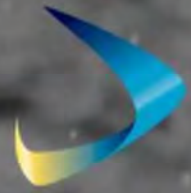


LOGAN CITY COUNCIL

APPROVED DOCUMENT

This is an approved document for Development Application

MCUI/36/2018



Premise

Approved for S/W Quality Only

WATER

INGENIA COMMUNITIES

PROPOSED DEVELOPMENT AT LOT 21 ON RP804908,
AND LOT 31 ON RP848575
CHAMBERS FLAT QLD

STORMWATER MANAGEMENT REPORT

Report No: ING-0001/1802326

Rev: 4

Date: 17th January 2019

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APPENDICES

Appendix A	Proposed Development Plan (By Others)
Appendix B	Pre and Post Development Catchment Plan
Appendix C	Rational Method Results
Appendix D	Civil Drainage Plans

DOCUMENT AUTHORISATION					
Job Code	Revision	Rev. Date	Report Details		
INGCCH	-	18/05/2015	Final		
	A	25/05/2015	Final - Modified property description		
	B	15/06/2016	Final - Revised modelling		
	C	16/08/2016	Final - Revised modelling to respond to RFI.		
	1	28/11/2017	Final - Revised Music Modelling		
ING-0001	2	14/06/2018	Additional Lot to be Developed		
	3	04/07/2018	Updated Plans		
	4	17/01/2019	Amended Report – RFI Response		
Prepared By		Reviewed By		Authorised By	
M. Roushani M. Nguyen		D. Niven		D. Niven	

1 INTRODUCTION

Premise Australia Pty Ltd (formerly 'O2 Environmental Pty Ltd') has been commissioned by Ingenia Communities to update the Stormwater Management Plan (SMP) in support of an amendment and an addition of lot to the proposed site layout.

The site contains an existing amended development conditions by Logan City Council (Application Number: MCUI/33/2015/B, Document Number: 11361170) for a Material Change of Use for 2-40 Koplick Road, Chambers Flat, or formally known as Lot 21 on RP804908. This report includes the additional lot as a part of the proposed development, 659-667 Chambers Flat Road, Chambers Flat QLD, formally known as Lot 31 on RP848575.

Furthermore, this report includes modelling and reporting to support 3 hectares of proposed post-development flows from the proposed development south of this proposed site (Application Number: COM/12/2017). This is in accordance to a private agreement by both parties, and is understood to be in agreement with Council.

It should be noted that revision 1 of this SMP superceded the previously approved SMP for the site, *Stormwater Management Plan – Chambers Pines Lifestyle Resort*, by O2 Environmental Pty Ltd (now 'Premise').

This report also incorporates amendments in accordance with Logan City Council's Information Request provided on the 5th October 2018 (Application Number: MCUI/36/2018, Document Number: 12317534).

The regional location of the Project area is presented in **Figure 1-1**.

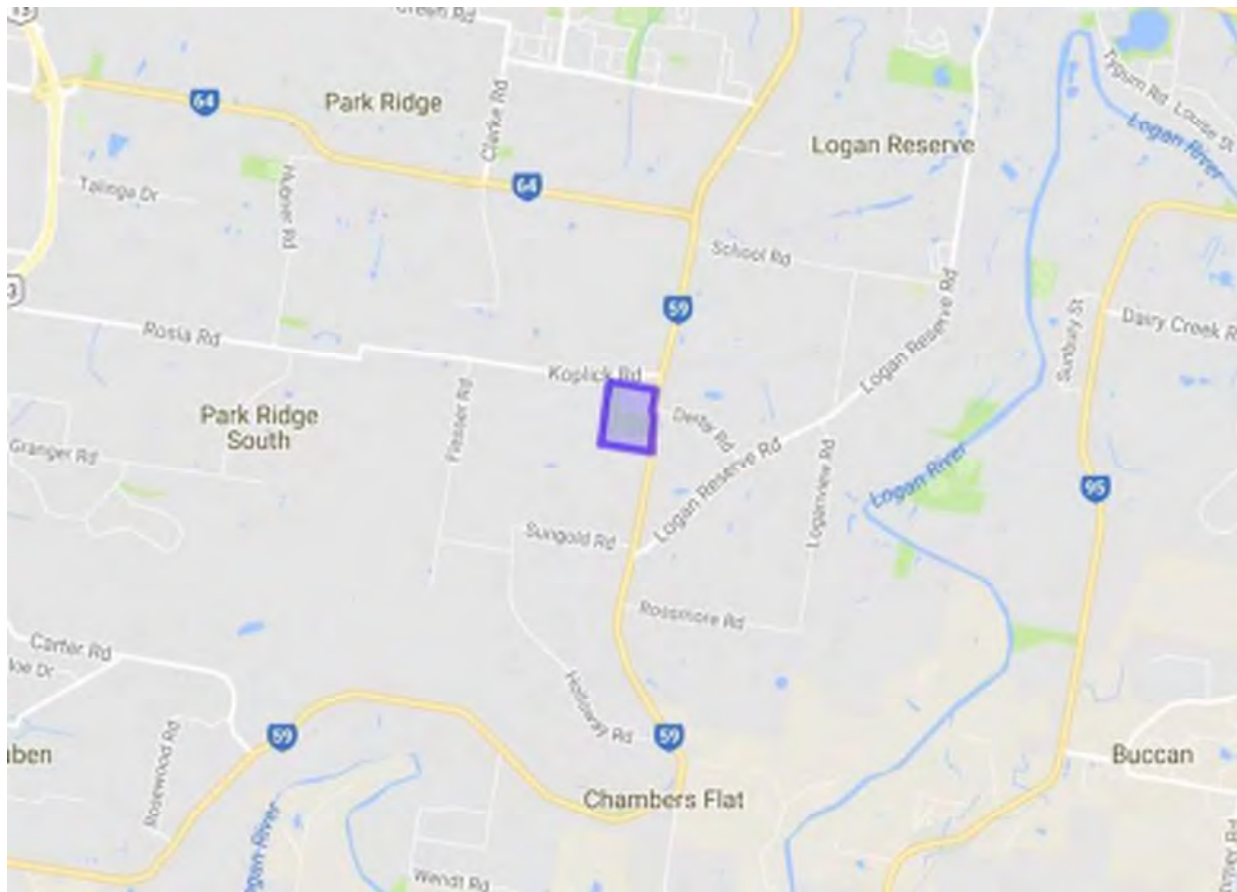


Figure 1-1 – Site location of Proposed Development Site (Source: Nearmap)

1.1 Objectives

This report takes into consideration and supports the existing approved Operational Works Application, and hence provides amendments to regions that are not under construction, or are deemed to be modifiable. In particular, this report provides a revised MUSIC modelling solution, in accordance with the latest Healthy Land and Water's (HLW) MUSIC Modelling guidelines. The former MUSIC modelling guidelines were set by the Water by Design Initiative, which had provided guidelines for MUSIC Version 3 onwards. However, the latest version of MUSIC, Version 6, has a particular set of updated and recommended parameters to be utilised when modelling bio-retention filter area. In addition to these modelling updates, this SBSMP has incorporated the minor layout changes since the previously approved SBSMP, to ensure the MUSIC model contains the latest configuration, and ensures the previous State Planning Policy's Water Quality Objectives (WQO's) are still adhered to.

The treatment measures proposed as part of this document are based on the requirements of the Logan City Council, the Queensland Urban Drainage Manual (QUDM) and the State Planning Policy at time of original lodgement. This objective will be achieved by:

- Ensuring the pre-development flow rates at the Lawful Points of Discharge (LPD) from the site are maintained for all standard AEP events; and
- Demonstrating that the water quality objectives for the site are met by providing treatment measures to ensure the 80%, 60%, 45% load based reduction targets for Total Suspended Solids (TSS), Total Nitrogen (TN) and Total Phosphorous (TP) are achieved.

2 SITE CHARACTERISTICS

2.1 Land Use

The pre-developed site and its surroundings are dominated by a rural residential land use, with large parcels of heavily vegetated land remaining in all directions around the site. The site of the proposed development was originally a golf course at the initial stages of this development application (now under construction in accordance with the Operational Works Application), and forms part of an existing caravan/relocatable home park. Chambers Flat Road forms the eastern site boundary, and Koplick Road on the northern site boundary.

Refer to **Appendix A** for Proposed Site Layout Plan and stormwater management structure locations.

2.2 Topography and Drainage

The site generally falls to the north, with the majority of runoff being conveyed through an existing concrete lined drain through the existing caravan/relocatable home park. On exiting the drain at the northern boundary of the existing caravan/relocatable home park, runoff is conveyed a short distance before passing through culverts under Chambers Flat Road.

