losses

Infiltration parameters applied to catchments simulate the volume of runoff lost into the soil as initial and continuing values. For the upstream catchment, values of 5 mm initial and 2.5 mm/hr continuing have been applied globally. These values are consistent with recommendations in *Australian Rainfall and Runoff (1987).*

7.2.6 *WBNM* Parameters

The *WBNM* parameters chosen for the hydrological modelling have been chosen in accordance with the *WBNM* documentation with reference to the specific characteristics of the catchment. These values control the sub-catchment lag and routing within each sub-catchment of the model and are listed in Table 7.3 below.

Table 7.3 WBNM Parameters

Parameter	Value
Catchment Lag Parameter	1.6
Impervious Lag Factor	0.1
Stream Lag Factor	1

7.2.7 Catchment Delineation

The hydrological assessment analysed the total catchment of Yaun Creek and delineated catchments based on:

- LiDAR survey obtained from the Department of Natural Resources and Mines; and
- land uses based on Council mapping and aerial photography.

A total catchment area of approximately 150 ha was delineated based on 22 sub-catchments. The area of each sub-catchment is shown within Table 7.4. Figure 7.2 illustrates the catchment location and extent.

Table 7.4 Catchment Characteristics

Catchment ID	Catchment Area (ha)	Fraction Impervious (%)
A	7.843	10
B	4.07	10
C	6.507	5
D	6.354	10
E	6.843	5
F	5.654	10
G	1.983	10
H	4.481	5
I	6.978	5
J	8.565	0
K	4.221	0
L	9.895	0
M	7.573	0
N	8.199	0
O	11.068	0
P	6.273	0
Q	10.525	0
R	9.664	0
S	7.116	0
T	5.426	5
U	4.05	0
V	5.715	0

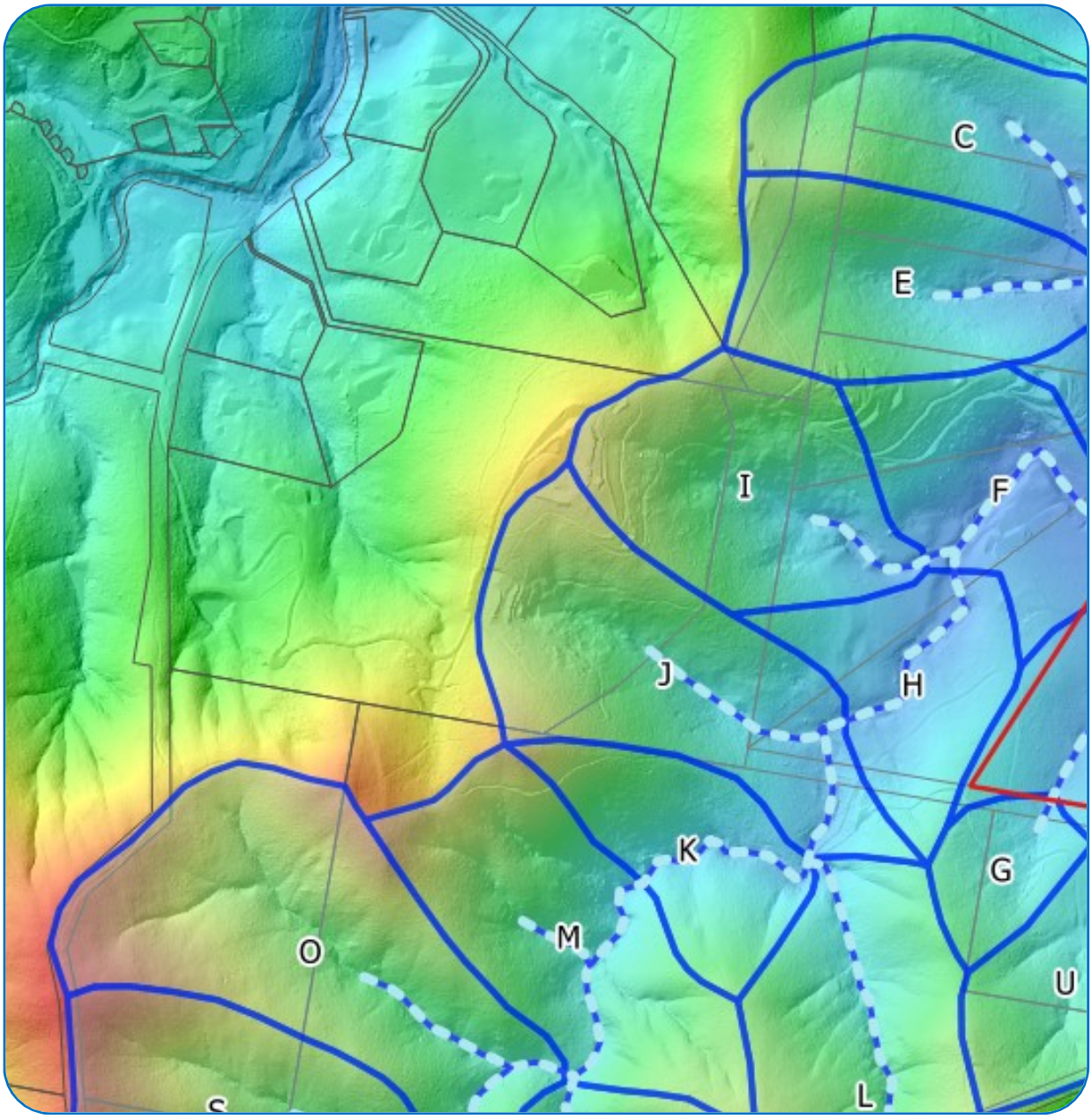


Figure 7.2 Catchment Plan

7.2.8 Critical Storm Duration

In order to determine the critical storm event for the Yaun Creek catchment, *WBNM* was relied upon to identify the storm duration that produced the highest peak flow immediately downstream of the site. Table 7.5 below presents the results of this assessment and concludes that the 90 minute duration storm is critical. Figure 7.3 presents the hydrographs for each duration.

Table 7.5 Critical Storm Durations

AEP	0.01 AEP Peak Flow (m^3/s)	Storm Duration (mins)
0.01	13.85	25
0.01	24.21	60
0.01	25.90	90
0.01	25.73	120
0.01	24.20	180
0.01	20.42	360
0.01	21.30	540
0.01	18.84	720
0.01	22.77	1440
0.01	17.85	1880

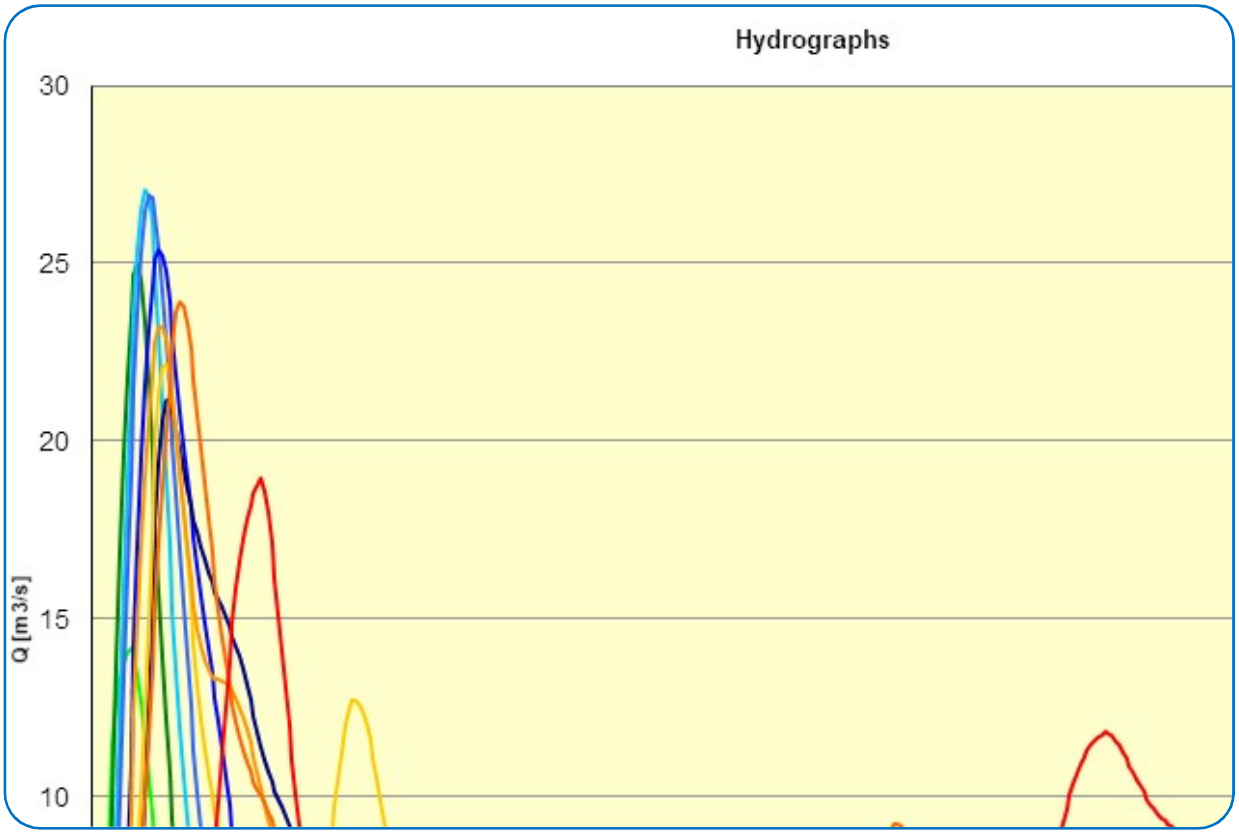


Figure 7.3 All Storm Duration Hydrographs – 0.01 AEP

7.2.9 Calculated Peak Flows

The expected inflow peak discharges generated for the critical storm event were extracted from the catchment hydrographs generated within *WBMN*. The peak result for each catchment are summarised within Table 7.6 below.

Table 7.6 *WBMN* Peak Catchment Discharge – 0.01 AEP

Catchment ID	Peak Flow (m^3/s)
A	2.819
B	1.586
C	2.316
D	2.351
E	2.419
F	2.123
G	0.83
H	1.67
I	2.461
J	2.799
K	1.529
L	3.154
M	2.526
N	2.7
O	3.457
P	2.153
Q	3.318
R	3.093
S	2.397
T	1.977
U	1.474
V	1.989

7.3 Hydraulics – *TUFLOW*

7.3.1 Digital Terrain Model (DTM)

The pre-development modelling scenario has relied upon LiDAR data of the site and contributing catchments. The post-development scenario utilises the preliminary civil design DTM provided by Mortons Urban Solutions. The proposed earthworks extends into the Yaun Creek inundation area on the western side of the site. The small catchment to the south of the site is to be routed through the site within a channel and culverts located midway through the channel underneath the internal road. The flowpath in the south-eastern corner of the site is to be piped under the proposed road.

7.3.2 Downstream Boundary Condition

The downstream boundaries of the model have been applied as free draining outlets. This arrangement is considered appropriate due to the relative steepness of the flowpaths and the absence of regional flooding in the vicinity.

7.3.3 Manning's Coefficient

Based on recommendations from; *Chow. V. T, (1959) Open Channel Hydraulics*, *Tweed Shire Council Development Design Specification D5 (V1.4),* the following Manning Coefficients were applied:

- Concrete/Road: 0.020
- Open water: 0.030
- Grassed areas: 0.050
- Light Vegetation : 0.060
- Dense Vegetation: 0.080

Figure 7.4 below presents the *TUFLOW* model layout.

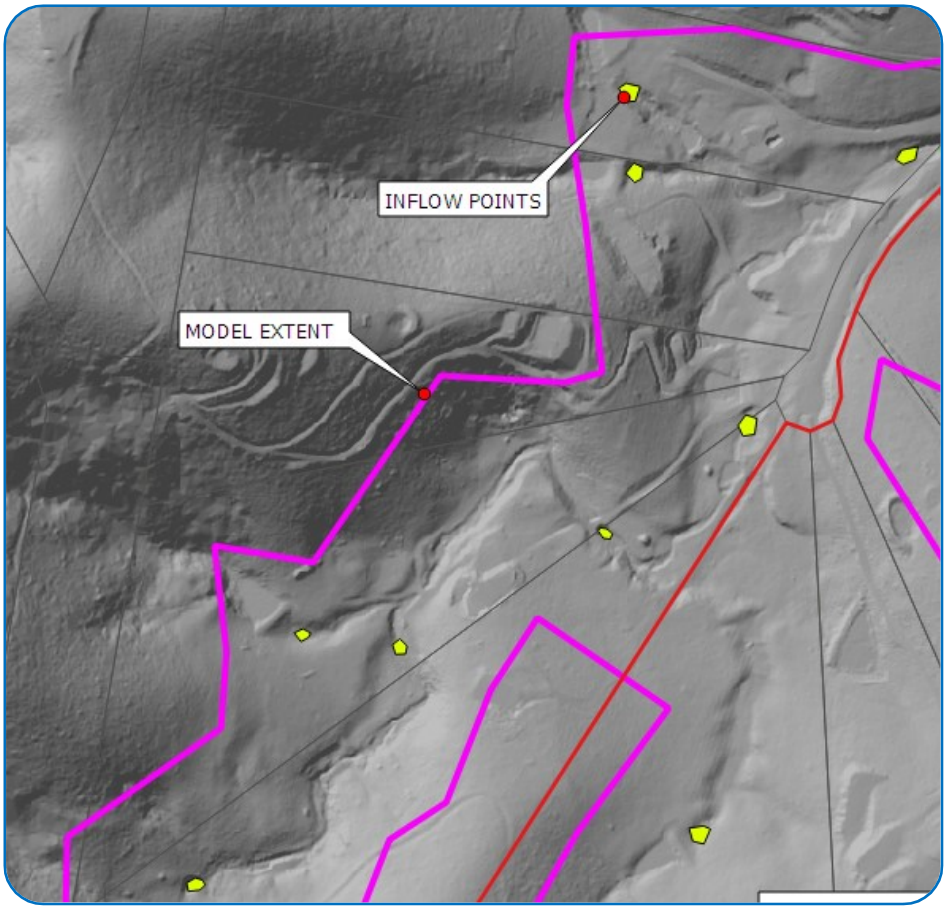


Figure 7.4 *TUFLOW* Model Layout

7.3.4 Hydraulic Model Simulation

Simulation of the adopted design storm events was undertaken based on:

- Applying the inflow hydrographs within the *TUFLOW* model; and
- Applying the appropriate downstream boundary condition to the hydraulic model.

Key model outputs describing flood behaviour include peak water levels, flows and velocities. Results of the hydraulic analysis have been presented in the following sections for the existing and design scenarios.

7.4 Modelling Results

The following sections provide a summary of the 2D modelling results for the pre and post-development scenarios.

7.4.1 Pre-Development Scenario

Pre-development flood depths and extents estimated by *TUFLOW* for the 0.01 AEP storm is presented in Figure 7.5 below. The existing flow regime can be described as follows:

- Yaun Creek briefly crosses the western edge of the site and then again in the northern corner downstream of Courtney Drive;
- Runoff from two external catchments to the south flow through the site; and
- Peak flood depths in Yaun Creek are approximately 3.6 m immediately upstream of Courtney Drive.

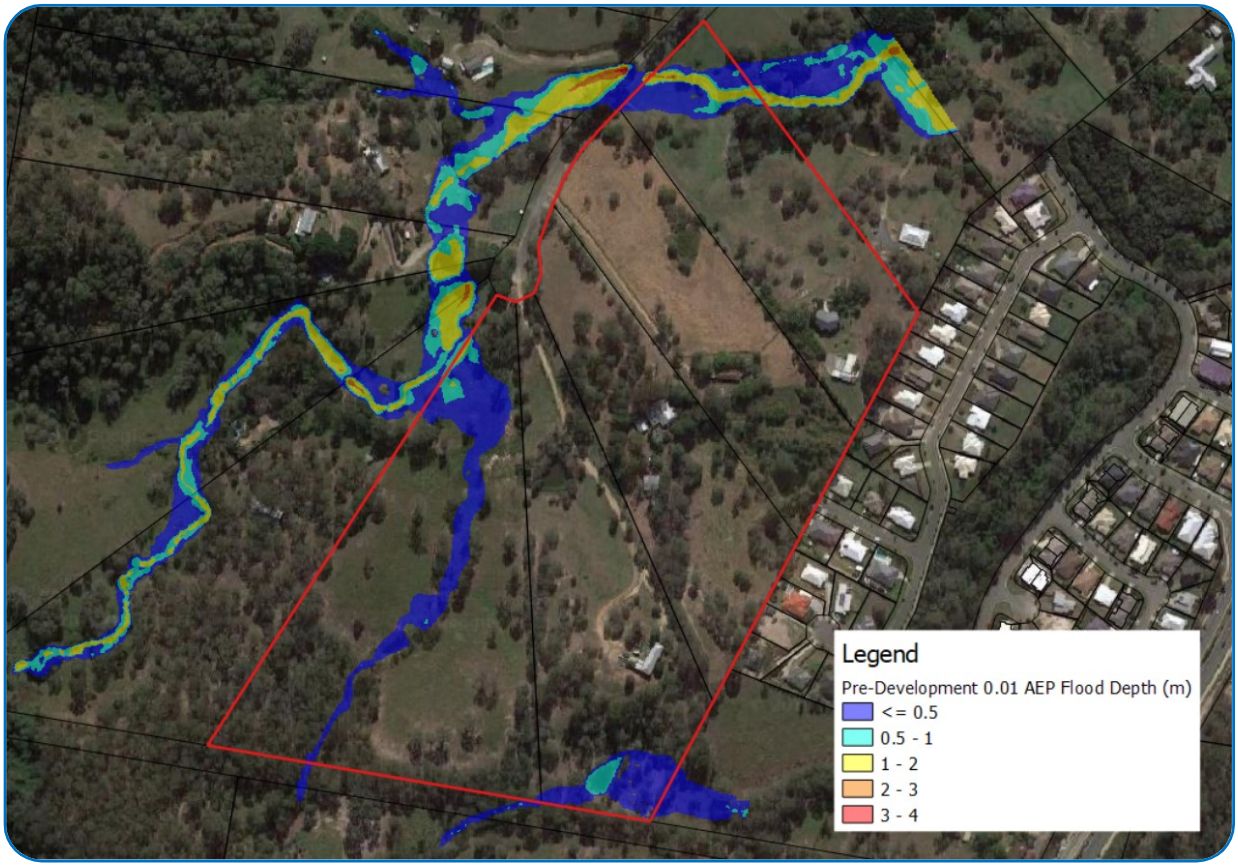


Figure 7.5 0.01 AEP Peak Flood Depth – Pre-Development

7.4.2 Post-Development Scenario

Post-development flood depths and extents estimated by *TUFLOW* for the 0.01 AEP storm are presented in Figure 7.6 below. Flow characterises in the post-development case can be summarised as follows:

- flow from Yaun Creek is in unimpeded by the proposed site earthworks;
- The south-western external catchment is routed through the site within a constructed waterway with flows conveyed underneath the internal road via a set a 1200 mm diameter RCP;
- The south eastern catchment is routed around the proposed development to the Council controlled park area;
- Peak flood depths in Yaun Creek are approximately 3.6 m immediately upstream of Courtney Drive;

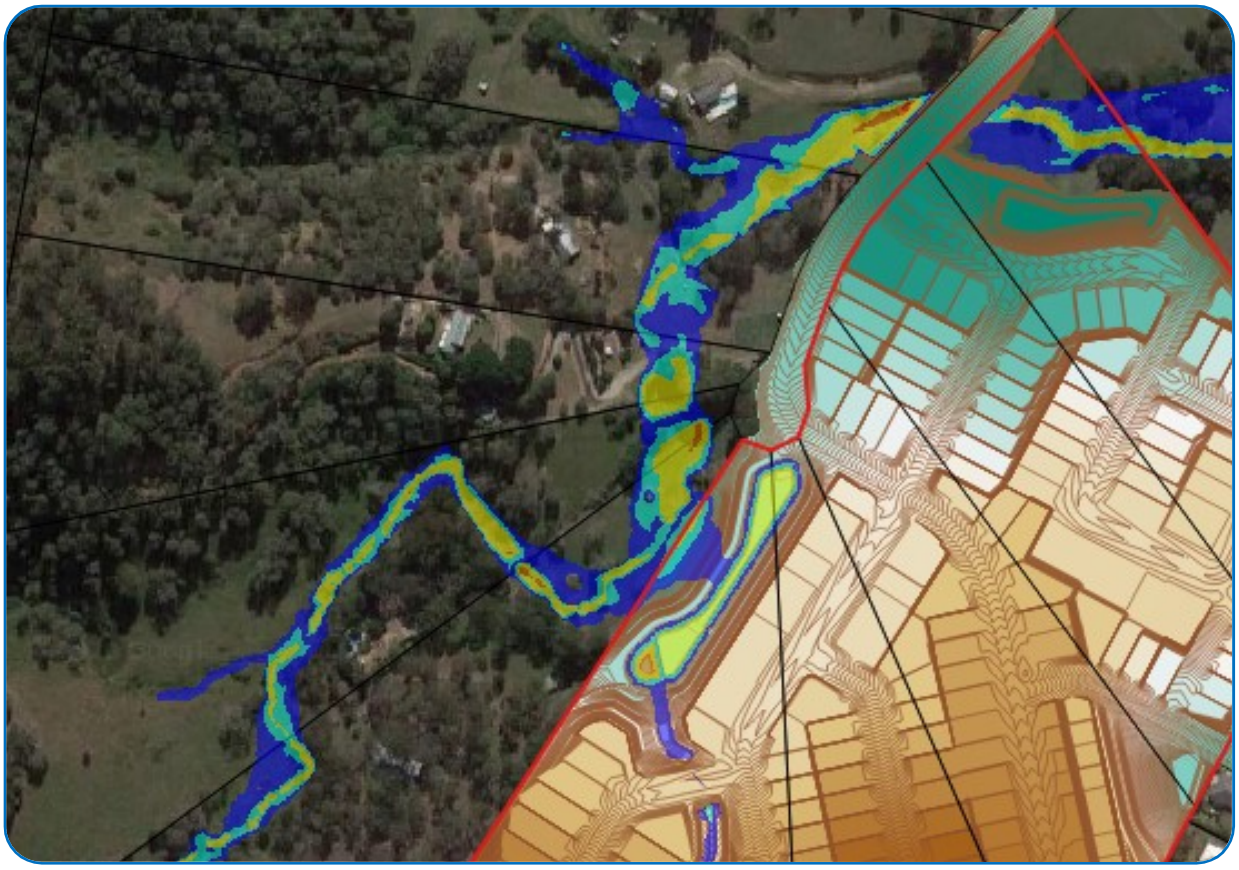


Figure 7.6 0.01 AEP Peak Flood Depth – Post-Development

- Flood depth within the constructed waterway is generally less than 0.5 m, refer to Figure 7.7;
- Peak velocity within the constructed waterway peaks at 2.0 m/s and is generally less than 1 m/s as shown on Figure 7.8; and
- The depth x velocity product within the channel is generally less than 0.6. The central rocked low flow portion of the channel peaks at 0.8. Figure 7.9 shows the DxV plot of the waterway.

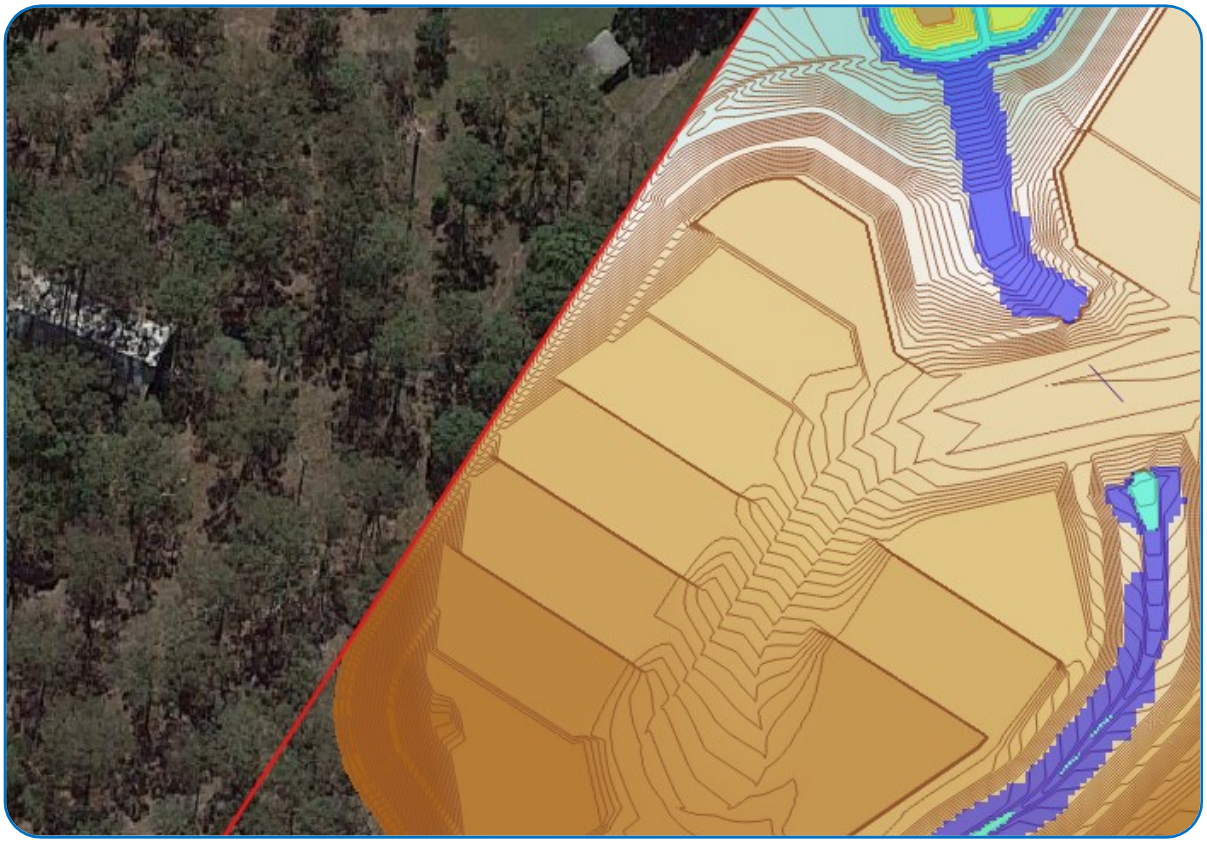


Figure 7.7 0.01 AEP Peak Flood Depth – Waterway

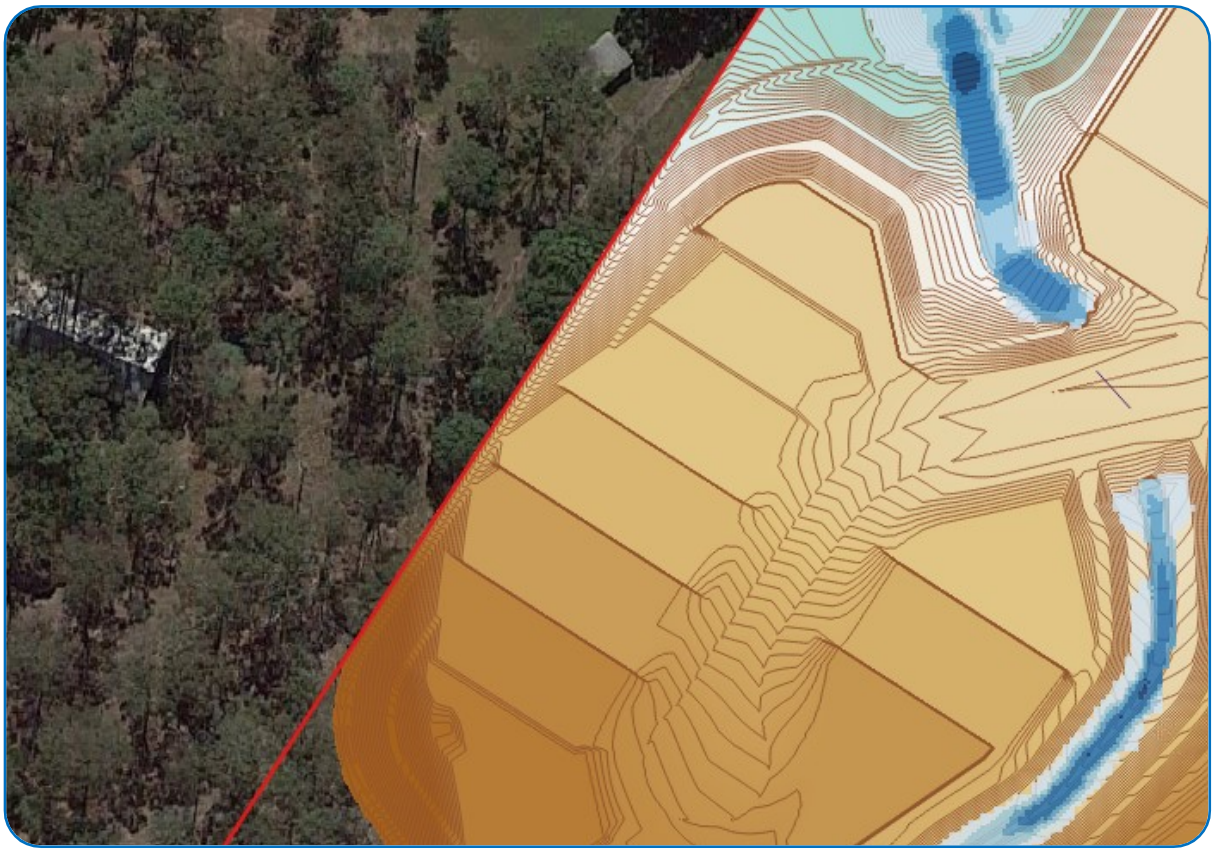


Figure 7.8 0.01 AEP Peak Flood Velocity – Waterway

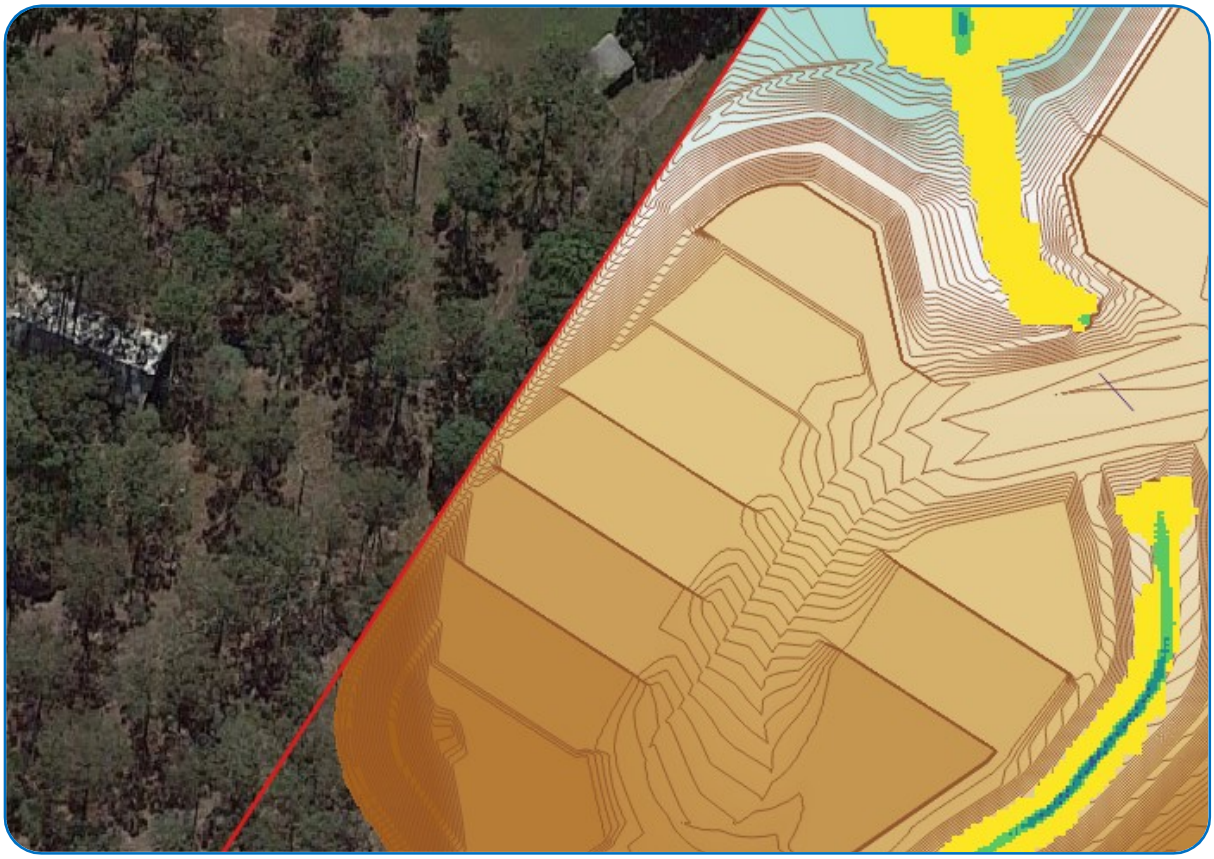


Figure 7.9 0.01 AEP Peak Flood Depth x Velocity – Waterway

7.5 Impact Assessment

In order to assess the impact of the proposed earthworks, a spatial analysis of the peak pre and post-development flood levels has been undertaken. The result of this analysis is presented in Figure 7.10 for 0.01 AEP storm event. These results indicate that the proposed earthworks will not cause any flood increase or nuisance to neighbouring properties.

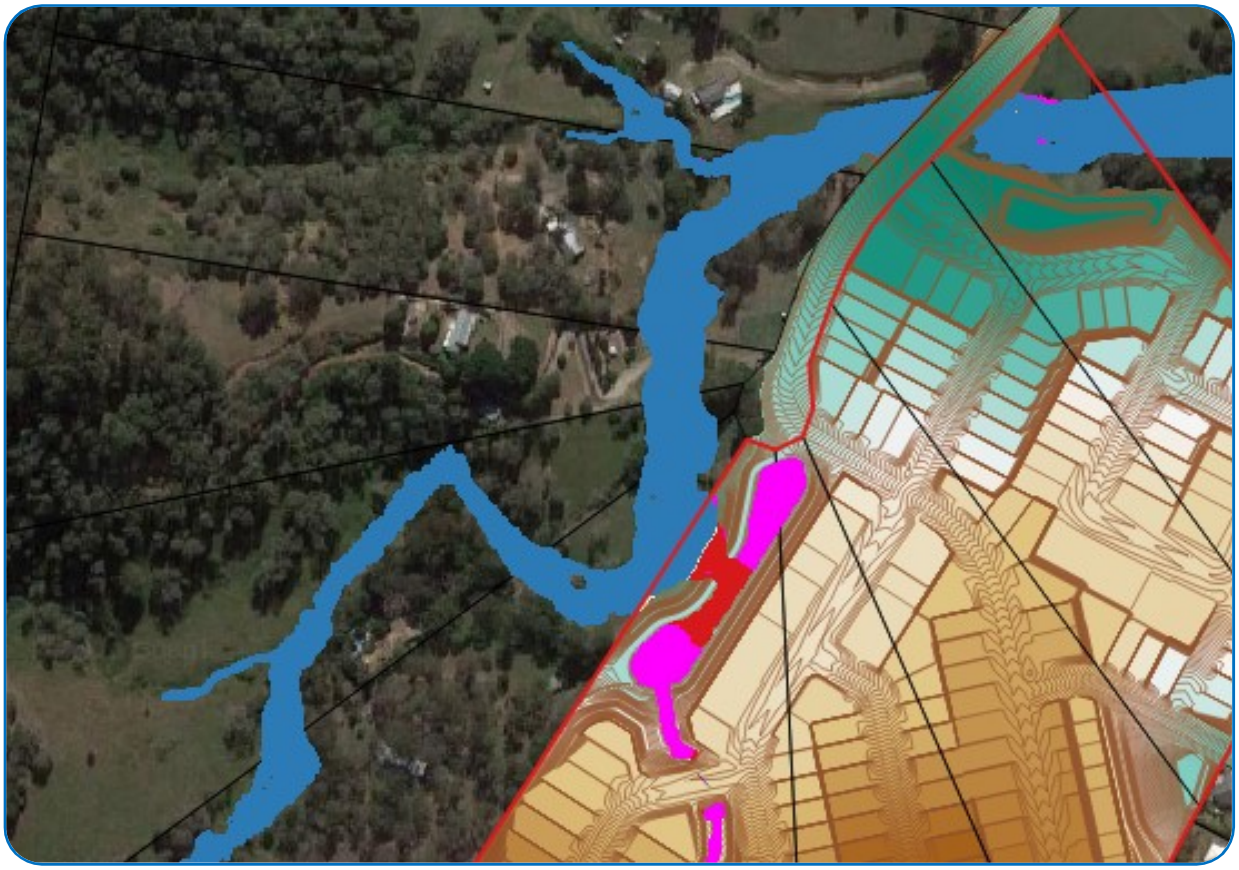


Figure 7.10 0.01 AEP Impact Plot

8 Conclusions

The report presents a comprehensive review of available Stormwater Quality Improvement Devices (SQID) and stormwater Best Management Practices (BMP) to ensure that the proposed development adequately addresses the management of stormwater quantity and quality during the operational phases of the development.