On leaving the site, runoff is conveyed through a number of downstream properties, eventually making its way to the Logan River. Refer to **Figure 2-1** for the indicative flow regime through the site.

2.2.1 Lawful Point of Discharge

For the purposes of this assessment, the existing caravan/relocatable home park has been taken into consideration. Therefore, the Lawful Point of Discharges (LPDs) for has been identified as the northern boundary of the existing caravan/relocatable home park where runoff enters the drainage reserve. The LPDs are illustrated in **Figure 2-1**. In addition to these LPD locations it is also intended to ensure that post-development discharge into the existing channel through the constructed Chamber Pines Lifestyle Resort is improved.



Figure 2-1 – Pre-Development Flow Characteristics & Lawful Points of Discharge

3 DATA

In the preparation of this report, information about the site was gathered from the following sources:

- Site Information by Logan City Council;
- Detailed Ground Survey Data by RPS Group;
- Proposed Site Layout by RPS Group;
- Civil Engineering Plans by Premise;
- Rainfall and Meteorological Data provided by Bureau of Meteorology (Rainfall data for Greenbank Thompson Road Station - #40659); and
- Aerial Imagery by Nearmap (Accessed Nov 2017).

4 STORMWATER QUALITY MANAGEMENT

4.1 Construction Phase Stormwater Quality Management

Management of stormwater during the construction phase is to be undertaken in accordance with the site-specific Erosion and Sediment Control Plan and other management plans provided by the supervising engineers or civil contractors.

Construction and establishment of proposed bioretention basins should be undertaken in accordance with Council guidelines and conditions as well as the approved engineering and landscape plans. The Water by Design 'Construction and Establishment Guidelines – Swales, Bioretention Systems and Wetlands, 2009' is a useful guide to utilise during the construction and establishment phase of the bioretention systems.

A major risk for establishing effectively functioning bioretention systems is the inflow of sediment-laden runoff during the construction and building phases. Captured sediment has the potential to clog the filter media, thus minimizing the overall effectiveness of the constructed system. Managing sediment during the construction and building phases is critical for the establishment of the bioretention system and in reducing the risk and costs associated with reconstructing the system. The typical phases associated in the creation of a development site are detailed below in **Figure 4-1**

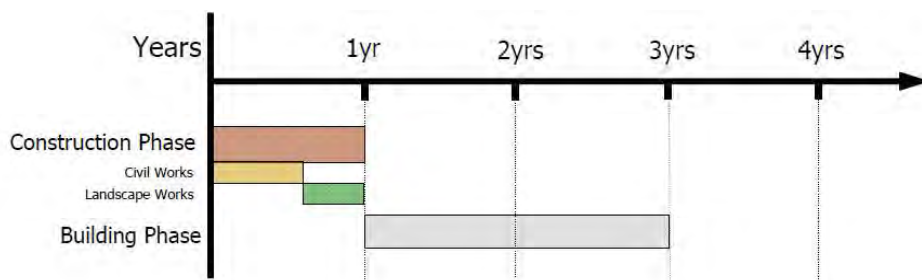


Figure 4-1 - Typical phases in the creation of a development site (Source: Water by Design, 2009)

To overcome challenges associated with delivering bioretention systems, one of the following four options for construction and establishment are recommended:

- 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.

Further information on each option is provided in the Water by design 'Construction and Establishment Guidelines – Swales, Bioretention Systems and Wetlands, 2009'.

For this development Option 1 is recommended. The filter media can be installed upon completion of civil works, including under-drainage and media, for the bioretention systems. Install protective layers on the surface of the filter media, allowing it to operate as a shallow sediment basin during the building phase. Surface protection options are:

- Filter cloth, topsoil and turf, and
- Coarse sand, topsoil and turf.

When 80-90% of building in the catchment is complete, remove the protective layers and establish vegetation in the bioretention system.

4.2 Stormwater Management Design Objectives

The following Water Quality Objectives (WQO's) have been adopted from the DERM 'Urban Stormwater Planning Guidelines – 2010', of which this development application has been assessed under, and is detailed in **Table 4-1** below.

Table 4-1 –Stormwater Management Design Objectives

Pollutant	Stormwater Management Design Objective
Suspended Solids	80% (Removal)
Total Phosphorus	60% (Removal)
Total Nitrogen	45% (Removal)
Gross Pollutants	90% (Removal)

4.3 Stormwater Quality Modelling

Stormwater Pollutant modelling for the development has been generated using the modelling program 'Model for Urban Stormwater Improvement Conceptualisation' (MUSIC) Version 6. A combination of Lumped and Split Catchment parameters and modelling have been adopted for the MUSIC modelling in accordance with Healthy Waterways MUSIC Modelling Guidelines Version 1.0, 2010 and Healthy Land and Water Stormwater Compliance: MUSIC Modelling parameters to evaluate expected stormwater quality outcomes for the proposed development.

The lumped approach provides general parameters for particular landuse types, whereas the split catchment approach provides parameters based on the surface type. The split and lumped catchment parameters were adopted from Tables 3.8 and 3.9 respectively from the *MUSIC Modelling Guidelines Version 1.0*.

Stormwater runoff is simulated using historical rainfall data. Data from Greenbank Thompson Road, station number 40659, has been used from 1980 - 1989 to represent the climatic period upon which the MUSIC model is based. Mean potential evapotranspiration from the Climate Atlas of Australia was used to account for rainfall loss. These designations are in line with requirements specified in the MUSIC Modelling Guidelines.

Catchment delineation and model arrangement was based on the proposed development layout. Refer to **Appendix B** for catchment plans outlining this information.

4.4 Surface Split Imperviousness Fraction

The surface split percentage used within the MUSIC modelling has been estimated from the lot layout plans provided by RPS Group. Refer to **Table 4-2** for fraction imperviousness calculations for each node.

Table 4-2 – MUSIC Model Catchment Parameters

Catchment Node	Modelled Parameter		
	Area (ha)	Land Use	Fraction Imperviousness (%)
WQ1	2.91	Urban Residential (Lumped)	70
WQ2	1.90	Urban Residential (Lumped)	85
WQ3	0.71	Urban Residential (Lumped)	70
WQ4	4.22	Urban Residential (Lumped)	85
WQ5	1.70	Urban Residential (Lumped)	65
WQ6+3ha	7.22	Urban Residential (Lumped)	85
Carpark	0.07	Road (Split)	90
Total	18.73		

The MUSIC model treatment train has been developed by directing all 3-month ARI stormwater flows for each proposed post sub-catchment to their respective stormwater treatment system via their respective internal stormwater drainage system. Refer to **Figure 4-2** for the MUSIC model treatment train.

A fraction impervious of 85% has been adopted for each proposed sub-catchment, as they are deemed Urban Residential landuse areas. A fraction impervious of 90% has been adopted for the amended carpark areas, given the overall sites density.

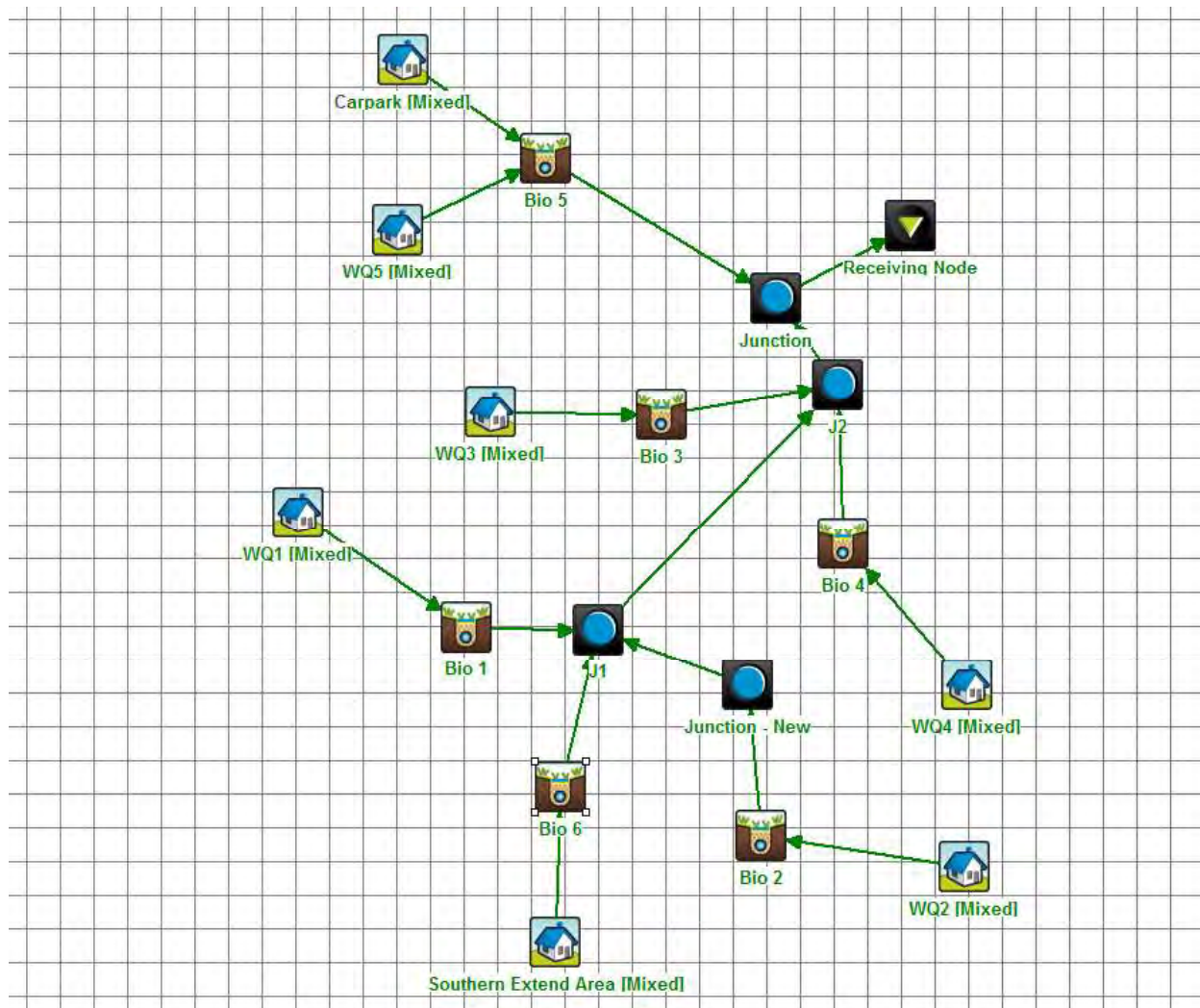


Figure 4-2 – MUSIC Model Schematic (Source: eWater MUSIC)

The following parameters have been adopted within the MUSIC model:

Table 4-3 – Rainfall-Runoff Parameters adopted

Catchment Characteristic	Urban Residential
Impervious Area Properties	
Impervious Area Rainfall Threshold (mm/day)	1
Pervious Area Properties	
Pervious Area Soil Storage Capacities	500
Pervious Area Soil Initial Storage (% of Capacity)	10
Field Capacity (mm)	200
Pervious Area Infiltration Capacity coefficient – a	211
Pervious Area Infiltration Capacity exponent - b	0.50
Groundwater Properties	
Groundwater Initial Depth	50
Groundwater Daily Recharge Rate (%)	28
Groundwater Daily Base Flow Rate (%)	27
Groundwater Daily Deep Seepage Rate (%)	0

Adopted from Healthy Waterways MUSIC Modelling Guidelines Version 1.0, 2010.

Rainfall Data is taken from BOM's Greenbank Thomson Road Station - #40659 with a 6-minute time step;

- A seepage loss rate of 0mm/hr has been assumed in modelling;
- All other parameters used within the modelling were based on Healthy Waterways MUSIC Modelling Guidelines Version 1.0, 2010 and Online Publications by Healthy Land and Water which provide MUSIC Parameters to be adopted for Version 6.

4.5 Bio-Retention System Parameters

The parameters utilised to analyse the bioretention systems are presented in **Table 4-4**.

Table 4-4 – Bio-retention System MUSIC Modelling Parameters

Bio-Retention Parameter	Bio 1	Bio 2	Bio 3	Bio 4	Bio 5	Bio 6
Surface Area (m2)	188	190	148	252	158	986
Extended detention depth (m)	0.15	0.15	0.15	0.15	0.15	0.3
Filter area (m2)	149	151	125	199	158	900
Unlined filter media perimeter (m)	0.01	0.01	0.01	0.01	0.01	0.01
Saturated hydraulic conductivity (mm/hr.)	180	180	180	180	180	180
Filter Depth (m)	0.5	0.5	0.5	0.5	0.5	0.5
TN content of filter media (%)	400 ⁽¹⁾	400 ⁽¹⁾	400 ⁽¹⁾	400 ⁽¹⁾	400 ⁽¹⁾	400 ⁽¹⁾
Orthophosphate content of filter media (mg/kg)	30 ⁽¹⁾	30 ⁽¹⁾	30 ⁽¹⁾	30 ⁽¹⁾	30 ⁽¹⁾	30 ⁽¹⁾
Is the base lined?	No	No	No	No	No	No
Vegetated with effective nutrient removal plants	Yes	Yes	Yes	Yes	Yes	Yes
Overflow weir width (m)	17	17	14	23	54	25
Exfiltration rate (mm/hr.)	0	0	0	0	0	0
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	N/A	N/A
If exfiltration rate has been used is the exfiltration rate justified?	N/A	N/A	N/A	N/A	N/A	N/A
Underdrain present?	Yes	Yes	Yes	Yes	Yes	Yes
Submerged zone with carbon present?	No	No	No	No	No	No
Confirmation that K and C* remain default	Yes	Yes	Yes	Yes	Yes	Yes

NOTE: (1) Recommended values from the HLW Music Modelling parameters for MUSIC Version 6 and in accordance with suppliers filter media.

4.6 The Treatment Train Effectiveness

Table 4-5 outlines the effectiveness of the MUSIC Model treatment train in achieving the set Water Quality Objectives (WQO's) for pollutant reduction for the overall proposed development.

Table 4-5 – Treatment Train Effectiveness at Receiving Node

Parameter	Reduction (%)	Objective (%)	Objective Achieved
Total Suspended Solids (kg/yr)	80	80	Yes
Total Phosphorus (kg/yr)	71	60	Yes
Total Nitrogen (kg/yr)	45	45	Yes
Gross Pollutants (kg/yr)	100	90	Yes

5 STORMWATER QUANTITY MANAGEMENT

5.1 Introduction

Hydrologic and hydraulic modelling has been undertaken to establish the flow regime and peak discharges at the outlet of the site to the north. The modelling has been undertaken in XPSWMM for both the pre-development and post-development scenarios.

5.2 Catchments

Site runoff is conveyed to the Lawful Points of Discharge (LPD's) located in the northern portions of the site. The site has a total area of approximately 19.88ha, while the external catchments cover an area of approximately 19.54ha.

The pre- and post-development catchment plans have been provided within **Appendix B**.

It is noted that in the post-development scenario, the upstream catchments EXT1 & EXT2 shall enter an open drain along the western property boundary, and flows shall be directed north to be discharged into Koplick Road. This ensures the flows do combine with the site's stormwater discharge and ensures the ultimate discharge point of these external flows are still retained. The details of these external catchments are demonstrated in the tables below.

External catchments fraction imperviousness was estimated based off aerial photography analysis. The pre and post fraction imperviousness is deemed not to change for the external catchments, with the exception of the southern neighbouring property, of which it has been agreed that the subject site will accept post-developed stormwater discharge for stormwater mitigation. It is assumed all other future development on neighbouring properties will provide their own stormwater management system.

The catchment parameters which have been adopted and do not change for the quantity assessment are provided below in **Table 5-1**, and the catchment parameters which have been adopted but are modified as a result of the proposed development have been provided in **Table 5-2**.

Table 5-1 – Adopted Catchment Parameters

Pre-Development Scenario			Post-Development Scenario		
Catchment	Area (ha)	Fraction Impervious (%)	Catchment	Area (ha)	Fraction Impervious (%)
Ext1	0.89	0	Same as Pre-Development Scenario		
Ext2	1.69	0			
Ext3	0.85	0			
Ext4	11.07	0			
Ext5	0.69	1			
Ext 7*	2.44				
Ext 8*	7.27				
C01	1.45	85			
C04	1.24	85			

*Note: Not used in the XPSWMM model as this is downstream of model, only for informational purpose

Table 5-2 – Adopted Catchment Parameters - Changes Due to Development

Pre-Development Scenario			Post-Development Scenario		
Catchment	Area (ha)	Fraction Impervious (%)	Catchment	Area (ha)	Fraction Impervious (%)
C02	1.53	85	P02	1.50	85
C03	1.15	85	P03	1.00	85
C05	0.33	13	P05	1.70	65
C06	1.37	0	P06	0.91	73
C07	0.94	0	P07	2.00	57
C08	1.24	0	P08	0.71	70
C09	1.51	0	P09	1.90	85
C10	5.62	0	P10	4.22	85
C11	1.33	7	P11	3.25	85
C12	2.17	8	P12	3.00	85
Ext6	4.36	7	Ext 6B*	0.73	0

*Note: In Post-Development scenario, External catchment Ext 6 was split into Ext 6B and P12 in which 3ha developed area will discharge to the project site.