A total of four (4) stormwater treatment basins have been proposed. Three (3) of these basins (Basins A, B and D) will be combined bioretention/detention basin for quality and quantity management. The remaining Basin C, will only be required for treatment of pollutant loads, as catchment reductions in the post-development case have resulted a the reduction of peak discharge.

The results of xpstorm modelling indicate that the inclusion of the three (3) proposed detention basins will mitigate the expected increases in peak discharge at PD-A, PD-B, and PD-D during all modelled AEP events.

All internal catchment will be directed to bioretention measure for quality treatment. MUSIC v6 modelling indicates that reduction target for TSS, TP, TN and gross pollutants of 80%, 60%, 45% and 90% respectively can be achieved for the rainfall data set simulated,

In order to simulate pre- and post-development flooding across the site the 1D - 2D hydrodynamic flood model TUFLOW has been utilised to route stormwater through the upper reaches of Yaun Creek and the other minor streams.

Spatial analysis of the peak pre and post-development flooding indicates that the proposed earthworks required to create the subdivision will not cause any flood increase or nuisance to neighbouring properties.

The level of detail provided within this report is suitable for development assessment only and should not be relied upon for construction purposes. A Detailed Stormwater Management Plan containing detailed engineering designs will be needed to finalise the stormwater concepts presented in this report. Detailed design documentation should be prepared in conjunction with civil operational works.

This report has been reviewed by a Registered Professional Engineer of Queensland (RPEQ), and certification has been provided that if the design parameters set out in this report are included within the development:

- there should be no worsening in peak discharge as a result of the proposed development; and
- stormwater pollutant load reductions in accordance with best practice should be achieved.

9 RPEQ Certification

I am aware that Council may rely upon the contents and findings of this assessment for the purposes of development assessment. In my opinion, the Council can rely upon the information contained within the report and there are no reservations or qualifications in respect to the information other than set out in the report.

I confirm that if the design parameters set out in this report are included within the development:

- there should be no worsening in peak discharge, as a result of the proposed development that would result in actionable damage to downstream properties; and
- stormwater pollutant load reductions in accordance with best practice should be achieved.



..... 16 November 2020.....

Brad Comley **RPEQ 17706** **DATE**

10 References

ANZECC (2000), Australian Water Quality Guidelines for Fresh and Marine Waters, ANZECC, Canberra.

Brisbane City Council (2003), Natural Channel Design Guidelines

Department of Environment and Resource Management 2009, Queensland Water Quality Guidelines September 2009.

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Water by Design, 2010. MUSIC Modelling Guidelines. South East Queensland Healthy Waterways Partnership, Brisbane, Queensland.

Water by Design, 2014. Bioretention Technical Design Guidelines (Version 1.1), Healthy Waterways Ltd, Brisbane.

Appendix A

Healthy Waters Code

City Plan code template

This code template supports the preparation of a development application against either the acceptable outcome(s) or performance outcome(s) contained in the code. Development assessment rules are outlined in **Section 5.3.3** of the City Plan.

Please note:

For assessment against the overall outcomes, refer to the appropriate code.

Impact assessable development requires assessment against the strategic framework prior to using this template.

9.4.5 Healthy waters code

9.4.5.1 Application

This code applies to assessing material change of use or reconfiguring a lot for development where indicated within **Part 5 Tables of assessment** unless either of the following circumstances apply:

(1) No increase in impervious area is required onsite for the development.

(2) A stormwater quality and quantity management plan previously approved by the Council has been fully implemented within the existing development layout.

Note: Where a development proposal meets either (1) and (2) above, this code is not applicable.

When using this code, reference should be made to **Section 5.3.2** and, where applicable, **Section 5.3.3,** in **Part 5**.

9.4.5.2 Purpose

(1) The purpose of the Healthy waters code is to protect the quality of the city's waters from the impacts of development.

(2) The purpose of the code will be achieved through the following overall outcomes:

(a) Urban stormwater quality management, wastewater management, and management of waters are based on the following principles:

(i) Development and construction activities are conducted in accordance with design objectives to achieve the water quality objectives, as specified in the *Environmental Protection (Water) Policy 2009*.

(ii) The ongoing management of urban stormwater quality incorporates design objectives that reflect the regional climate and the site's landscape characteristics.

(iii) Development is undertaken in accordance with best practice environmental management.

(iv) Development avoids adverse impacts on the City of Gold Coast's waters or, where this is not feasible, adverse impacts are minimised.

(b) Water resource catchments are protected from contamination by chemicals.

(c) The drainage capacity of the Woongoolba Flood Mitigation Scheme Area for rainfall events up to 1 in 10 year 72 hours is maintained (contained within the Scheme drains within a 4 day period) and this capacity is not be eroded due to cumulative impact of development.

9.4.5.3 Criteria for assessment

Table 9.4.5-2: Healthy waters code – for assessable development

Performance outcomes	Acceptable outcomes	Does the proposal meet the acceptable outcome? If not, justify how the proposal meets either the performance outcome or overall outcome
Stormwater quality		
PO1 Development appropriately manages stormwater quality to: (a) protect natural ecosystems; (b) integrate stormwater treatment into the urban landscape; (c) protect water quality; (d) reduce runoff and peak flows; and (e) meet the water quality objectives and environmental values for Queensland waters. **Note: Water quality objectives and environmental values for Queensland waters are contained within *Schedule 1* of the *Environmental Protection (Water) Policy 2009*. Water quality objectives are locally specific and vary between and within river catchments.**	**AO1.1** Where development is: (a) for a dwelling house, dual occupancy or multiple dwelling on a lot less than 5000m²; or (b) light industry or business activity on a lot less than 2500m², it complies with the 'Deemed to comply' requirements detailed in the **Stormwater quality management guidelines** in **SC6.9 City Plan policy – Land development guidelines**.	**N/A**
	AO1.2 Where development is: (a) for a dwelling house, dual occupancy or multiple dwelling on a lot equal to or more than 5000m² but less than 1.25 ha; or (b) light industry or business activity on a lot equal to or more than 2500m² but less than 1.25 ha; (c) it complies with the 'Deemed to comply' requirements detailed in the **Stormwater quality management guidelines** in **SC6.9 City Plan policy – Land development guidelines**.	**N/A**

Performance outcomes	Acceptable outcomes	Does the proposal meet the acceptable outcome? If not, justify how the proposal meets either the performance outcome or overall outcome
	AO1.3 Where development is not listed in AO1 a Stormwater Quality Management Plan is prepared by a suitably-qualified person in accordance with the **Stormwater quality management guidelines** in **SC6.9 City Plan policy – Land development guidelines**.	**COMPLIES WITH AO1.3** This Stormwater Management Plan has been prepared by a suitably-qualified person and is in accordance with the Stormwater quality management guidelines in SC6.9 City Plan policy – Land development guidelines.
Stormwater quantity		
PO2 Stormwater quantity management outcomes demonstrate no adverse impact on stormwater flooding or the drainage of properties external to the subject site.	**AO2** A stormwater quantity management plan is prepared by a suitably qualified person and demonstrates: (a) achievable stormwater quantity control measures for discharge during both the construction and operational phases of development designed in accordance with the *Queensland Urban Drainage Manual (QUDM)* unless subject to specific requirements of **SC6.9 City Plan policy – Land development guidelines**; (b) on-site detention systems that are designed to restrict peak outflows for Q2, Q5, Q10, Q20 Q50 and Q100 to pre-development conditions.	**COMPLIES WITH AO2** This Stormwater Management Plan has been prepared by a suitably-qualified person in accordance with the QUDM and includes on-site detention systems designed to restrict peak outflows for Q2, Q5, Q10, Q20 Q50 and Q100 to pre-development conditions.
Woongoolba Flood Mitigation Scheme Catchment Area		
PO3 In the Woongoolba Flood Mitigation Scheme Catchment Area, shown on the **Water catchments and dual reticulation overlay map**, peak outflow and its timing for Q2, Q5 and	**AO3** No acceptable outcome provided.	**N/A**

Performance outcomes	Acceptable outcomes	Does the proposal meet the acceptable outcome? If not, justify how the proposal meets either the performance outcome or overall outcome
Q10 for rainfall events up to 72 hours does not change as a result of development.		
Protection of natural flows (discharge)		
PO4 Construction and operational activities avoid or minimise adverse impacts of altered stormwater quality and quantity.	**AO4** Development achieves the Frequent Flow Management and Waterway Stability Management design objectives as stated within *Chapter 2 – Section 2.4.2 of the Urban Stormwater Quality Planning Guidelines 2010*. **Note: A site Stormwater Quality and Quantity Management Report prepared by a suitably qualified person is Council's preferred method required to be submitted to Council to address Acceptable outcome AO6.** **The intent in the above guidelines (*Urban Stormwater Quality Planning Guidelines 2010*) can be achieved by adopting Water Sensitive Urban Design (WSUD) concepts within the development.**	**COMPLIES WITH AO4** This Stormwater Management Plan has been prepared by a suitably-qualified person and adopts Water Sensitive Urban Design (WSUD) concepts within the development to manage stormwater quality and quantity.
Wastewater management		
PO5 Development does not discharge wastewater to receiving waters or areas external to the site unless demonstrated to be the best-practice environmental management for that site and takes into consideration: (a) the applicable water quality objectives for the receiving waters; and (b) the potential adverse impact on ecosystem health of receiving waters.	**AO5** Where the development involves the discharge of wastewater, a Wastewater Management Plan (WWMP) is prepared, demonstrating compliance with the performance outcome, by a suitably qualified person and submitted to the Council, detailing all of the following: (a) wastewater type; (b) climatic conditions; (c) water quality objectives;	**N/A**

Performance outcomes	Acceptable outcomes	Does the proposal meet the acceptable outcome? If not, justify how the proposal meets either the performance outcome or overall outcome
	(d) best-practice environmental management; (e) waste management hierarchy; and (f) the WWMP provides for the management of wastewater in accordance with a wastewater management hierarchy that: (i) avoids wastewater discharge to waterways; or (ii) if wastewater discharge to the environment cannot practicably be avoided wastewater discharge to waterways is minimised through re-use, recycling, recovery and treatment for disposal to sewer, surface water and groundwater.	

Performance outcomes	Acceptable outcomes	Does the proposal meet the acceptable outcome? If not, justify how the proposal meets either the performance outcome or overall outcome	
Erosion and sediment control (ESC)			
PO6 Development does not cause erosion or allow sediments to leave the site.	**AO6** An **Erosion hazard assessment** completed in accordance with the criteria in **Table 9.4.5-3** is undertaken to establish the level of risk for soil erosion and sediment pollution to the environment. **Where the Erosion hazard assessment has a risk score less than or equal to 10:** A deemed to comply report is prepared by a suitably qualified person for Council approval, including conceptual location and design drawings of each treatment measure in plan and section views, in accordance with the *Best Practice Erosion and Sediment Control: International Erosion Control Association, (IECA) 2008, Australasia Chapter 2008.* **Where the Erosion hazard assessment has a risk score greater than 10:** A conceptual erosion and sediment control plan (ESCP) is prepared by a suitably qualified person for Council approval in accordance with the *Best Practice Erosion and Sediment Control: International Erosion Control Association, (IECA) 2008, Australasia Chapter 2008.*	**COMPLIES VIA CONDITIONS** It is considered reasonable that conditions be placed on the development approval to ensure that am Erosion and Sediment Control plan is developed in conjunction with the OPW for Change to Ground Level.	

Appendix B

Stormwater Management Design Drawings

VERSION: C

DATE : 16.11.20

CONCEPTUAL STORMWATER MANAGEMENT DRAWING SET

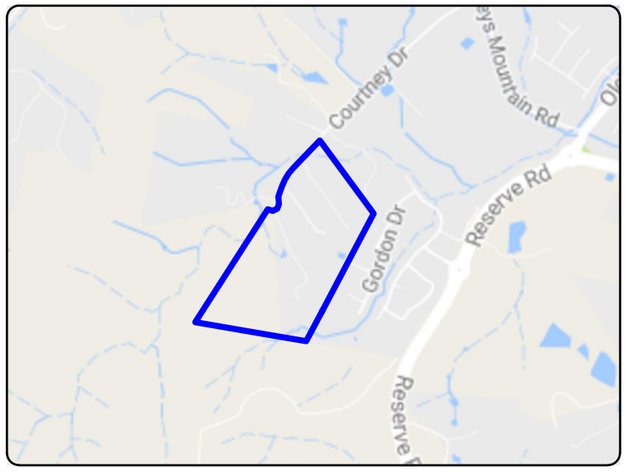
Courtney Drive

for

Gallery Homes Pty Ltd

Project No: BC-16090

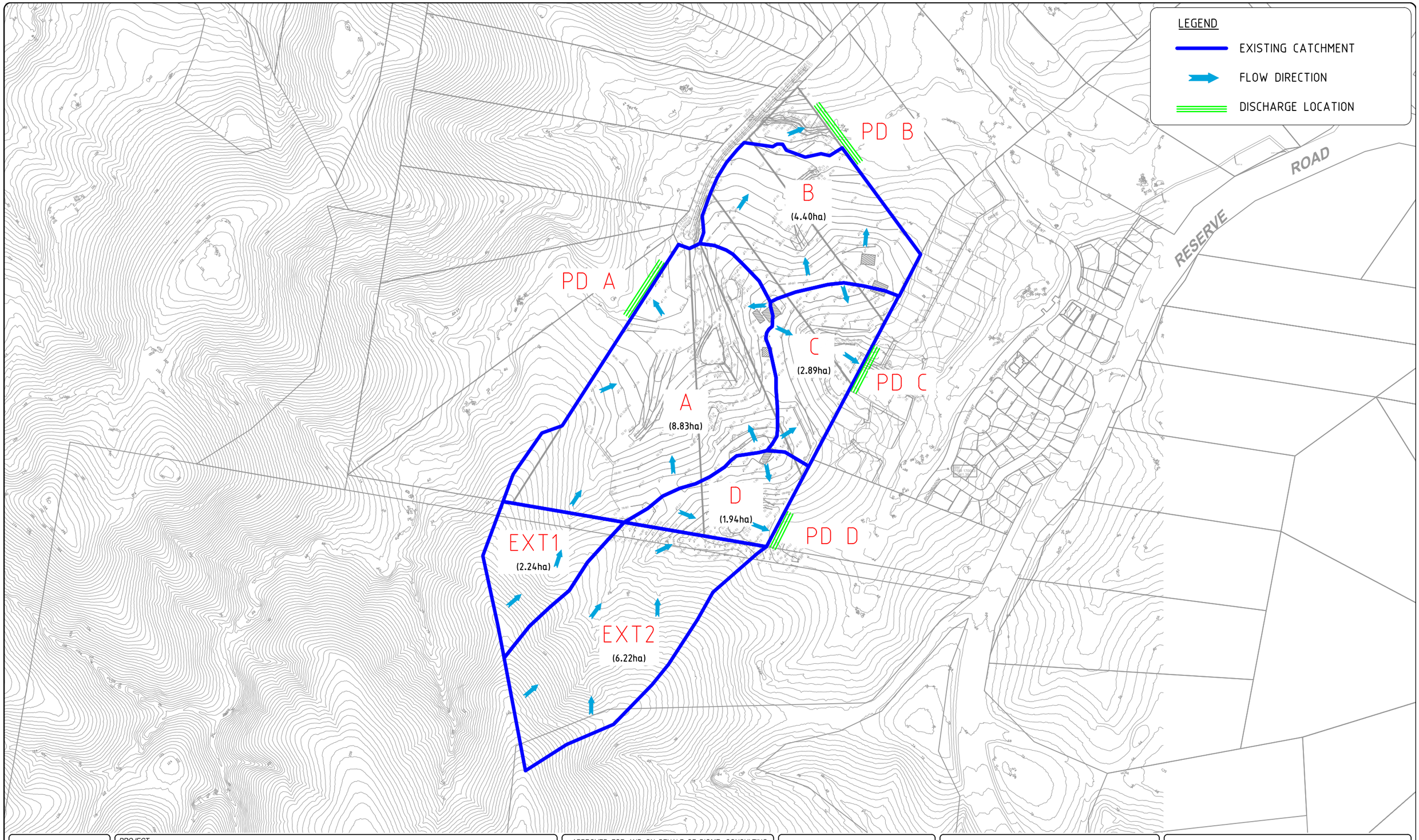
November 2020



Site Location

SCHEDULE OF DRAWINGS	
Drawing No.	Drawing Title
DWG-100	LOCALITY AND DRAWING INDEX PLAN
DWG-200	EXISTING CATCHMENT PLAN
DWG-201	DEVELOPED CATCHMENT PLAN
DWG-210	MUSIC CATCHMENT PLAN
DWG-300	OPERATIONAL CONTROL PLAN
DWG-310	BASIN A LAYOUT AND CROSS SECTION
DWG-311	BASIN B LAYOUT AND CROSS SECTION
DWG-312	BASIN C LAYOUT AND CROSS SECTION
DWG-313	BASIN D LAYOUT AND CROSS SECTION
DWG-314	WATERWAY & DRAINAGE EASEMENT LAYOUT AND CROSS SECTIONS
DWG-320	BASIN A - INLET AND OUTLET DETAILS
DWG-321	BASIN B - INLET AND OUTLET DETAILS
DWG-322	BASIN C - INLET AND OUTLET DETAILS
DWG-323	BASIN D - INLET AND OUTLET DETAILS
DWG-330	BIORETENTION - FILTER MEDIUM DETAILS
DWG-340	BASIN A - DEPTH INDICATOR AND DEEP WARNING SIGNAGE

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16.11.20

PROJECT No. :
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DRAWING No. :
DWG-200