5.3 Rainfall Intensities

Intensity Frequency Duration (IFD) data for the site has been derived in accordance with Australian Rainfall and Runoff 1987 (ARR87) though the use of the AUS-IFD software. The rainfall intensities generated and adopted for the quantity assessment are shown below in **Table 5-3**.

Table 5-3 - IFD Data Applied to Hydrologic Assessment

Duration	63% AEP	39% AEP	18% AEP	10% AEP	5% AEP	2% AEP	1% AEP
(mins)	(mm/hr)	(mm/hr)	(mm/hr)	(mm/hr)	(mm/hr)	(mm/hr)	(mm/hr)
10	90	114	142	158	180	209	232
15	75	96	119	133	152	176	195
20	65	83	104	116	133	155	171
25	58	75	93	104	119	139	154
30	53	68	85	95	109	127	141
45	43	55	69	77	88	103	114
60	36	47	58	66	75	88	98
90	28	35	45	50	58	68	75
120	23	29	37	41	48	56	62

5.4 Hydrologic Modelling

The XPSWMM runoff-routing model has been used to estimate design flood discharges within the site. The model represents the sub-catchments as a network of nodes linked to a 1D drainage network. Each node is defined by its pervious (undeveloped) and impervious (developed) area, fraction impervious and average catchment slope. A fraction impervious of 0% has been applied to the undeveloped portion and 100% to the developed portion. The net rainfall is routed through the network after appropriate losses (initial and continuing) and roughness factors are applied, resulting in a surface runoff hydrograph for each sub-catchment. Hydrologic modelling has been undertaken using the Laurenson Runoff-Routing Method.

5.4.1 Hydrologic Roughness

Hydrologic roughness in the form of Manning's n values were applied to represent the undeveloped and developed portions of the catchment. The following values were adopted:

- Developed catchment $n = 0.014$; and
- Undeveloped catchment $n = 0.045$.

5.4.2 Rainfall Losses

The following Initial Loss (IL) and Continuing Losses (CL) were applied to the XPSWMM modelling. The following loss rates were adopted:

- Developed catchment IL = 1mm CL = 0mm/hr; and
- Undeveloped catchment IL = 12mm CL = 2mm/hr.

5.4.3 Rational Method Comparison

The peak discharges provided by the hydrologic model for the 1% AEP event have been compared against those obtained from the Rational Method calculations to ensure the model is providing reasonable results. The results of this comparison are provided in **Table 5-4** and demonstrate the hydrologic model is providing results within a reasonable range of the Rational Method calculations and is therefore considered appropriate for use in this analysis.

Refer to **Appendix C** for Rational Method results.

Table 5-4 – Rational Method Comparison – 1 % AEP Event

Pre-Development Scenario			Post-Development Scenario		
Catchment ID	1% AEP - Peak Discharges (m ³ /s)		Catchment ID	1% AEP - Peak Discharges (m ³ /s)	
	Rational Method	XPSWMM		Rational Method	XPSWMM
Ext1	0.29	0.36	P05	1.32	1.06
Ext2	0.55	0.63	P06	0.72	0.62
Ext3	0.28	0.34	P07	1.51	1.18
Ext4	3.74	3.54	P08	0.56	0.48
Ext5	0.24	0.31	P09	1.47	1.35
Ext6	1.49	1.72	P10	3.25	2.93
C01	1.13	1.00	P01	2.39	2.22
C02	1.19	0.98	P12	2.12	1.83
C03	0.90	0.81	P02	1.16	1.00
C04	0.97	0.84	P03	0.78	0.71
C05	0.19	0.19	Ext 6B	0.31	0.30
C06	0.62	0.64			
C07	0.45	0.48			
C08	0.56	0.59			
C09	0.71	0.73			
C10	2.28	2.15			
C11	0.57	0.60			
C12	0.97	0.91			

5.5 Hydraulic Modelling

The unsteady state, 1D hydrodynamic model XPSWMM was used to analyse the flood behaviour of the site with the proposed mitigation measures. Flood levels, discharge and velocity can be extracted from the model as functions of time at required locations.

The key objective of the hydraulic modelling was to demonstrate that the post development peak discharge from the site for the 63% to 1% AEP events is less than or equal to the pre-development peak discharge.

5.6 Drainage Reserve

As detailed in the previous SBSMP reports, and in the constructed plans for the development, a 30m drainage reserve has been constructed, which shall allow stormwater runoff to be captured and mitigated through the site.

The previously approved civil plans are based off a number of channel sections shown below in **Figure 5-1**, with corresponding channel sections illustrated in **Figure 5-2** to **Figure 5-4**.

Scaled civil drainage drawings which correspond to the figures below have been provided in **Appendix D**, in accordance with the Information Request by LCC dated 5th Oct 2018.

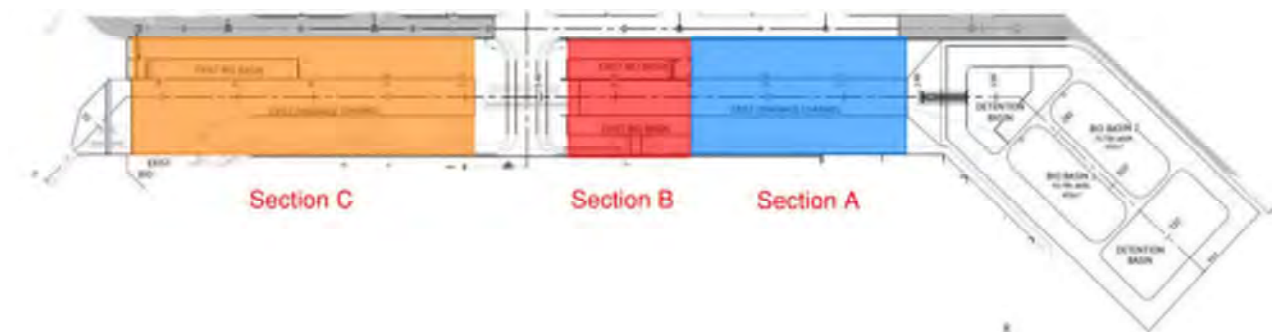


Figure 5-1 – Drainage Reserve Cross-Section Regions

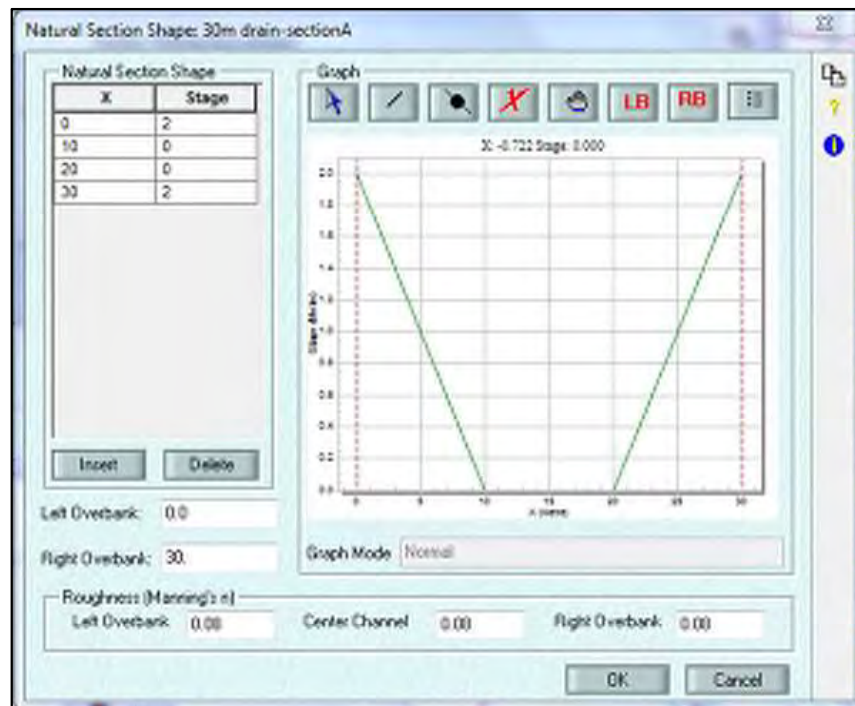


Figure 5-2 – Constructed Drainage Reserve Profile - Section A

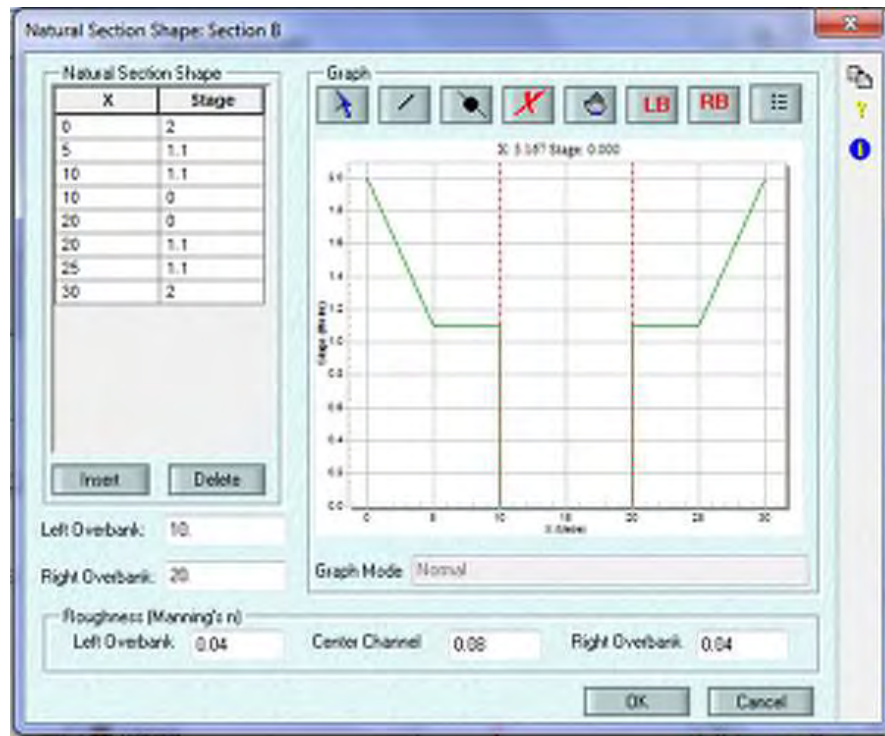


Figure 5-3– Constructed Drainage Reserve Profile - Section B (Upstream of Road Crossing)

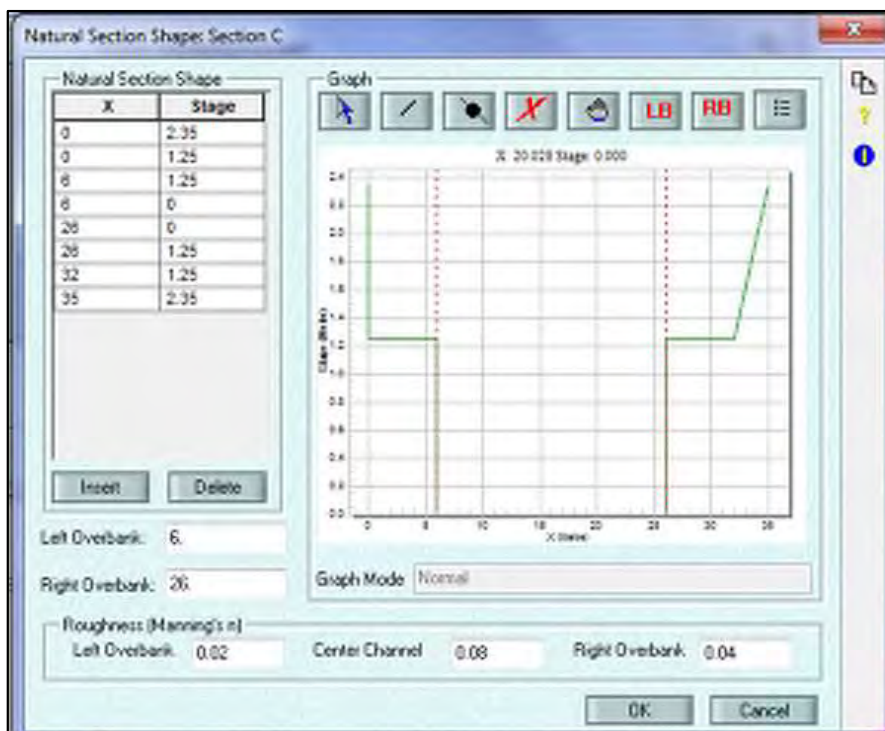


Figure 5-4 – Constructed Drainage Reserve Profile - Section C (Upstream of Existing Downstream Channel)

5.7 Mitigation Measures

Modelling was undertaken to demonstrate the anticipated performance of the constructed drainage reserve and proposed detention basin. The purpose of the stormwater infrastructure is to detain the increase in peak flow rate as a result of the development and ensure the pre-development peak flow rates are being maintained at the LPD and flows entering the existing downstream drainage channel. To ensure the pre-development peak discharges are not exceeded for all AEP events, staged outlet structures have been constructed within the drainage channel areas for mitigation purposes.

To achieve the above objectives, two control structures have been constructed (at time of writing of this report) within the drainage channel reserve. Furthermore, as the project has evolved, an additional On-Site Drainage (OSD) area has been proposed, which shall mitigate post-developed flows for portions of the site, and the external neighbouring development to the south. Mitigation shall be achieved via a combination of low-flow pipes, orifice and riser pit control structure, and high-flow bypass (weir) structures. These are located:

1. At the internal road crossing of the drainage reserve, and
2. At the upstream end of the existing concrete lined drain in the existing relocatable home/caravan park.
3. Dedicated OSD located at the south-west corner of the site.

The location of these control structures is illustrated in **Figure 5-5** and **Figure 5-6**. These have also been detailed within **Appendix D** for further clarity.

5.7.1 Constructed Hydraulic Control Structures

Location 1

Control of runoff at Location 1 will be provided by the road formation, combined with the following culvert arrangement:

- 2 x 600mm RCPs at 24.80m AHD;
- 2 x 900mm RCPs at 25.15m AHD; and
- Road level set at 26.329m AHD.

Location 2

Control of runoff at Location 2 will be provided by forming a bund across the drainage reserve, between the proposed development and the existing relocatable home/caravan park. In addition to the bund, the following culvert and weir arrangement will be required:

- 2 x 600mm RCPs at 23.85m AHD;
- 2 x 600mm RCPs at 24.43m AHD; and
- Weir level set at 25.3m AHD.

5.7.2 Proposed OSD Structure (Location 3)

Location 3

An OSD structure (including Bio Basin area) are proposed to locate at south west of the site.

- Orifice 200mm at 28.2m AHD;
- Riser 450x450mm at 28.7m AHD connecting to a low flow 300mmRCP;
- 3 x 750mmRCP low flow pipes at 28.2m AHD;
- Weir level set at 29.2 AHD.

Refer to **Table 5-5** for the proposed OSD Elevation-Area Relationship details.