REVISION No. :
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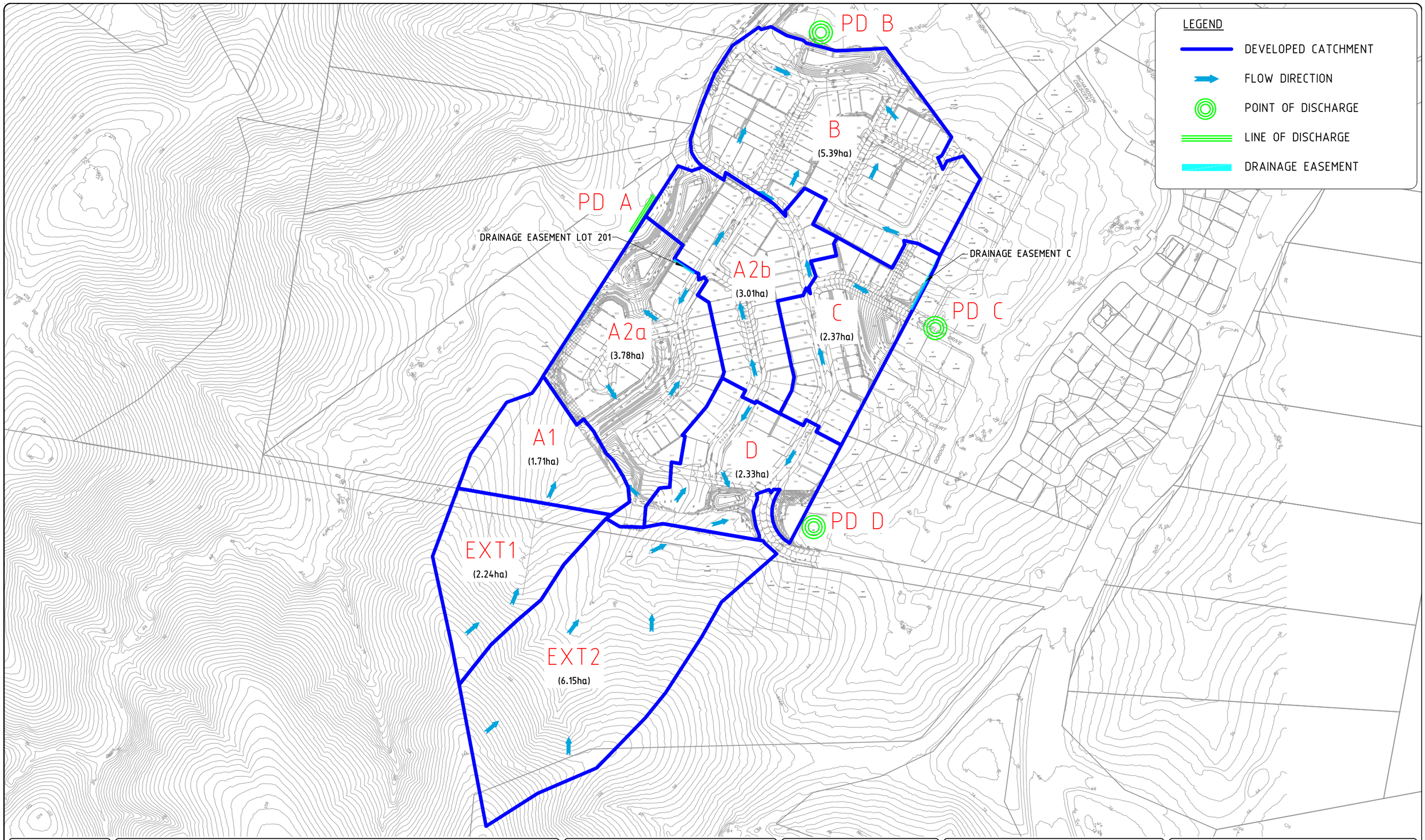
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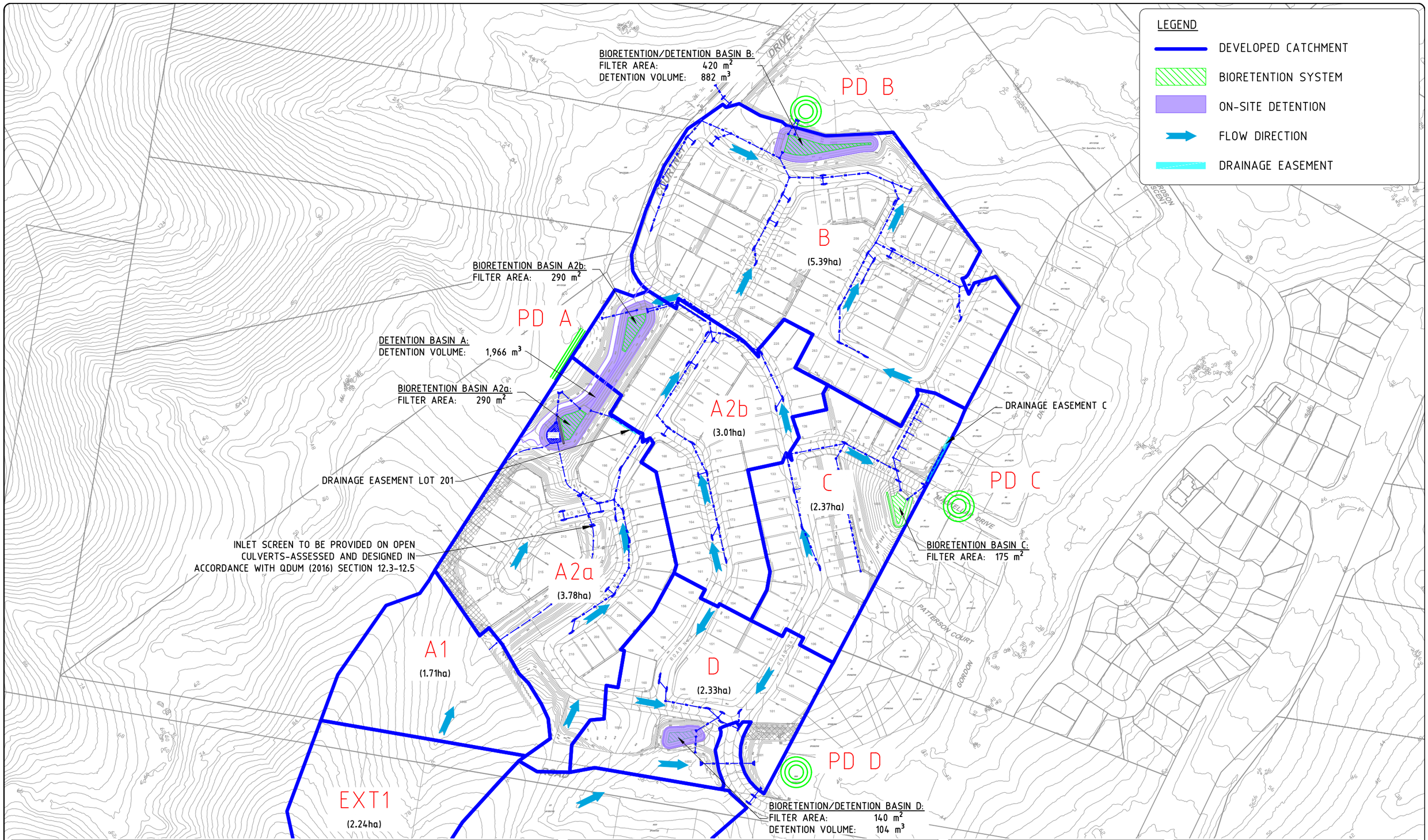
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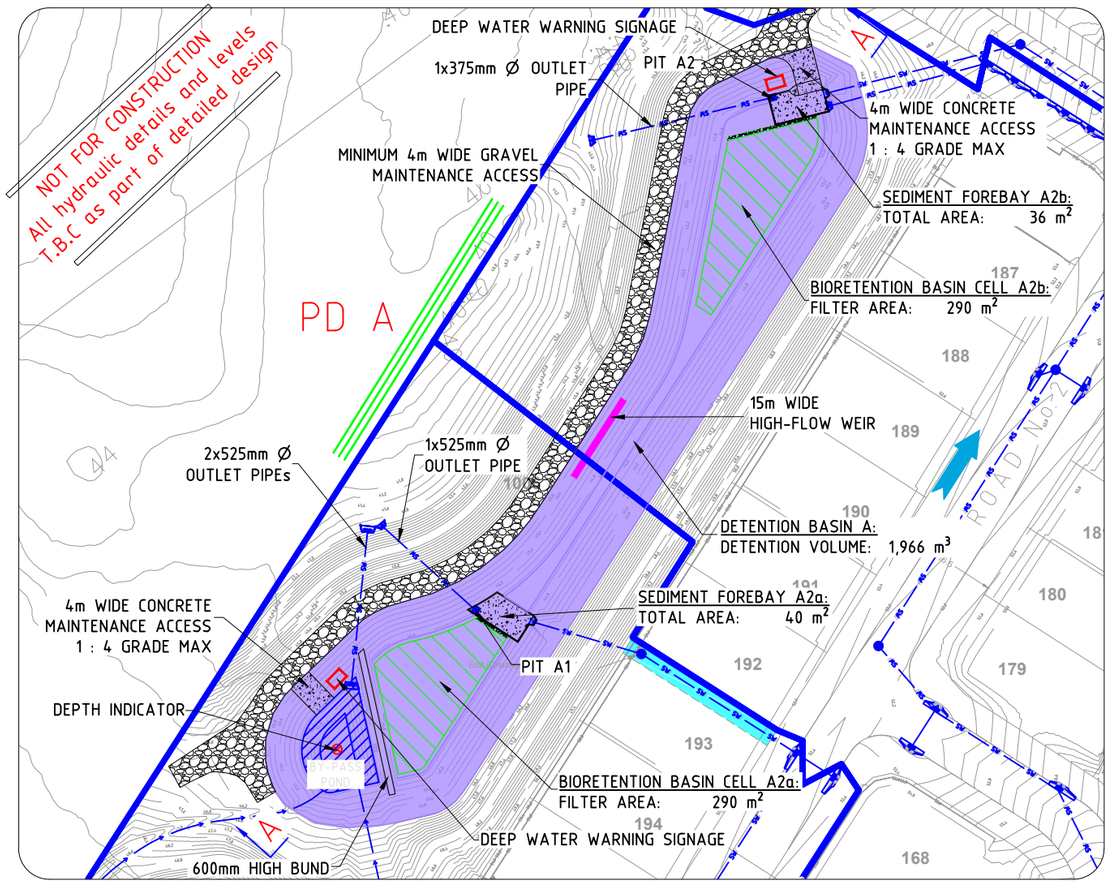
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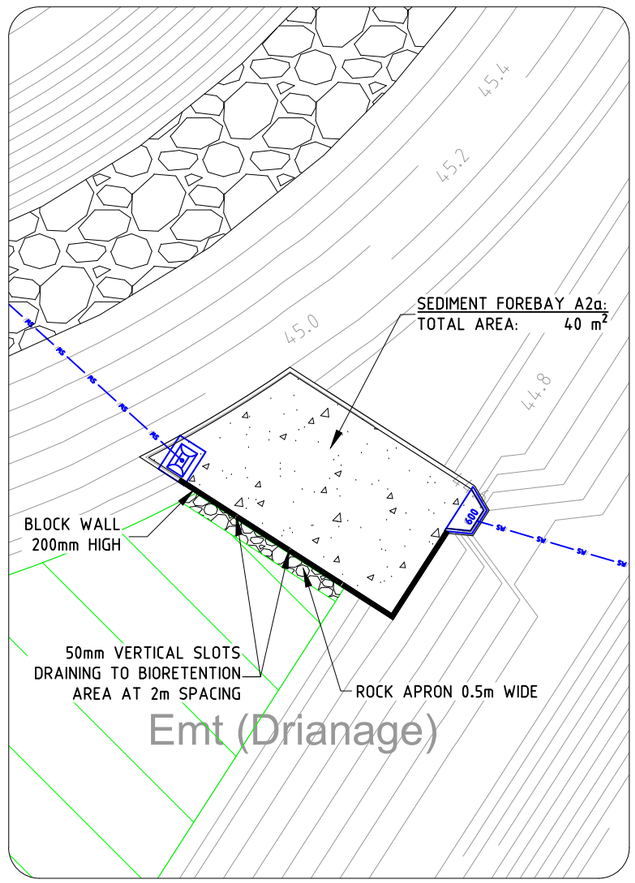
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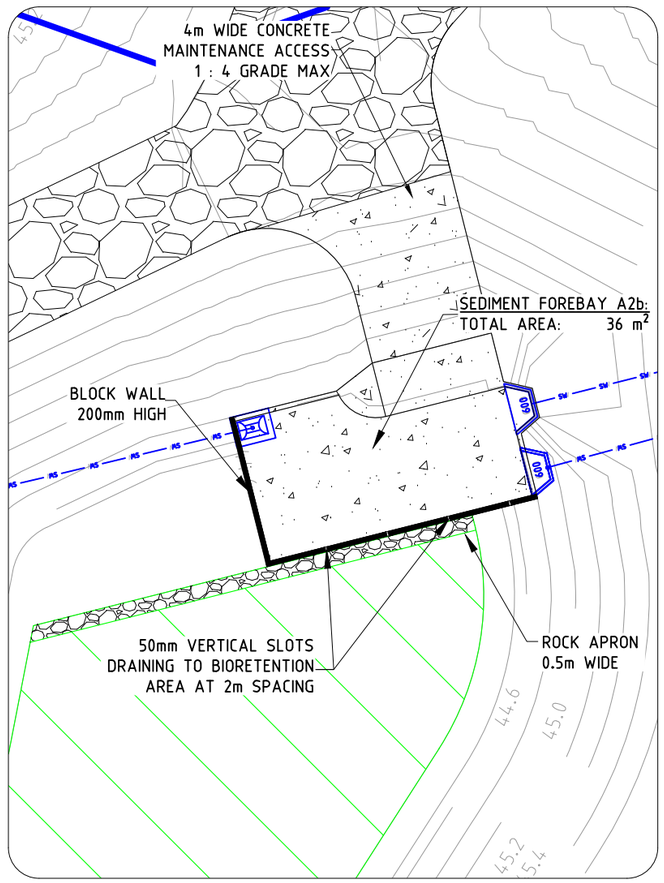
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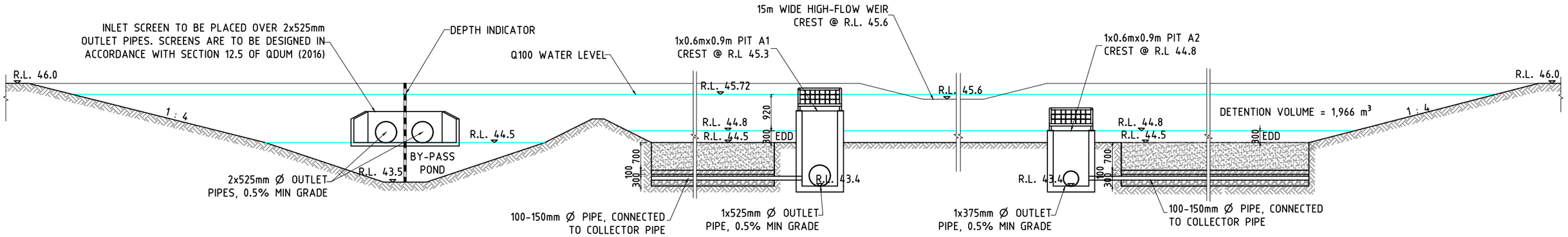
SEDIMENT FOREBAY A2a PLAN VIEW