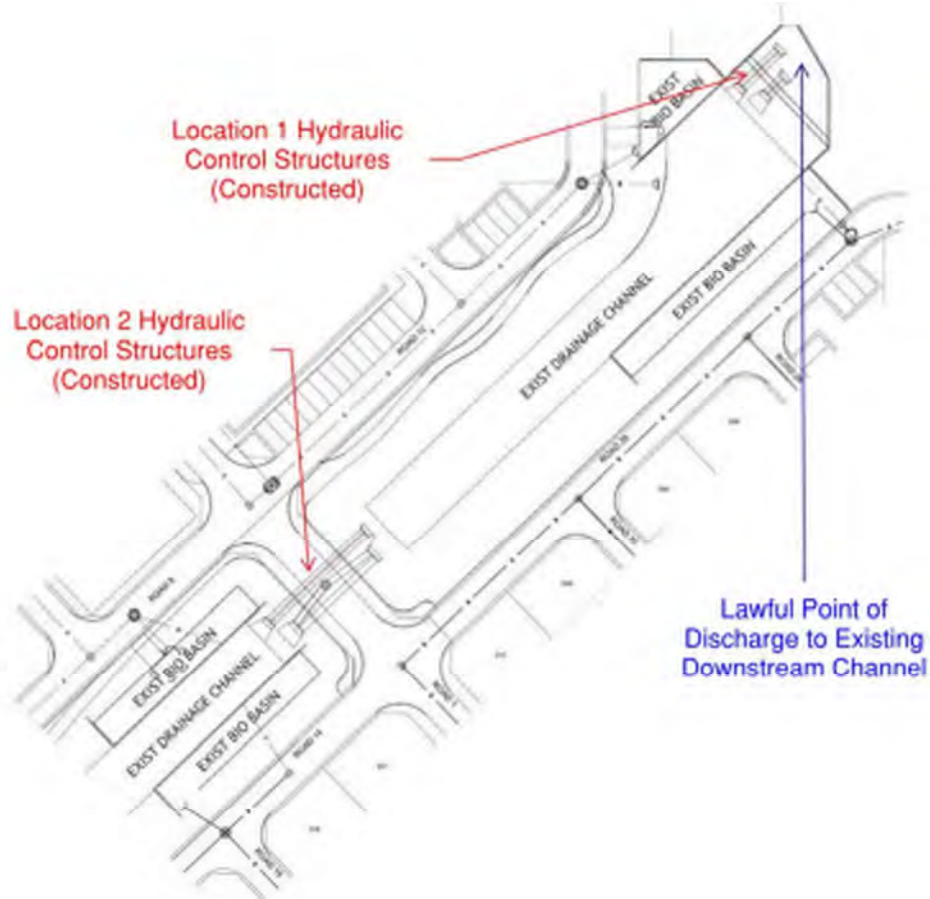


Figure 5-5 - Location of Constructed Flow Control Structures (Drainage Channel)

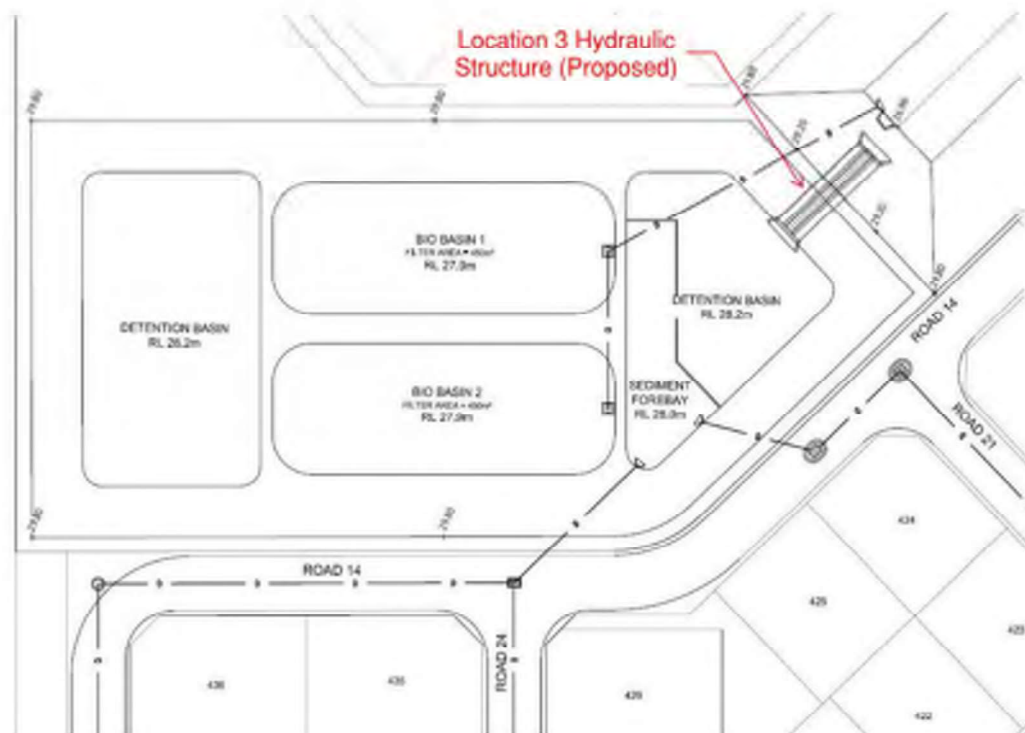


Figure 5-6 - Location of Proposed Flow Control Structures (OSD Structure)

Table 5-5 – OSD Elevation Area relationship

RL (m)	Area (m ²)
28.0	971
28.2	1163
28.4	2381
28.6	2549
28.8	2721
29.0	2899
29.2	3081
29.4	3268
29.5	3363

5.8 LPD Flow Rate Assessment

The peak flow rate has been assessed at the LPDs to determine the difference in peak flow rates as a result of the development and mitigation structures. **Table 5-6** summarises the peak flow rates from the site in the pre- and post-development (mitigated) scenarios, demonstrates that there are no increase in flow rates from pre to post-development as a result of the increase in impervious surfaces.

Table 5-6 – Peak Flow Impact Assessment

Event	Outlet 1			Outlet 2		
	Peak-Discharge (m ³ /s)		Change in Peak Flow	Peak-Discharge (m ³ /s)		Change in Peak Flow
	Pre-Development	Post-Development (Mitigated)		Pre-Development	Post-Development (Mitigated)	
1% AEP	12.56	10.44	-17%	0.19	0	-100%

5.9 Performance of Mitigation and Detention Volumes

The performance of the control structures has been assessed by comparing the mitigated post-development peak flow rate with the pre-development peak flow rate.

The three locations used to assess the performance is shown below in **Figure 5-7**. The location of the 'Discharge to Existing Channel' is also demonstrated in **Figure 5-5** above for clarity.

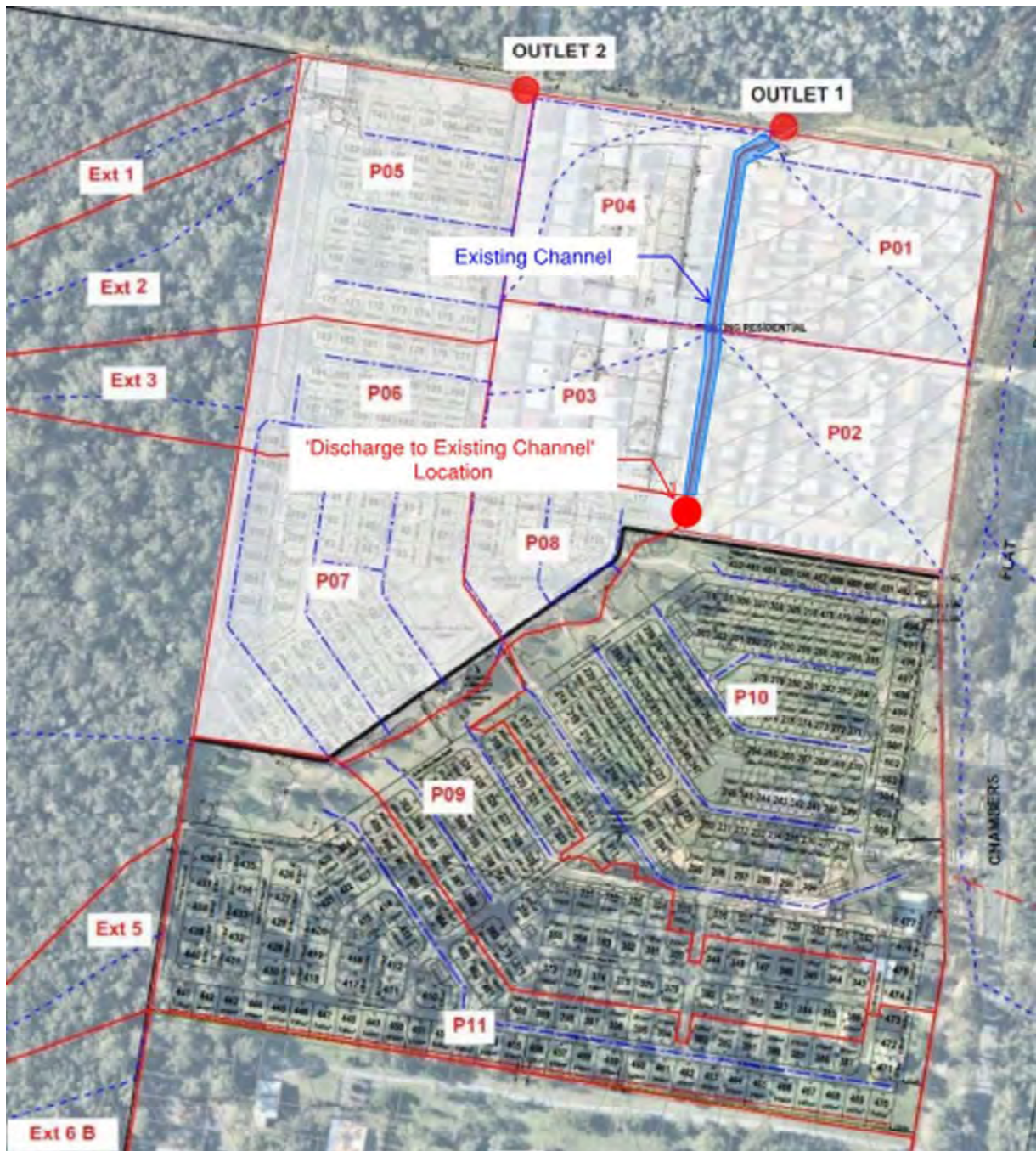


Figure 5-7 - Location of Peak Flow Comparisons

A summary of the peak flow assessment for the three identified location above is tabulated below in **Table 5-7**.

Table 5-7 – Performance of Control Structures

AEP Event	Discharge to Existing Channel		Outlet 1		Outlet 2	
	Pre-Development (m ³ /s)	Post-Development – mitigated (m ³ /s)	Pre-Development (m ³ /s)	Post-Development – mitigated (m ³ /s)	Pre-Development (m ³ /s)	Post-Development – mitigated (m ³ /s)
1%	9.81	7.65	12.56	10.44	0.19	0.00
2%	8.63	6.32	11.15	9.02	0.04	0.00
5%	7.16	5.50	9.21	7.32	0.00	0.00
10%	5.87	4.60	7.61	6.24	0.00	0.00
18%	5.00	3.84	6.45	5.55	0.00	0.00
39%	3.44	2.82	4.41	4.40	0.00	0.00
63%	2.26	2.01	2.92	2.82	0.00	0.00

As can be seen in **Table 5-7**, the post-development peak discharges have been mitigated to below the pre-development peak discharges for the 63% to 1% AEP events. It also indicates that the flows discharged from the subject site into the existing downstream channel have been reduced via the use of mitigation structures.

5.10 Water Surface Levels

The cross-sectional peak water surface levels at particular reporting locations have been provided below in **Table 5-8**, which correspond to the reporting locations illustrated in **Figure 5-8**. The 1% AEP Event peak water surface levels demonstrate that the flows are contained within the drainage reserve. For clarity, the 1% AEP Peak Water Surface Level and top of channel banks have been illustrated in the scaled longitudinal and cross-sectional civil drawings within **Appendix D**.

Table 5-8 – Peak Water Surface Levels within Drainage Reserve

Reporting ID	Design AEP Event (m ³ /s)						
	1%	2%	5%	10%	18%	39%	63%
Basin	29.429	29.367	29.258	29.074	28.955	28.773	28.644
CS1	27.450	27.411	27.351	27.316	27.291	27.241	27.193
CS2	26.968	26.922	26.853	26.817	26.792	26.741	26.693
CS3	26.605	26.530	26.379	26.330	26.299	26.243	26.193
CS4	26.499	26.410	26.094	25.990	25.928	25.815	25.739
CS5	26.455	26.364	26.009	25.879	25.792	25.604	25.480
CS6	26.428	26.336	25.986	25.848	25.757	25.540	25.379
CS7	25.614	25.549	25.496	25.444	25.393	25.161	24.968
CS8	25.585	25.523	25.477	25.427	25.377	25.133	24.917
CS9	25.561	25.503	25.461	25.413	25.364	25.111	24.872
CS10	25.548	25.492	25.453	25.406	25.358	25.103	24.858
CS11	25.538	25.484	25.447	25.401	25.354	25.097	24.850
CS12	25.536	25.482	25.446	25.400	25.353	25.097	24.849
CS13	25.534	25.481	25.444	25.399	25.352	25.096	24.849



Figure 5-8 – Water Surface Level Reporting Locations

The cross-sections provided in **Appendix D** demonstrates that the entire constructed channel has been designed to contain a freeboard of 300mm to the adjacent road and proposed lots, in accordance with QUDM and LCC requirements.

The peak water level through the channel was used to determine the detention volume provided by the proposed drainage reserve and the control structures. In doing so, it has been determined that approximately **3,672m³** of detention storage is provided within the proposed drainage reserve. The proposed OSD Basin at the south-west corner of the site provides approximately **3,366m³** for mitigation for runoff from the southern side of the amended proposed site and the additional 3ha from the southern neighbouring area (in accordance with the agreement between the northern and southern developers). The utilisation of the combined storage and mitigation systems demonstrate that the post-development flows are maintained at pre-development peak flow rates.

5.11 Road Crossings

In accordance with the *Queensland Urban Drainage Manual* (QUDM), 2008 Section 10.02, all flow over roadways must satisfy the following conditions:

- A maximum Depth x Velocity product of 0.4m²/s, and
- A maximum flow depth of 200mm.

Table 5-9 provides an assessment of the flow conditions at the internal road crossing for the 1% AEP storm event.

Table 5-9 - Assessment of Flow Conditions at Road Crossing

Description	1% AEP Value
Road Level (m AHD)	26.33
Peak Water Level (m AHD)	26.47
Flow Depth over Road (mm)	140
Peak Flow Rate over Road (m ³ /s)	1.77
Flow Velocity over Road (m/s)	0.63
Velocity x Depth Product (m ² /s)	0.09

The results in **Table 5-9** demonstrate compliance with these above conditions for the internal road crossing for the major storm event.

6 SUMMARY

This SMP has been developed for the amended proposed residential development located on Chambers Flat Road. A treatment train has been devised and updated in accordance with the latest modelling guidelines, to ensure the current layout achieves the relevant water quality objectives for runoff exiting the site, and that pre-development peak flow rates can be maintained.

Based on this assessment, the following conclusions have been made:

- The relevant water quality objective for the site are load based reduction targets of:
 - TSS 80%;
 - TP 60%;
 - TN 45%; and
 - GP 90%.
- These water quality objectives can be achieved by continuing to implement the treatment train consisting of multiple bioretention systems, which provide a total of **1,682m²** of filter area (at 0.5m depth) located at multiple regions throughout the proposed development site.
- Detention of runoff will be required to ensure pre-development peak flow rates from the site are maintained. A storage volume of approximately **3,672m³** and **3,366m³** have been modelled, in accordance with the constructed drainage channel and the proposed OSD located in the south-west corner of the site, respectively. The XPSWMM modelling demonstrates that the peak post-development stormwater flows from the site have been maintained at the pre-development peak flow rates or better, and furthermore demonstrate that QUDM safety objectives have been achieved within the site.

7 QUALIFICATIONS

Our analysis and overall approach has been specifically catered for the particular requirements of **Ingenia Communities**, and may not be applicable beyond this scope. For this reason, any other third parties are not authorised to utilise this report without further input and advice from Premise Australia Pty Ltd.

Premise Australia Pty Ltd has relied on the information provided within **Section 4** of this report.

This report has modelled the water quality components of the report via the latest industry standards and in accordance with relevant design standards to ensure accurate water quality device design. Consequently, pollutant loads are subject to change based on unforeseen and future circumstances, hence appropriate design parameters have been adopted within the relevant water quality models.

8 RPEQ CERTIFICATION

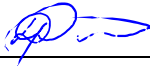
As Registered Professional Engineer of Queensland (RPEQ) for this project, I certify that the modelling undertaken as part of this assessment has been undertaken in accordance with current engineering best practice and complies with the LCC Planning Scheme.