1:200



SEDIMENT FOREBAY A2b PLAN VIEW

1:200



BASIN A CROSS SECTION A - A

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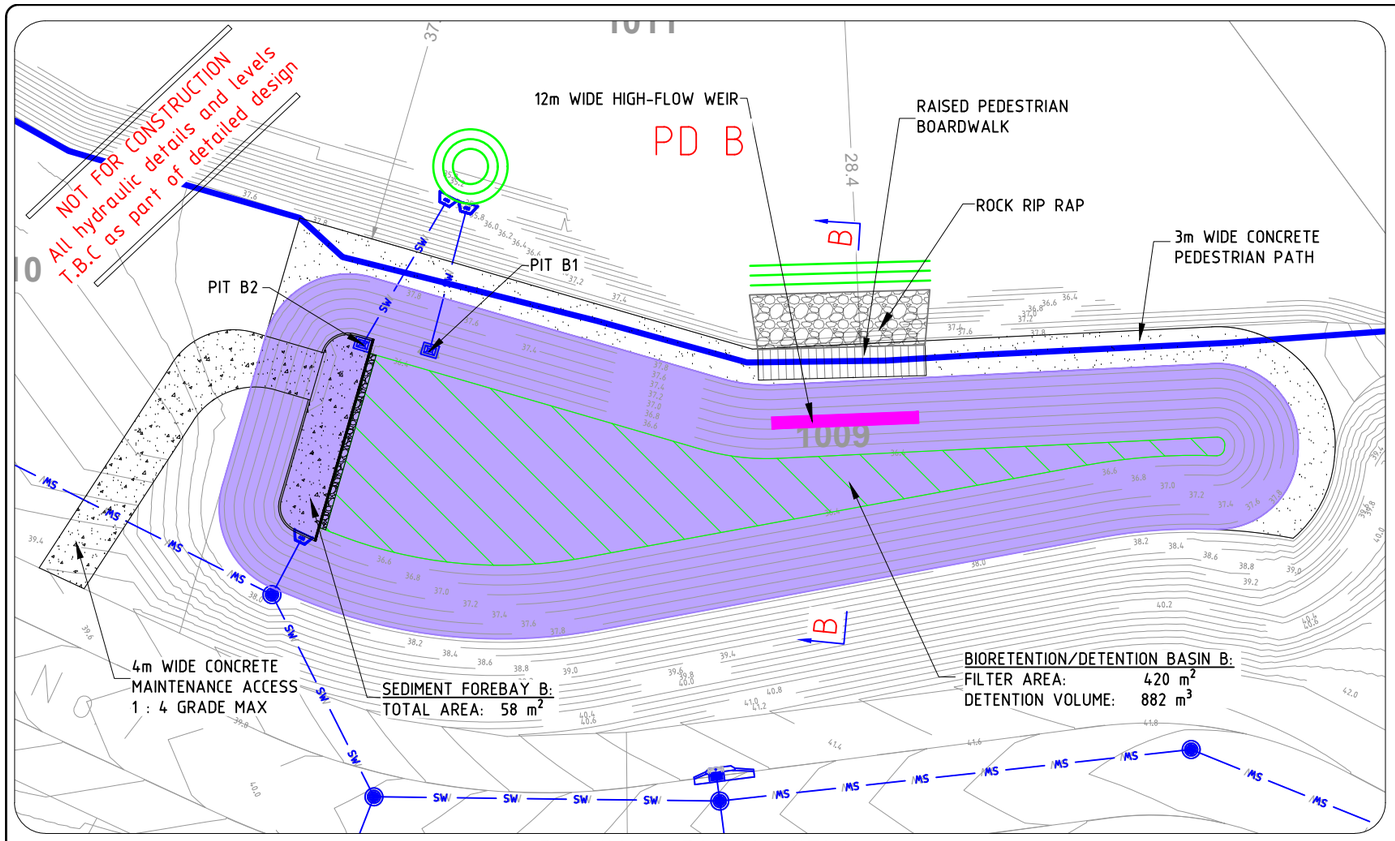
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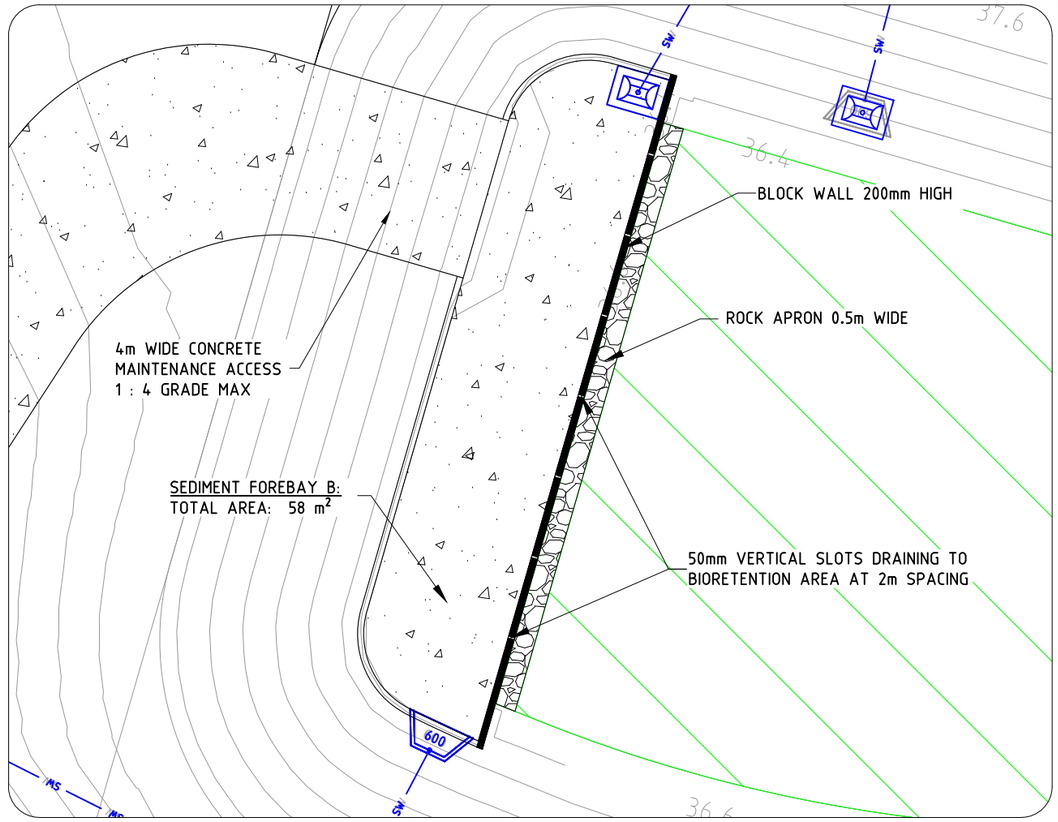
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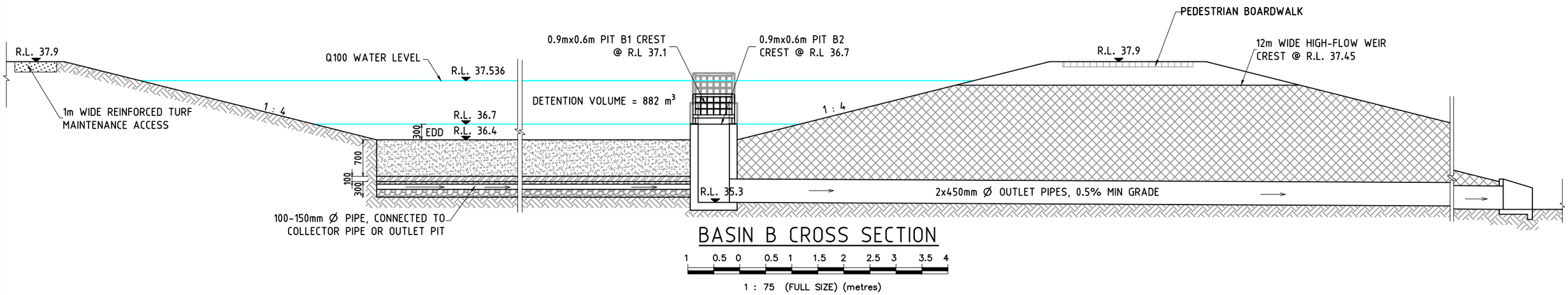
BASIN B PLAN VIEW

1:500



SEDIMENT FOREBAY B PLAN VIEW

1:150



BASIN B CROSS SECTION

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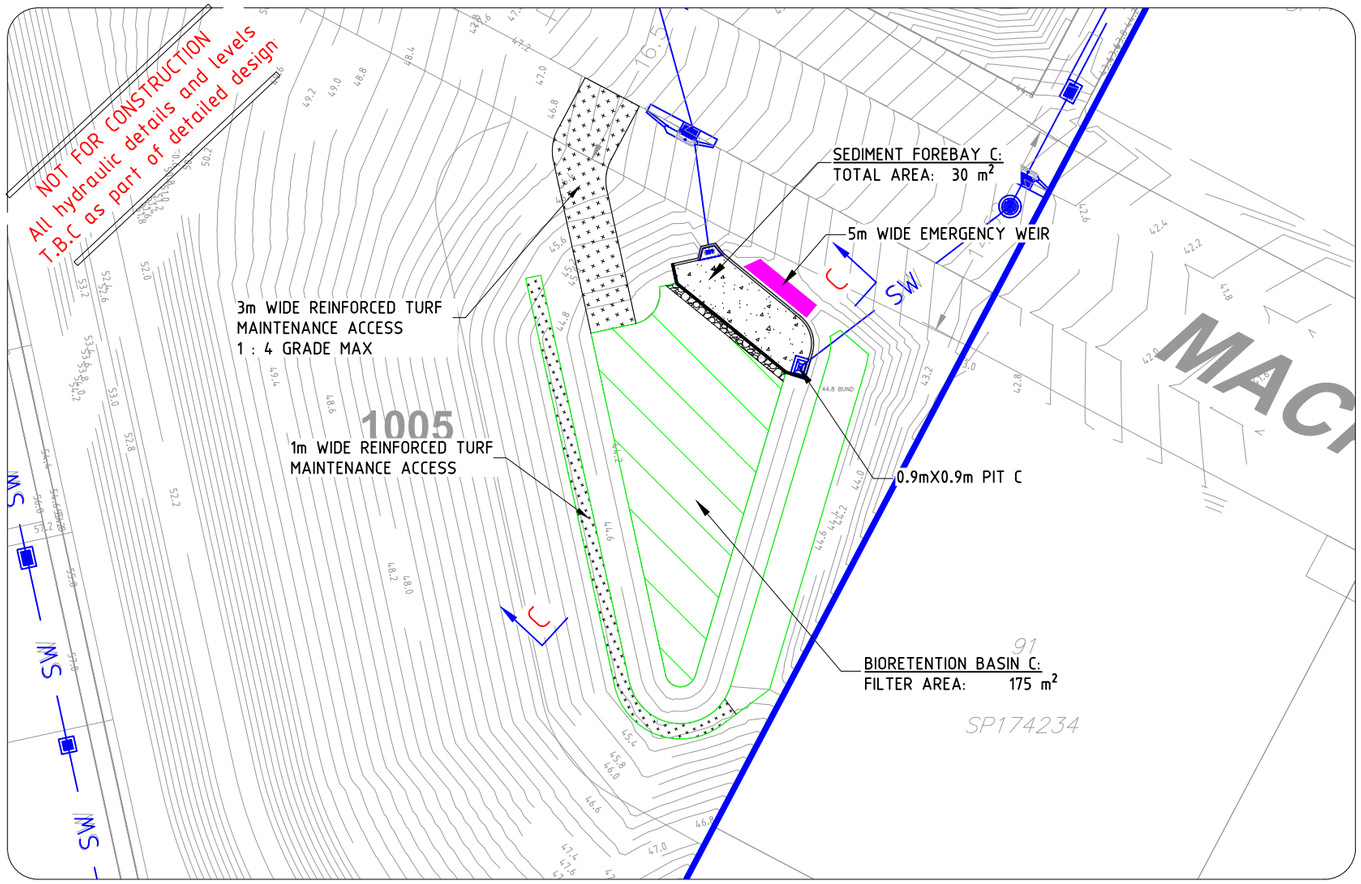
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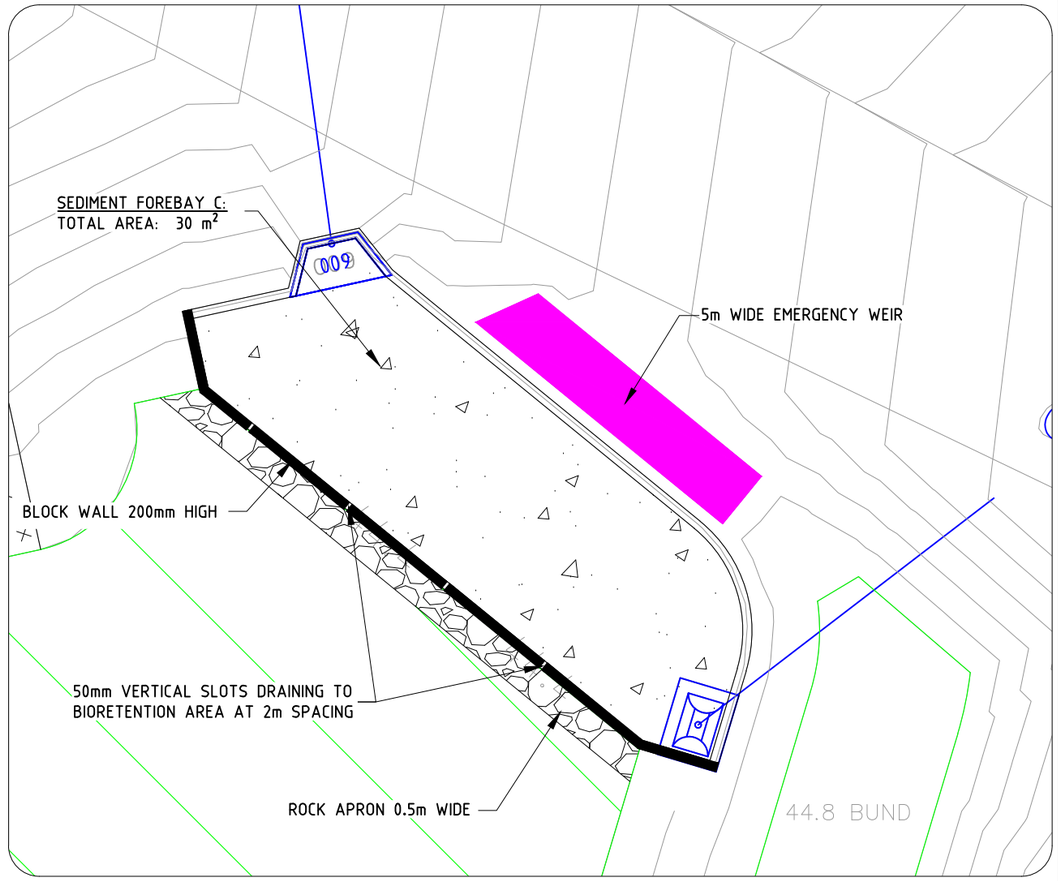
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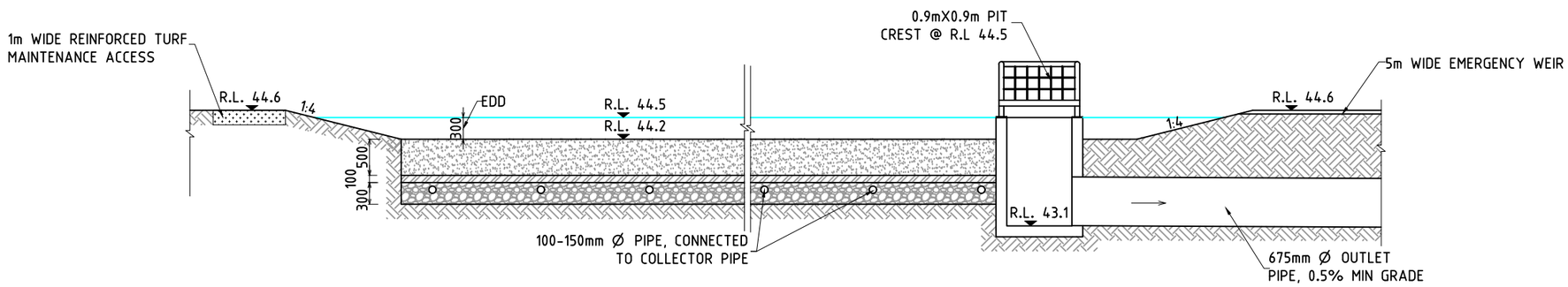
BASIN C PLAN VIEW

SCALE: 1/250



SEDIMENT FOREBAY C PLAN VIEW

SCALE: 1/100



BASIN C CROSS SECTION C - C

1 0.5 0 0.5 1 1.5 2 2.5 3 3.5 4

1 : 75 (FULL SIZE) (metres)

DATE
16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-312

REVISION No. :
C

PROJECT
PROPOSED SUBDIVISION

DRAWING TITLE
BASIN C LAYOUT AND CROSS SECTION

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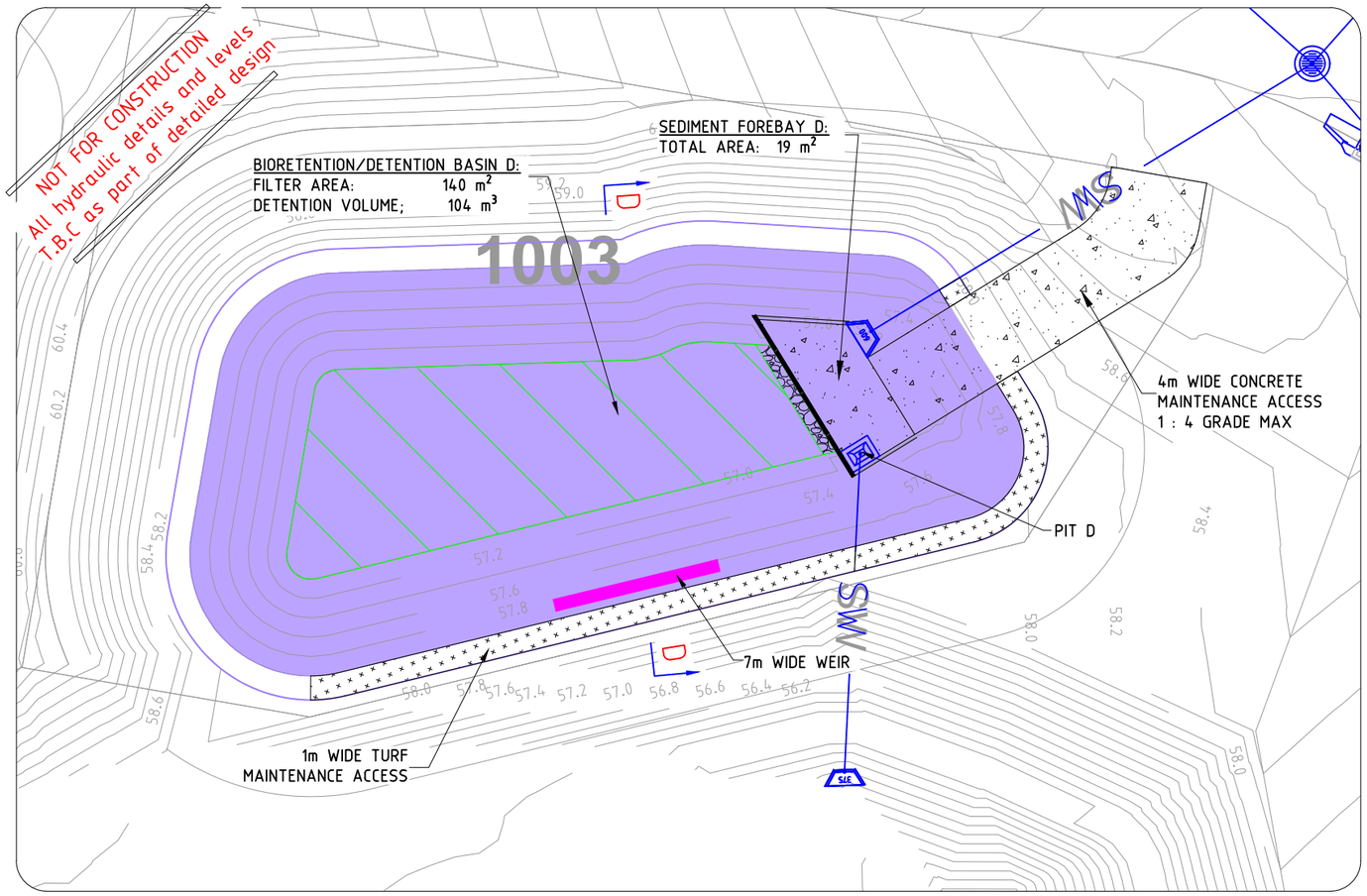
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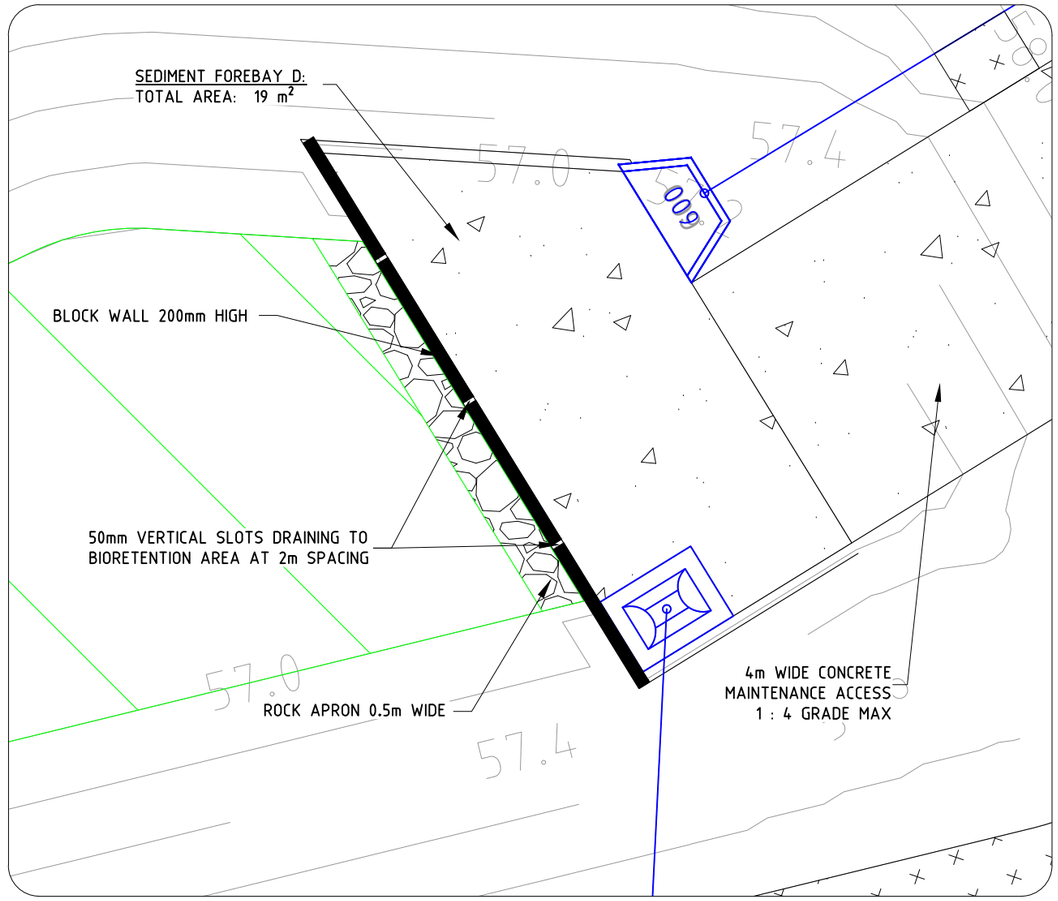
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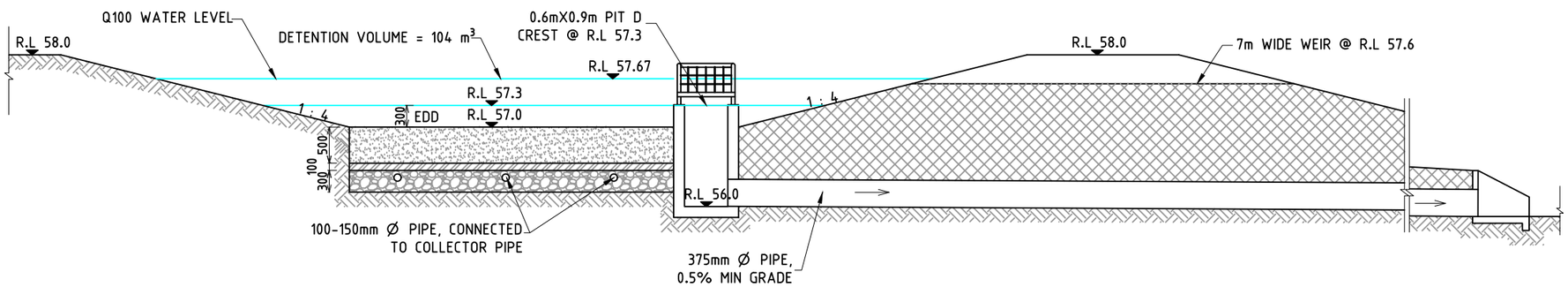
BASIN D PLAN VIEW

SCALE: 1/250



SEDIMENT FOREBAY D PLAN VIEW

SCALE: 1/75



BASIN D CROSS SECTION

1 0.5 0 0.5 1 1.5 2 2.5 3 3.5 4

1 : 75 (FULL SIZE) (metres)

DATE
16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-313

REVISION No. :
C

PROJECT
PROPOSED SUBDIVISION

DRAWING TITLE
BASIN D - LAYOUT AND CROSS SECTION

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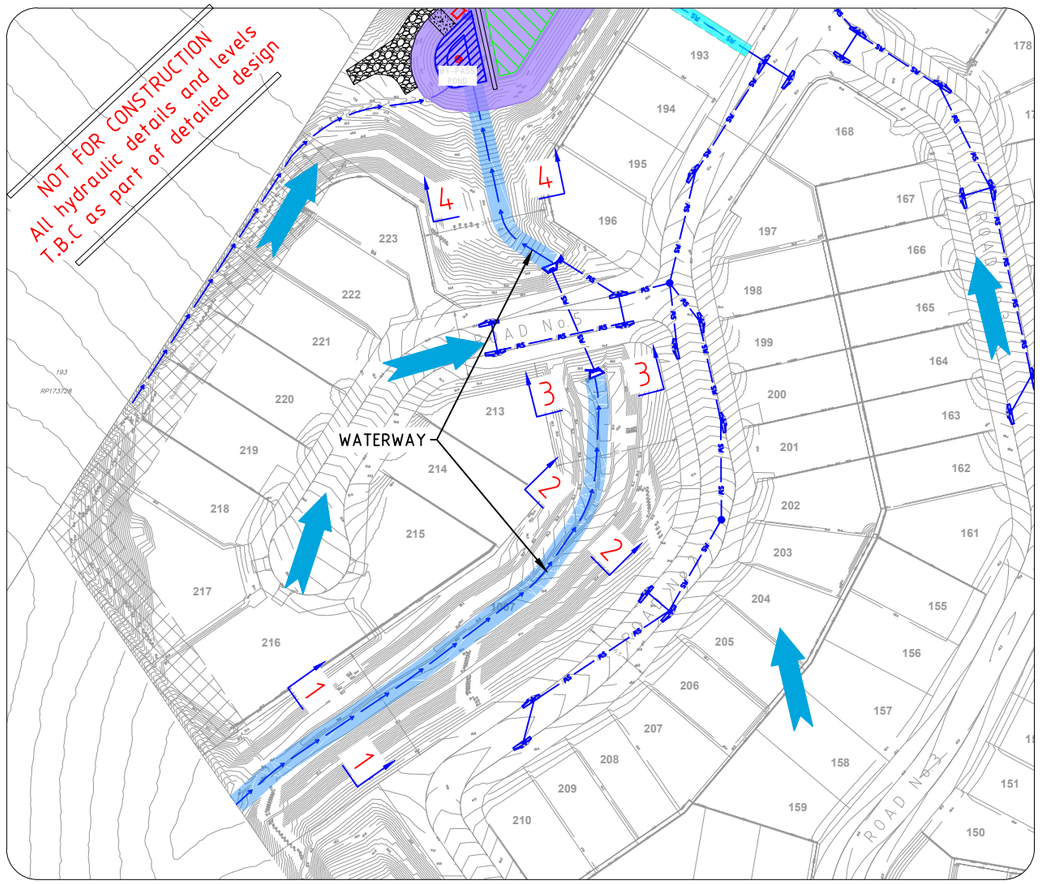
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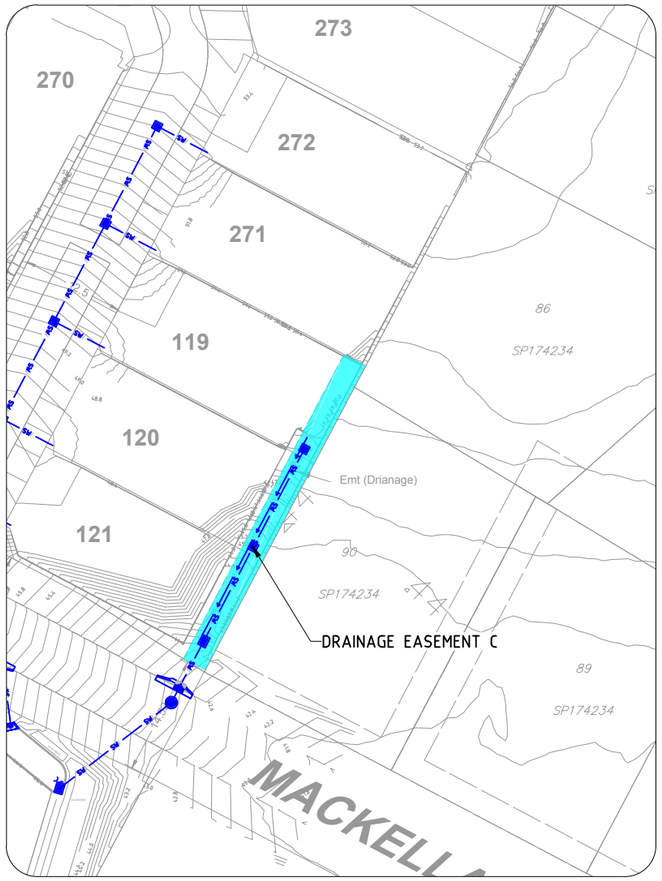
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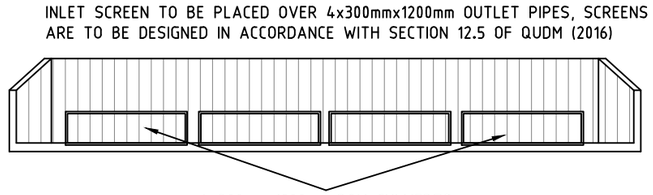
WATERWAY & CHANNEL A2a PLAN VIEW

SCALE 1:1500



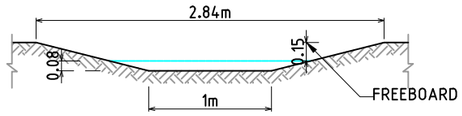
CHANNEL C PLAN VIEW

SCALE 1:750



WATERWAY CULVERT CROSSING CROSS SECTION

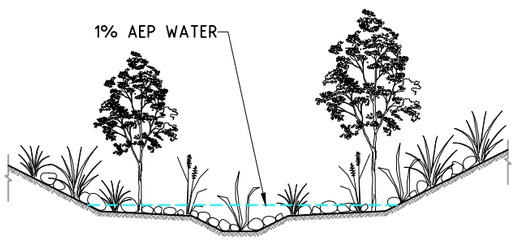
N.T.S



DRAINAGE EASEMENT C CROSS SECTION

0.5 0 0.5 1.0 1.5 2.0 2.5

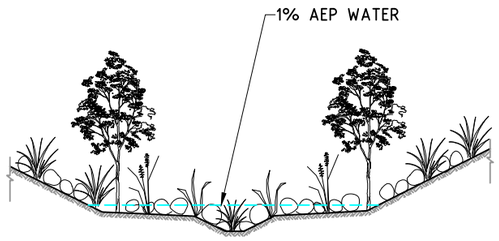
1 : 50 (FULL SIZE) (metres)



WATERWAY SECTION 1 - 1

2 0 2 4 6 8 10

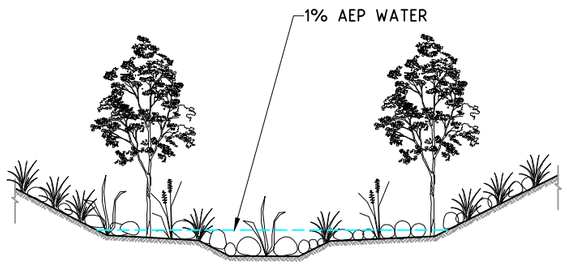
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WATERWAY SECTION 2 - 2

2 0 2 4 6 8 10

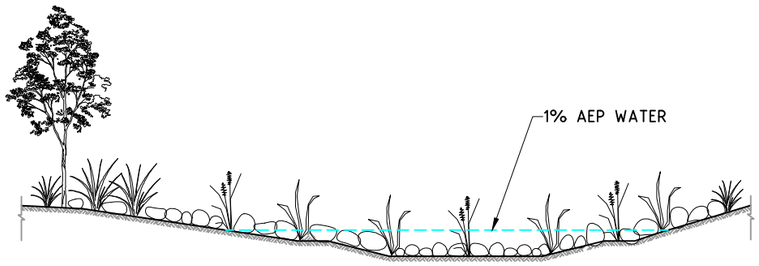
1 : 200 (FULL SIZE) (metres)



WATERWAY SECTION 3 - 3

2 0 2 4 6 8 10

1 : 200 (FULL SIZE) (metres)



WATERWAY SECTION 4 - 4

2 0 2 4 6 8 10

1 : 200 (FULL SIZE) (metres)

DATE
16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-314

REVISION No. :
C

PROJECT
PROPOSED SUBDIVISION

DRAWING TITLE
WATERWAY & DRAINAGE EASEMENT - LAYOUT AND CROSS SECTIONS

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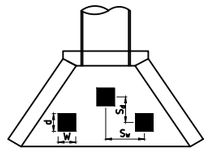
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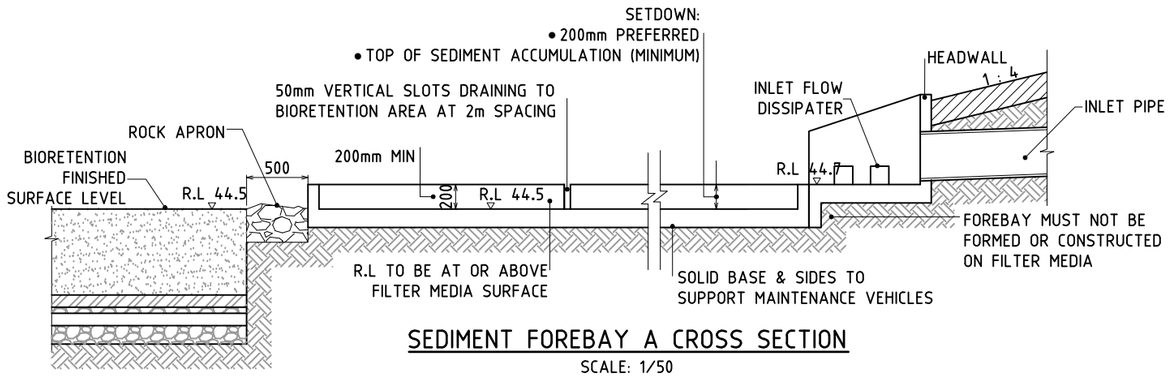
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NOT FOR CONSTRUCTION
All hydraulic details and levels T.B.C as part of detailed design

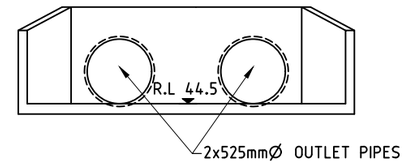


NOTE: ENERGY DISSIPATER LAYOUT TO BE DESIGNED AS PER BRISBANE CITY COUNCIL'S STORMWATER OUTLETS IN PARKS AND WATERWAYS

INLET SCOUR PROTECTION DETAILS
N.T.S

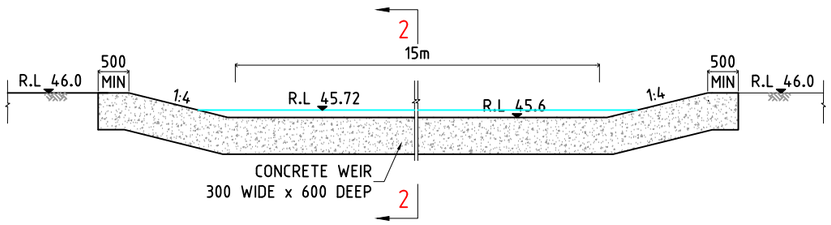


SEDIMENT FOREBAY A CROSS SECTION
SCALE: 1/50

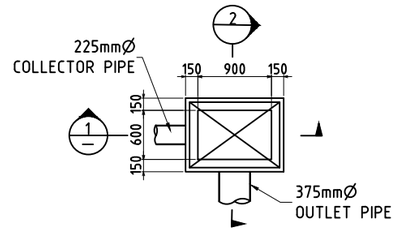


HEADWALL
(2x525mm OUTLET PIPES)
SCALE: 1/50

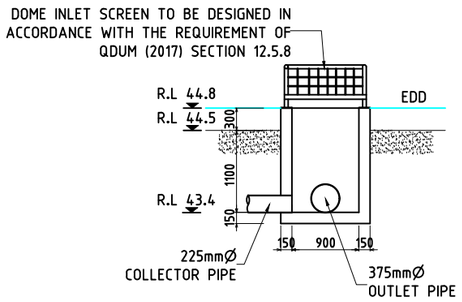
INLET DETAILS



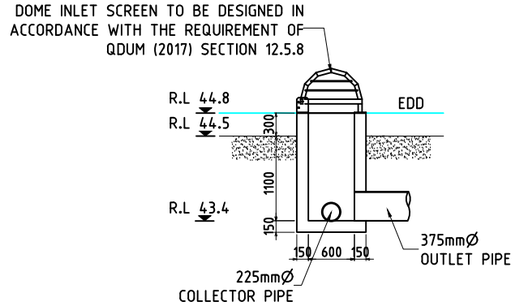
WEIR DETAIL SECTION 1-1
SCALE: 1/100



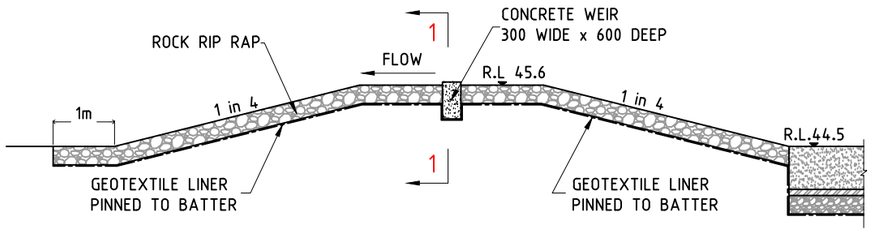
OVERFLOW STRUCTURE PLAN - PIT A1
SCALE: 1/75



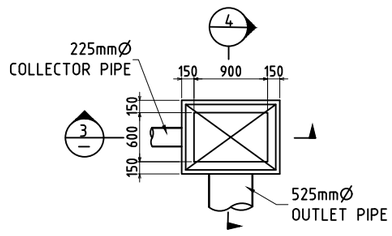
SECTION 1 - 1
SCALE: 1/75



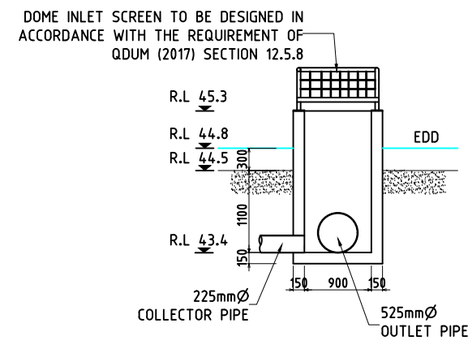
SECTION 2 - 2
SCALE: 1/75



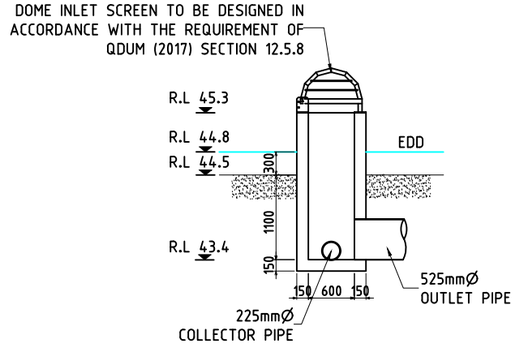
WEIR CROSS SECTION 2-2
SCALE: 1/100



OVERFLOW STRUCTURE PLAN - PIT A2
SCALE: 1/75



SECTION 3 - 3
SCALE: 1/75



SECTION 4 - 4
SCALE: 1/75

OUTLET DETAILS

DATE
16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-320

REVISION No. :
C

PROJECT
PROPOSED SUBDIVISION

DRAWING TITLE
BASIN A - INLET AND OUTLET DETAILS

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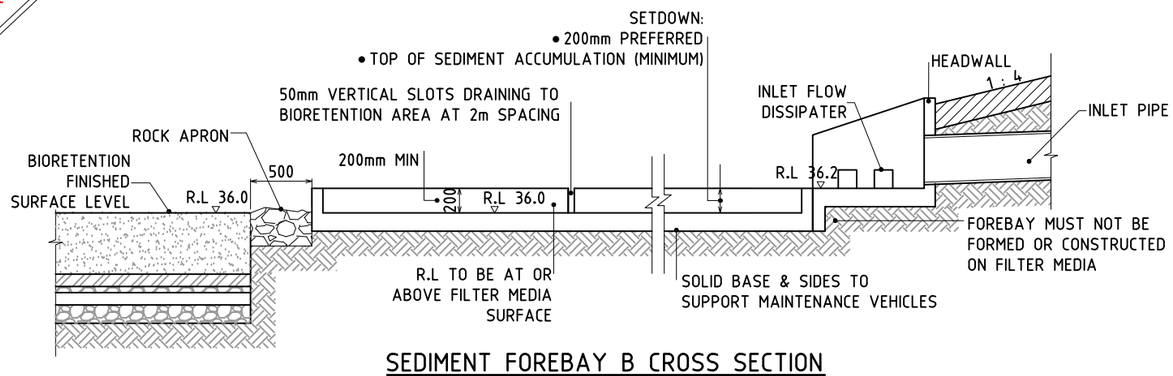
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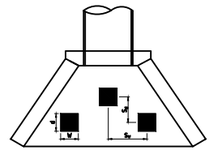
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All hydraulic details and levels T.B.C as part of detailed design



SEDIMENT FOREBAY B CROSS SECTION

SCALE: 1/50

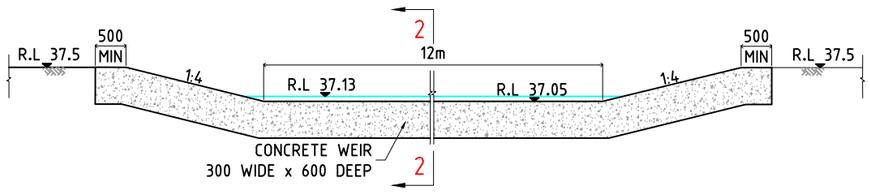


NOTE: ENERGY DISSIPATER LAYOUT TO BE DESIGNED AS PER BRISBANE CITY COUNCIL'S STORMWATER OUTLETS IN PARKS AND WATERWAYS

INLET SCOUR PROTECTION DETAILS

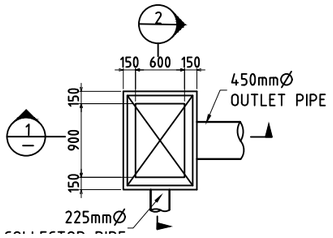
N.T.S

INLET DETAILS



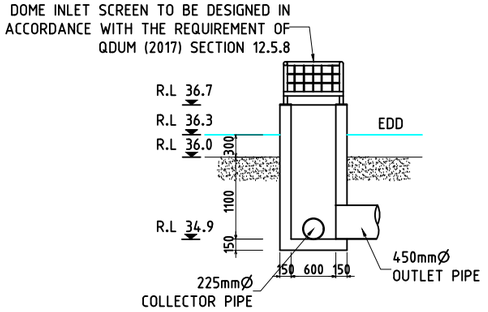
WEIR DETAIL SECTION 1-1

SCALE: 1/100



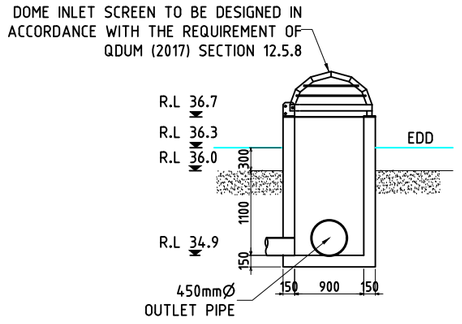
OVERFLOW STRUCTURE PLAN - PIT B1

SCALE: 1/75



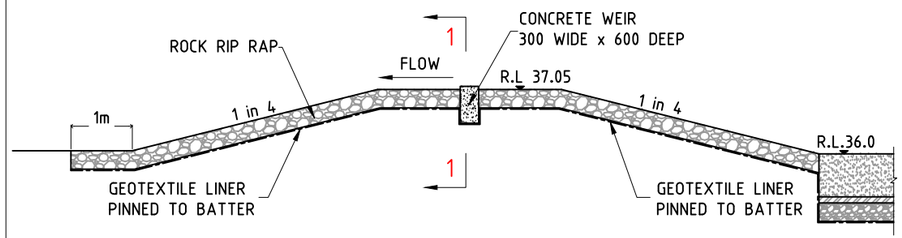
SECTION 1 - 1

SCALE: 1/75



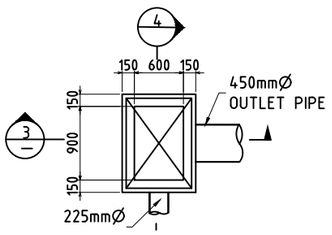
SECTION 2 - 2

SCALE: 1/75



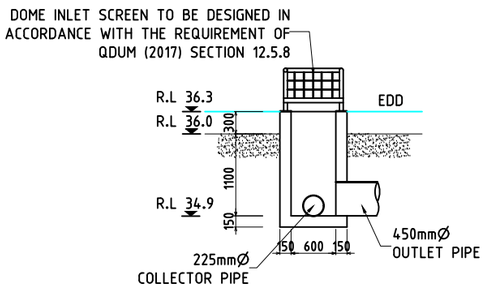
WEIR CROSS SECTION 2-2

SCALE: 1/100



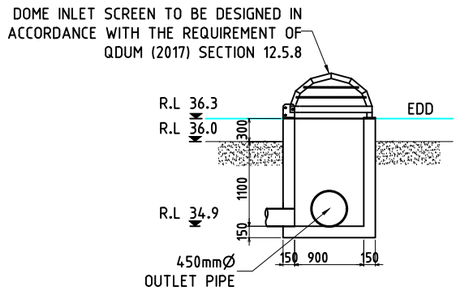
OVERFLOW STRUCTURE PLAN - PIT B2

SCALE: 1/75



SECTION 3 - 3

SCALE: 1/75



SECTION 4 - 4

SCALE: 1/75

OUTLET DETAILS

DATE
16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-321

REVISION No. :
C

PROJECT
PROPOSED SUBDIVISION

DRAWING TITLE
BASIN B - INLET AND OUTLET DETAILS

CLIENT
GALLERY HOMES PTY LTD

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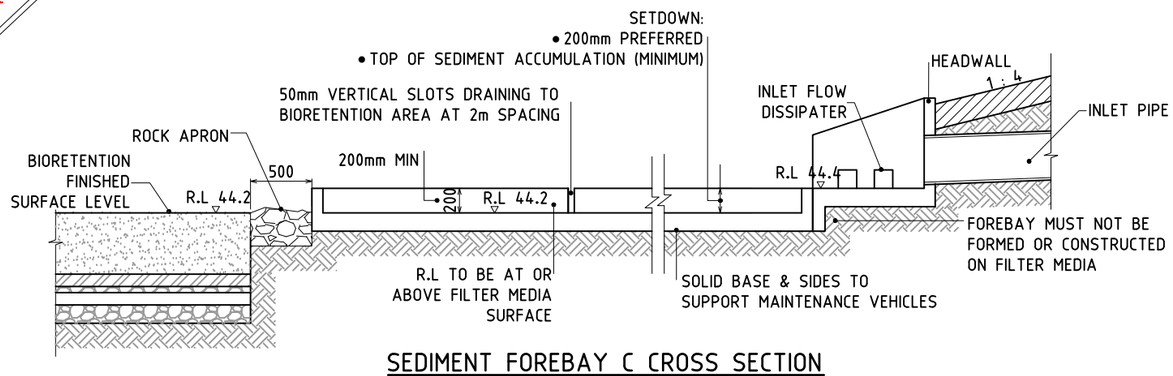
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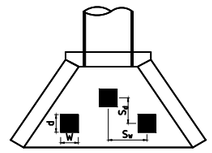
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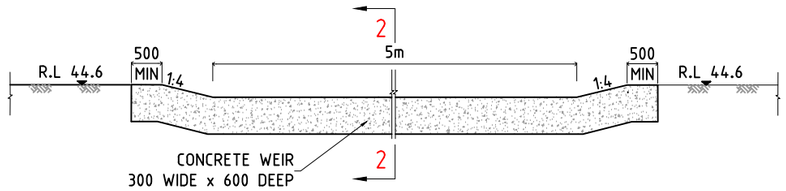


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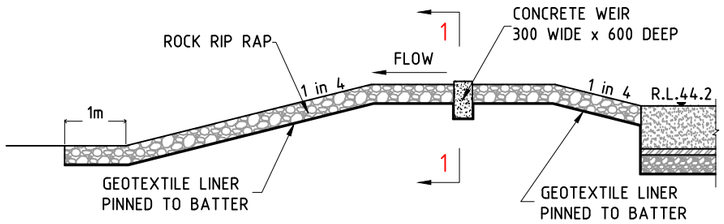
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INLET DETAILS



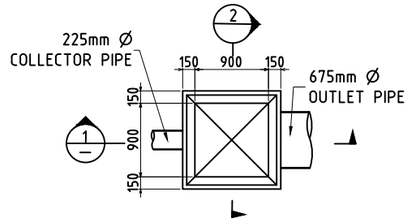
WEIR DETAIL SECTION 1-1

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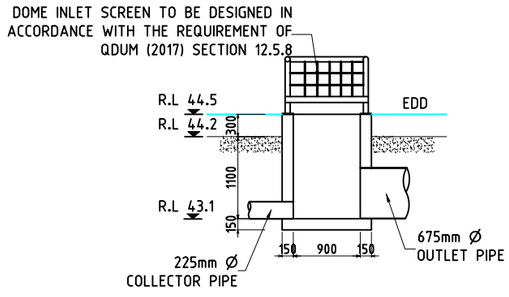
WEIR CROSS SECTION 2-2

SCALE: 1/100



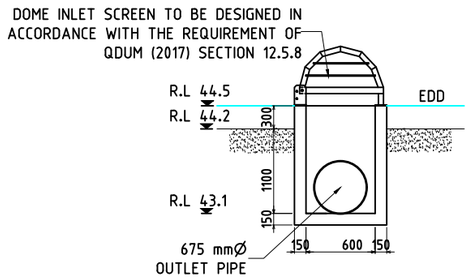
OVERFLOW STRUCTURE PLAN - PIT C

SCALE: 1/75



SECTION 1 - 1

SCALE: 1/75



SECTION 2 - 2

SCALE: 1/75

OUTLET DETAILS

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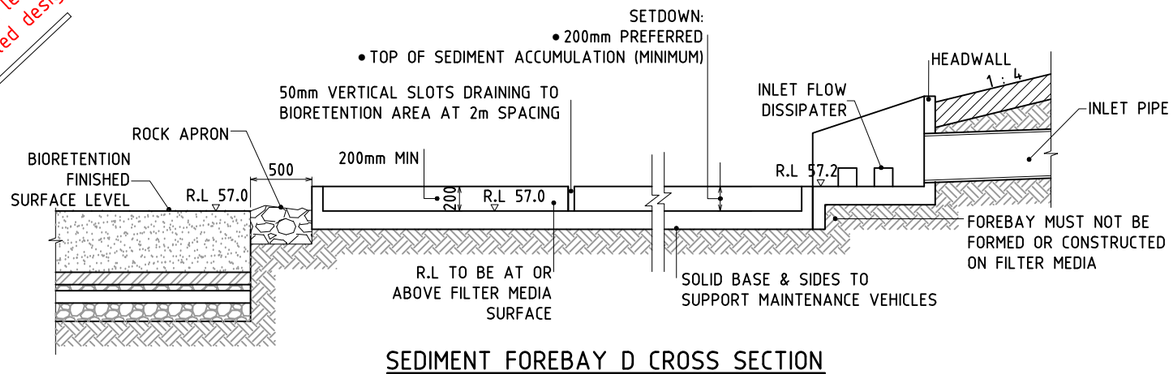
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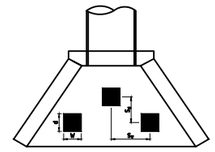
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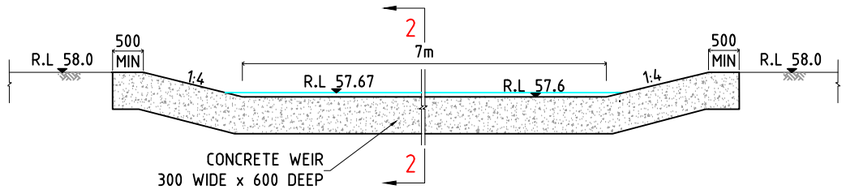


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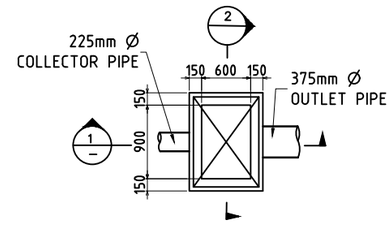
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INLET DETAILS



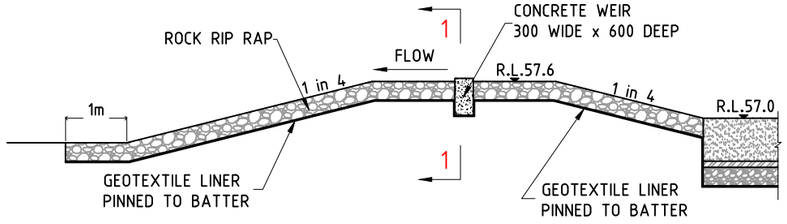
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SCALE: 1/100



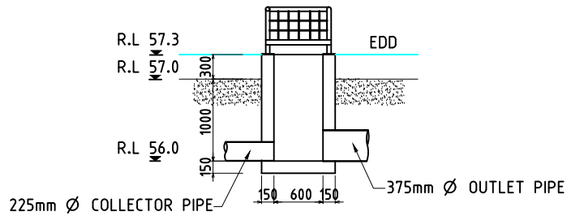
OVERFLOW STRUCTURE PLAN - PIT D

SCALE: 1/75



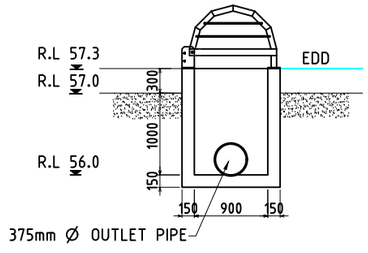
WEIR CROSS SECTION 2-2

SCALE: 1/100



SECTION 1 - 1

SCALE: 1/75



SECTION 2 - 2

SCALE: 1/75

OUTLET DETAILS

DATE
16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-323

REVISION No. :
C

PROJECT
PROPOSED SUBDIVISION

DRAWING TITLE
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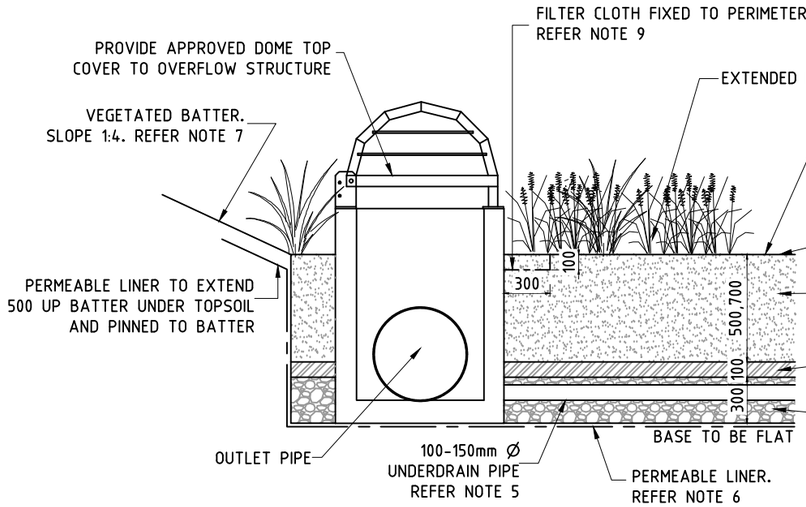
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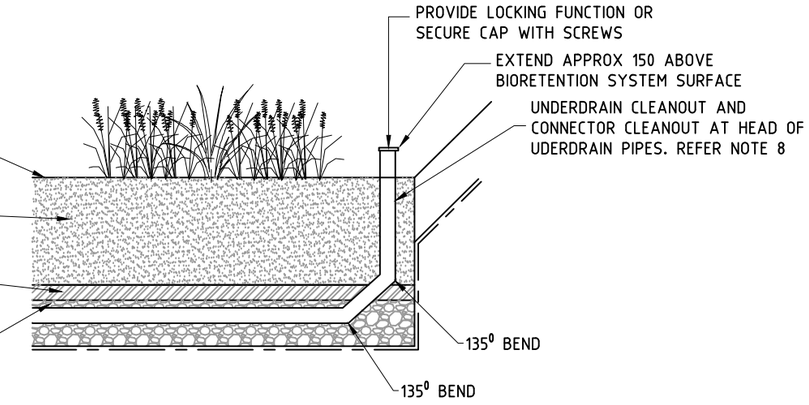
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BIORETENTION MEDIA (TYPE 3-CONVENTIONAL) CROSS SECTION

N.T.S



BIORETENTION BASIN SUBSOIL DRAIN CLEANOUT (TYPE 3-CONVENTIONAL)

N.T.S

NOTES (IN ACCORDANCE WITH IPWEAQ STANDARD DRAWINGS DS-070 AND DS-073):

1. BIORETENTION SYSTEM SURFACE. SURFACE LEVEL IS TOP OF FILTER MEDIA. SURFACE TO BE PLANTED AS PER PROJECT DRAWINGS AND THE 'BIORETENTION TECHNICAL DESIGN GUIDELINES' (WATER BY DESIGN).
2. FILTER MEDIA SPECIFICATION SHALL BE IN ACCORDANCE WITH THE 'ADOPTION GUIDELINES FOR STORMWATER BIOFILTRATION SYSTEMS (CRC FOR WATER SENSITIVE CITIES) AND THE BIORETENTION TECHNICAL DESIGN GUIDELINES (WATER BY DESIGN). BIORETENTION HYDRAULIC CONDUCTIVITY SHALL BE IN ACCORDANCE WITH PRACTICE NOTE 1: IN SITU MEASUREMENT OF HYDRAULIC CONDUCTIVITY' (FAWB). THE NUMBER OF SAMPLES TO BE TESTED SHALL BE IN ACCORDANCE WITH THE 'CONSTRUCTION AND ESTABLISHMENT GUIDELINES - SWALES, BIORETENTION SYSTEMS AND WETLANDS' (WATER BY DESIGN).
3. CONSTRUCTION TOLERANCES SHALL BE IN ACCORDANCE WITH THE 'CONSTRUCTION AND ESTABLISHMENT GUIDELINES -SWALES, BIORETENTION SYSTEMS AND WETLANDS' (WATER BY DESIGN)
4. TRANSITION LAYER AND DRAINAGE LAYER. DEPTHS AND SPECIFICATION TO BE IN ACCORDANCE WITH THE 'BIORETENTION TECHNICAL DESIGN GUIDELINES' (WATER BY DESIGN)
5. UNDERDRAIN. SLOTTED RIGID PIPE LAID FLAT. PIPE SHOULD NOT BE INSTALLED WITH A FILTER SOCK SURROUNDING PIPE. UNDERDRAIN PIPES SHALL BE SEALED INTO PITS USING GROUT OR OTHER APPROVED WATERTIGHT SEAL.
6. PERMEABLE LINER. NON-WOVEN GEOTEXTILE FILTER CLOTH TO BASE AND SIDES OF BIORETENTION SYSTEM. FILTER CLOTH NOT TO BE PLACED BETWEEN ANY FILTER LAYERS. PERMEABLE LINER AS PER 'BIORETENTION TECHNICAL DESIGN GUIDELINES' (WATER BY DESIGN)
7. VEGETATED BATTER. SLOPE AND PLANTING TO BE IN ACCORDANCE WITH 'BIORETENTION TECHNICAL DESIGN GUIDELINES' (WATER BY DESIGN).
8. INSPECTION/CLEANOUT POINT. VERTICAL SOLID PIPE SECTION ATTACHED TO THE END OF EACH UNDERDRAIN IN ACCORDANCE WITH THE 'BIORETENTION TECHNICAL DESIGN GUIDELINES'(WATER BY DESIGN)
9. FILTER CLOTH TO BE FIXED TO PERIMETER OF PIT TO AVOID RUNNELLING OF WATER BETWEEN PIT AND SOIL INTERFACE. BEGIN FILTER CLOTH 100 ABOVE SURFACE. EXTEND TO 100 BELOW SURFACE. CONTINUE 300 HORIZONTALLY INTO FILTER MEDIA.
10. FOR GENERAL DESIGN AND CONSTRUCTION NOTES REFER TO DS-078
11. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

DATE
16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-330

REVISION No. :
C

PROJECT
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BIORETENTION – FILTER MEDIUM DETAILS

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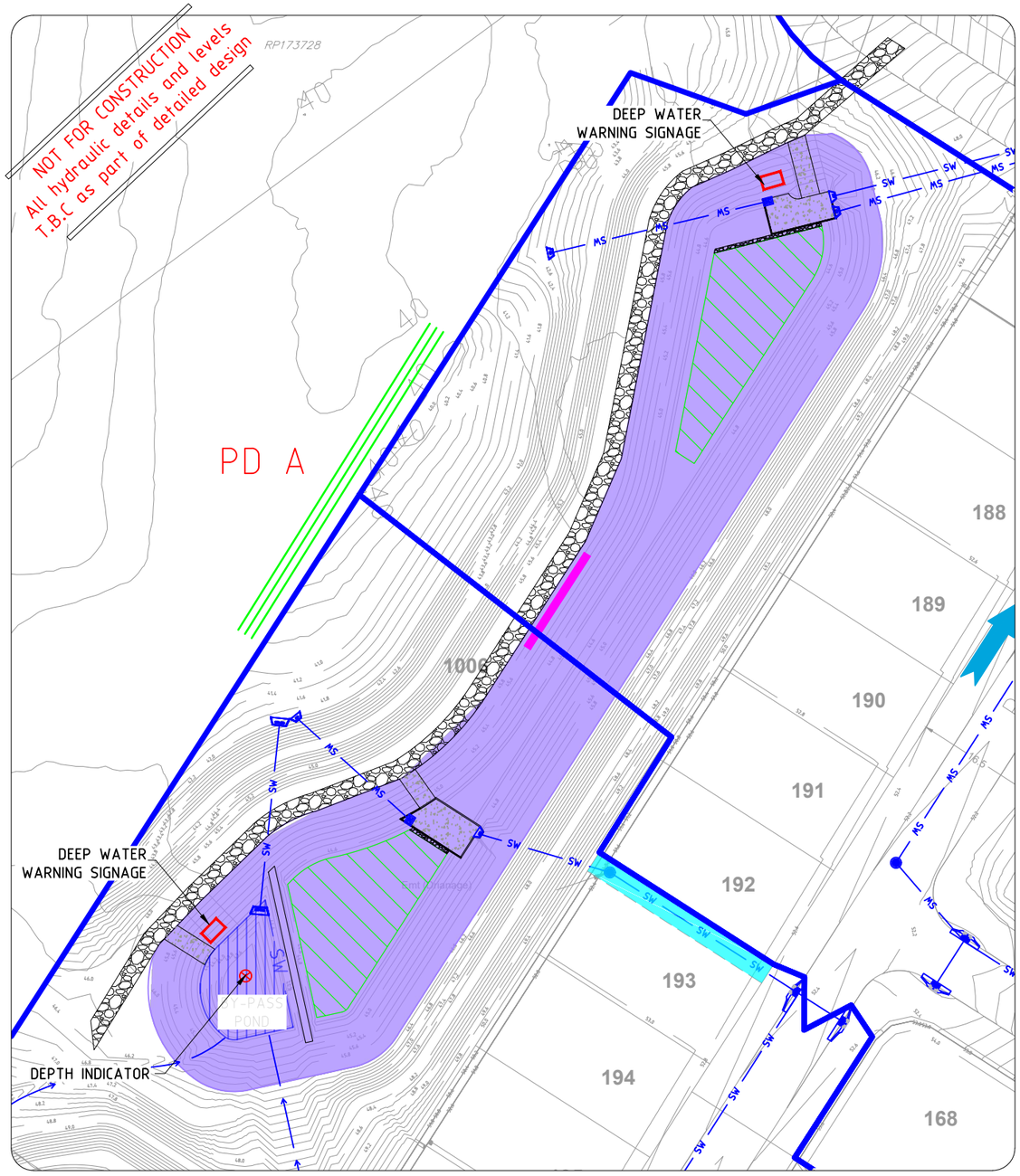
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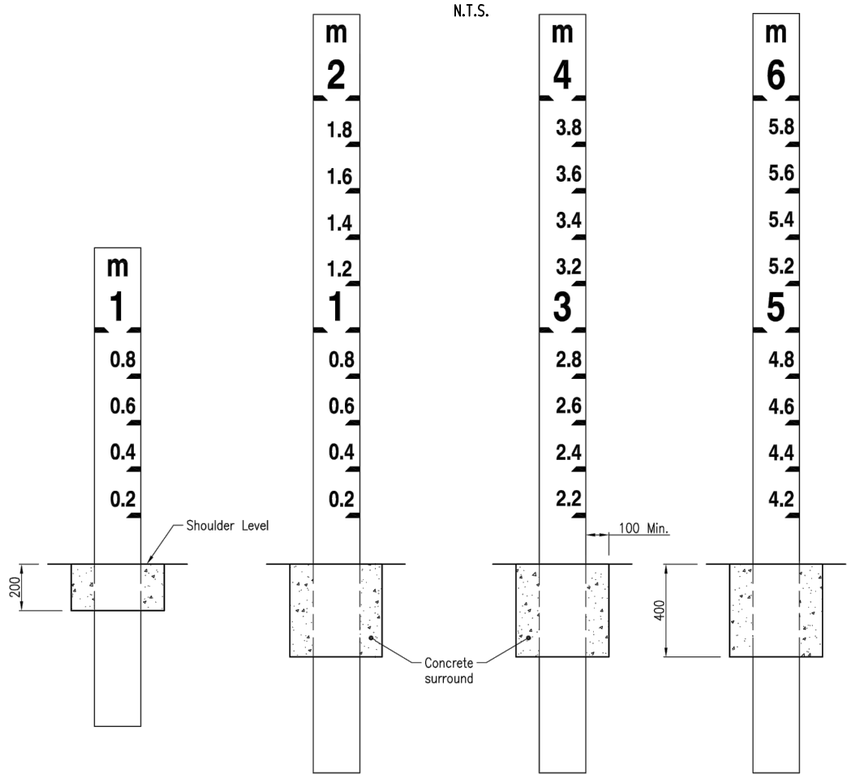
10 0 10 20 30 40

1 : 750 (FULL SIZE) (metres)



DEEP WATER WARNING SIGNAGE

N.T.S.



G9-22-1A G9-22-1 G9-22-2 G9-22-3

DEPTH INDICATORS

N.T.S.

NOTE:

FLOOD DEPTH INDICATORS:

1. DEPTH INDICATORS ARE TO HAVE A ZERO LEVEL RELATIVE TO THE LOWEST POINT IN THE BASIN FLOOR SO AS TO CLEARLY SHOW CURRENT WATER DEPTHS
2. G9-22-2 AND G9-22-3 INDICATORS TO BE ERECTED ON PROGRESSIVELY HIGHER GROUND WHERE FLOOD DEPTHS ARE IN EXCESS OF 1.5m OR 3.5m RESPECTIVELY.

CONCRETE SURROUND:

3. TO BE CLASS N20/20 AND PROVIDE 100mm MINIMUM COVER ALL ROUND.
4. DIMENSIONS ARE IN MILLIMETRES UNLESS SHOWN OTHERWISE.

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16.11.20

PROJECT No. :
BC-16090

DRAWING No. :
DWG-340

REVISION No. :
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PROJECT
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DRAWING TITLE
BASIN A - DEPTH INDICATOR AND DEEP WARNING SIGNAGE

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Appendix C

Rational Method Model Validation

The Rational Method has been relied upon to validate the flows reported within *xpstorm* for both the pre and post development cases. In accordance with QUDM 2013 Section 4, the Rational Method provides a simple means of estimating peak discharge and is therefore considered suitable for validation purposes. Equation 4.2 from QUDM has been relied upon.

$$Q_y = (C_y \,.\, {}^{t}I_y \,.\, A)/360$$

where:

Q_y	=	peak flow rate (m^3/s) for annual exceedance probability (AEP) of 1 in '*y*' years
C_y	=	coefficient of discharge for AEP of 1 in '*y*' years
A	=	area of catchment (ha)
${}^{t}I_i$	=	average rainfall intensity (mm/h) for a design duration of '*t*' hours and an AEP of 1 in '*y*' years
t	=	the nominal design storm duration as defined by the time of concentration (t)

Coefficient of Discharge

Based on the infiltration characteristics of the internal and external catchments, the following discharge coefficients have been calculated using the method presented within Book 8 of Australian Rainfall and Runoff (1998) and based on Equation 4.4 from QUDM (2013).

$$C_y = F_y \,.\, C_{10}$$

where:

C_y	=	coefficient of discharge for AEP of 1 in '*y*' years
F_y	=	frequency factor for AEP of 1 in '*y*' years
C_{10}	=	10 year discharge coefficient value for Tables 4.5.3 and 4.5.4 of QUDM

Based on Table SC6.9.2-7, Runoff Coefficient vs Development Category of the City Of Gold Coast Land Development Guidelines, C_{10} values have been determined for each catchment and the coefficient of discharge values for the nominated AEP's for the existing and developed catchment condition are shown in Tables C.1 and C.2, respectively.

Table C.1 Coefficient of Discharge – Existing Case

Catchment & Point of Discharge ID			C_2	C_5	C_{10}	C_{20}	C_{50}	C_{100}
External	PD-A	Ext-1	0.68	0.76	0.8	0.84	0.92	0.96
Internal		A	0.68	0.76	0.8	0.84	0.92	0.96
	PD-B		0.68	0.76	0.8	0.84	0.92	0.96
	PD-C		0.68	0.76	0.8	0.84	0.92	0.96
	PD-D		0.68	0.76	0.8	0.84	0.92	0.96

Table C.2 Coefficient of Discharge – Developed Case

Catchment & Point of Discharge ID			C_2	C_5	C_{10}	C_{20}	C_{50}	C_{100}
External	PD-A	Ext-1	0.68	0.76	0.8	0.84	0.92	0.96
Internal	PD-A	A1	0.68	0.76	0.80	0.84	0.92	0.96
	PD-A	A2a	0.72	0.81	0.85	0.89	0.98	1.00
	PD-A	A2b	0.72	0.81	0.85	0.89	0.98	1.00
	PD-B		0.72	0.81	0.85	0.89	0.98	1.00
	PD-C		0.72	0.81	0.85	0.89	0.98	1.00
	PD-D		0.72	0.81	0.85	0.89	0.98	1.00

Time of Concentration

The time of concentration (t_c) for each catchment was calculated in accordance with Section 4.6 of QUDM. A standard inlet time of concentration of 5 min has been adopted within developed catchments from Table 4.6.2 of QUDM (2013).

The time of concentration (t_c) for the initial flow length within the undeveloped catchments was estimated using Friend's Equation for overland flow (QUDM Section 4.6.6).

$$t_c = (107nL^{0.333})/S^{0.2}$$

where:

t_c	=	Time of concentration (min)
n	=	Horton's roughness value
L	=	Overland sheetflow path length (m)
S	=	Slope (%)

For channel flows, times where estimated using approximate stream velocities as provided within QUDM Table 4.6.6 and based an average catchment surface slope.

Table C.3 and C.4 below present the parameters relied upon to calculate the time of concentration (t_c) to the discharge point of each catchment for the existing and developed case, respectively.

Table C.3 t_c Parameters Remaining Flow Length – Existing Case

Parameter	External	Internal			
	Ext-1	A	B	C	D
Standard Inlet Time					
t_c (min)	-	-	-	-	-
Sheet Flow					
Flow Length (m)	50	50	50	50	50
Horton's Roughness Value	0.04	0.04	0.04	0.04	0.04
Slope (%)	25	8	11	14	16
t_c (min)	8.3	10.4	9.7	9.3	9.0
Channel Flow					
Flow Length (m)	150	341	217	111	78
Velocity (m/s)	3	1.5	1.5	1.5	3
Slope (%)	25	8	11	14	16
t_c (min)	0.83	3.79	2.41	1.23	0.43
TOTAL t_c (min)	**9.11**	**14.18**	**12.16**	**10.52**	**9.48**

Table C.4 Time of Concentration - Developed Case

Parameter	External	Internal					
	Ext-1	A1	A2a	A2b	B	C	D
Standard Inlet Time							
t_c (min)	-	-	5	5	5	5	5
Sheet Flow							
Flow Length (m)	50	50	-	-	-	-	-
Horton's Roughness Value	0.04	0.04	-	-	-	-	-
Slope (%)	25	20	-	-	-	-	-
t_c (min)	8.3	8.6	-	-	-	-	-
Channel Flow							
Flow Length (m)	150	65					
Velocity (m/s)	3	3					
Slope (%)	25	30					
t_c (min)	0.83	0.36					
Pipe Flow							
Flow Length (m)	-	-	280	320	270	250	140
Velocity (m/s)	-	-	1	1	1	1	1
Slope (%)	-	-	1	1	1	1	1
t_c (min)	-	-	4.67	5.33	4.50	4.17	2.33
TOTAL t_c (min)	**9.11**	**9.01**	**9.67**	**10.33**	**9.50**	**9.17**	**7.33**

Rainfall Intensity

Based on the calculated t_c for each catchment, Intensity-Frequency-Duration data (IFD) has been obtained from the Bureau of Meteorology for nominated AEP's and the existing and developed cases are presented in Tables C.5 and C.6, respectively.

Table C.5 Rainfall Intensity (mm/h) – Existing Case

Catchment & Point of Discharge ID			I_2	I_5	I_{10}	I_{20}	I_{50}	I_{100}
External	PD-A	Ext-1	119.45	148.87	171.12	196.19	228.32	252.10
Internal		A	100.20	124.61	143.10	162.92	188.74	207.40
	PD-B		107.35	133.50	153.20	175.04	202.88	223.15
	PD-C		113.16	140.71	161.40	184.88	214.36	235.94
	PD-D		117.60	14643	168.16	192.78	224.03	247.07

Table C.6 Rainfall Intensity (mm/h)– Developed Case

Catchment & Point of Discharge ID			I_2	I_5	I_{10}	I_{20}	I_{50}	I_{100}
External	PD-A	Ext-1	119.45	148.87	171.12	196.19	228.32	252.10
Internal		A1	95.7	120.0	149.5	171.9	197.1	229.5
		A2a	93.0	116.7	145.2	166.6	191.0	221.8
		A2b	90.8	113.8	141.5	162.4	186.0	215.7
	PD-B		93.7	117.5	146.3	168.0	192.6	223.8
	PD-C		95.1	119.2	148.5	170.6	195.6	227.6
	PD-D		102.5	128.4	160.6	185.4	212.6	249.0

Peak Discharge

Based on the above parameters, estimates of the expected peak discharge generated at the site's existing discharge locations for nominated AEP's have been calculated, and are presented in Tables C.7 and C.8 below.

Table C.7 Peak Discharge (m^3/s) – Existing Case

Catchment & Point of Discharge ID			0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
External	PD-A	Ext-1	0.51	0.71	0.86	1.03	1.32	1.52
Internal	PD-A	A	1.63	2.26	2.74	3.27	4.15	4.76
	PD-B		0.89	1.24	1.50	1.80	2.28	2.62
	PD-C		0.62	0.86	1.04	1.25	1.58	1.82
	PD-D		0.43	0.60	0.72	0.87	1.11	1.28

Table C.8 Peak Discharge (m^3/s) – Developed Case

Catchment & Point of Discharge ID			0.393 (2 yr ARI)	0.181 (5 yr ARI)	0.095 (10 yr ARI)	0.049 (20 yr ARI)	0.02 (50 yr ARI)	0.01 (100 yr ARI)
External	PD-A	Ext-1	0.51	0.71	0.86	1.03	1.32	1.52
Internal	PD-A	A1	0.39	0.54	0.65	0.79	1.00	1.16
	PD-A	A2a	0.88	1.23	1.49	1.79	2.28	2.57
	PD-A	A2b	0.69	0.96	1.15	1.39	1.76	1.99
	PD-B		1.23	1.71	2.06	2.48	3.16	3.56
	PD-C		0.59	0.83	1.00	1.20	1.53	1.73
	PD-D		0.55	0.77	0.94	1.13	1.45	1.64