Name: Daniel Niven

RPEQ No: 12581

Date: 17 January 2019

Signature:

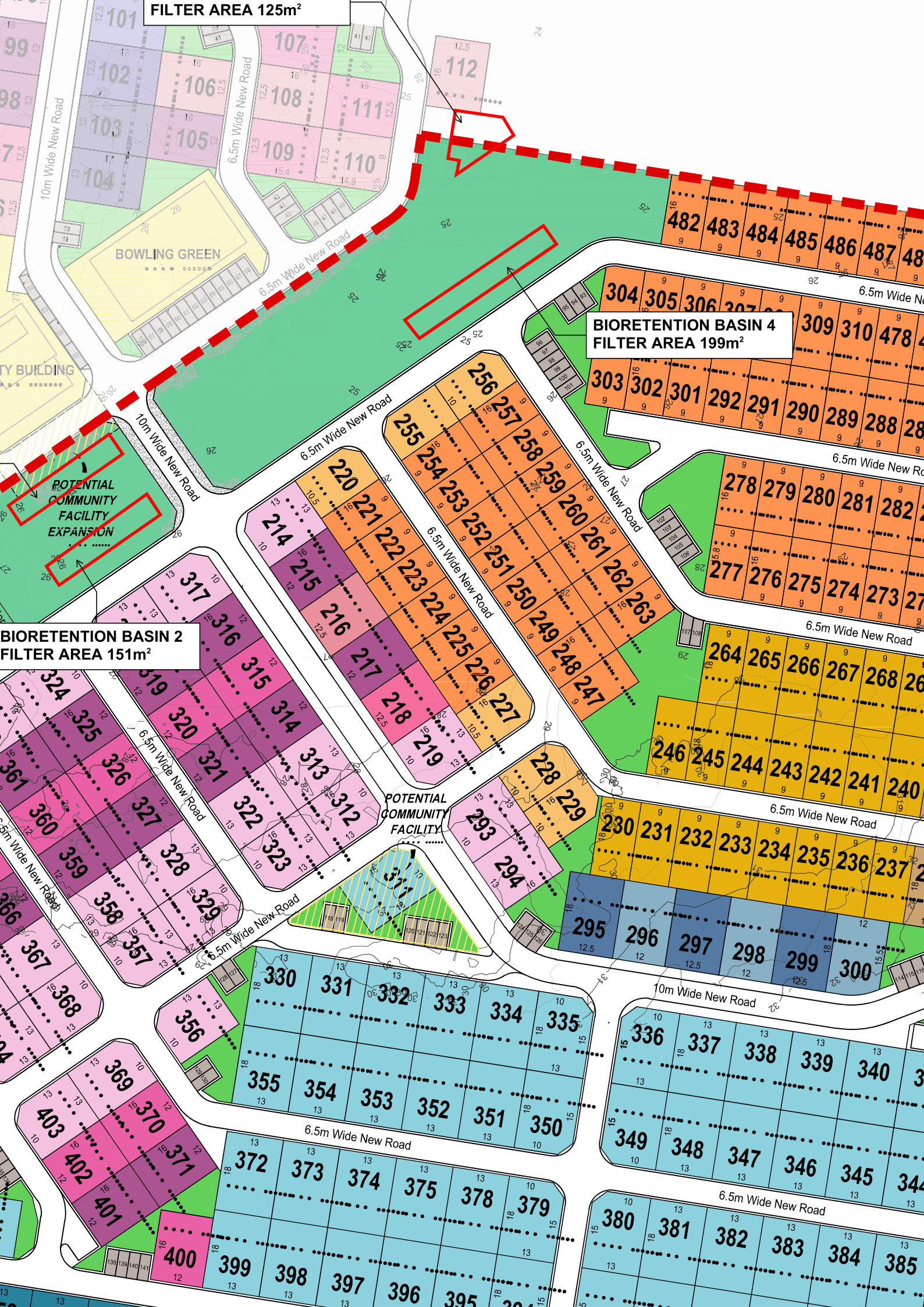
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9 REFERENCES

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**APPENDIX A
PROPOSED DEVELOPMENT PLAN
(BY OTHERS)**

FILTER AREA 125m²



BIORETENTION BASIN 4
FILTER AREA 199m²

BIORETENTION BASIN 2
FILTER AREA 151m²

POTENTIAL
COMMUNITY
FACILITY

POTENTIAL
COMMUNITY
FACILITY
EXPANSION

APPENDIX B
PRE AND POST DEVELOPMENT CATCHMENT PLANS





OUTLET 1

LAWFUL POINT



CHAMBERS ROAD

FLAT

CHAMBERS

EXISTING RESIDENTIAL

P01

P02

P04

P03

P08

P10

P07

P09

P11

P12

Ext 1

Ext 2

Ext 3

Ext 4

Ext 5

Ext 6 B

Ext 7

Ext 8

Note: : so deve 3ha o 32 R



WQ5

WQ3

WQ1

WQ4

WQ2

WQ6

EXISTING RESIDENTIAL

FLAT

CHAMBERS ROAD

3ha
(NEIGHBOUR)

Note: ...
so ...
deve ...
3ha o ...
32 R ...

RP68700

APPENDIX C

RATIONAL METHOD CALCULATIONS

RATIONAL METHOD SUMMARY

Pre-Development Scenario											Job Number	ING-0001							
Catchment	Ext1	Ext2	Ext3	Ext4	Ext5	Ext6	C01	C02	C03	C04			C05	C06	C07	C08	C09	C10	C11
Area (ha)	0.89	1.69	0.85	11.07	0.69	4.36	1.45	1.53	1.15	1.24	0.33	1.37	0.94	1.24	1.51	5.62	1.33	2.17	24.05.2018
											Designer	WIN							
											Reviewer	MR							

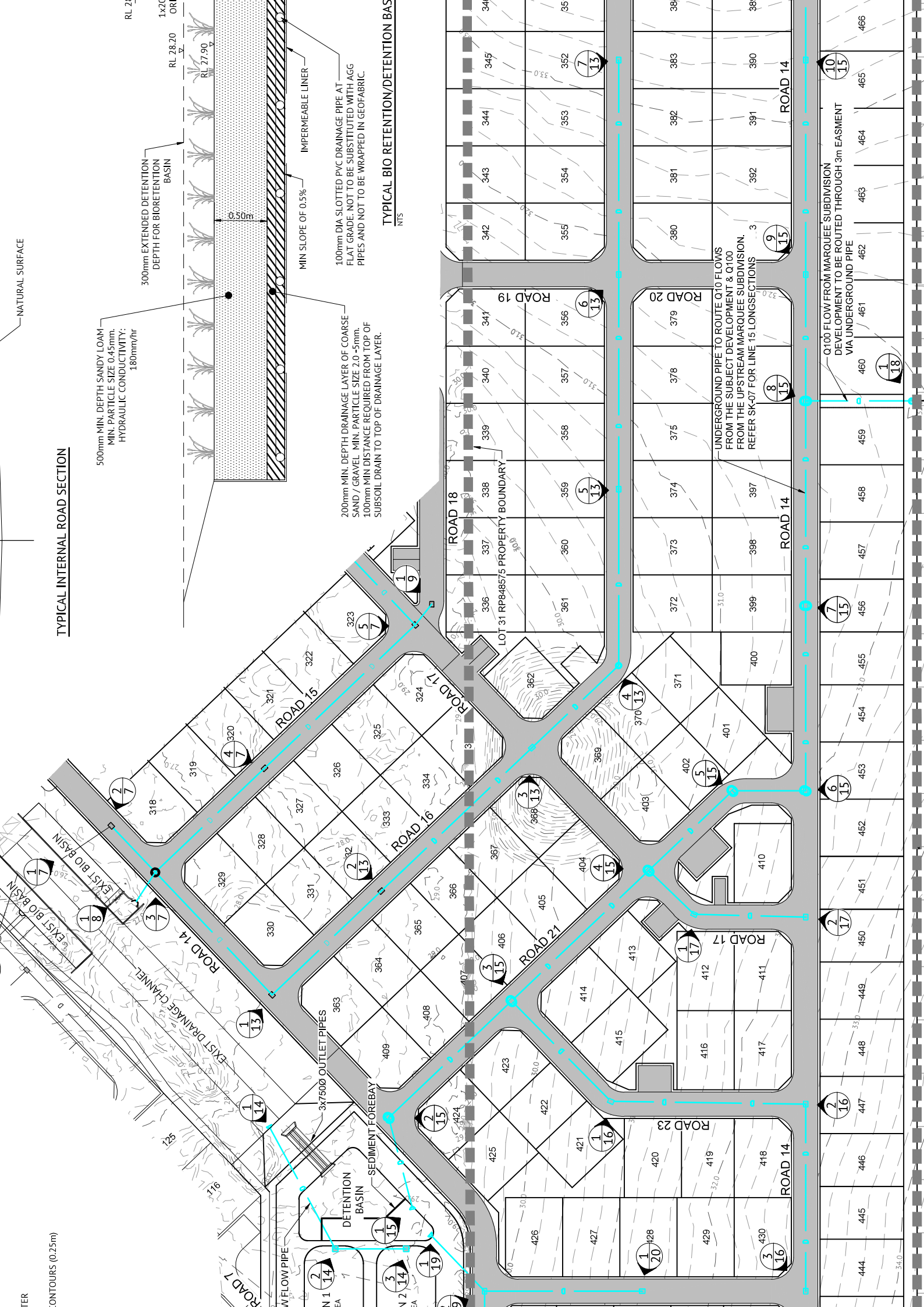


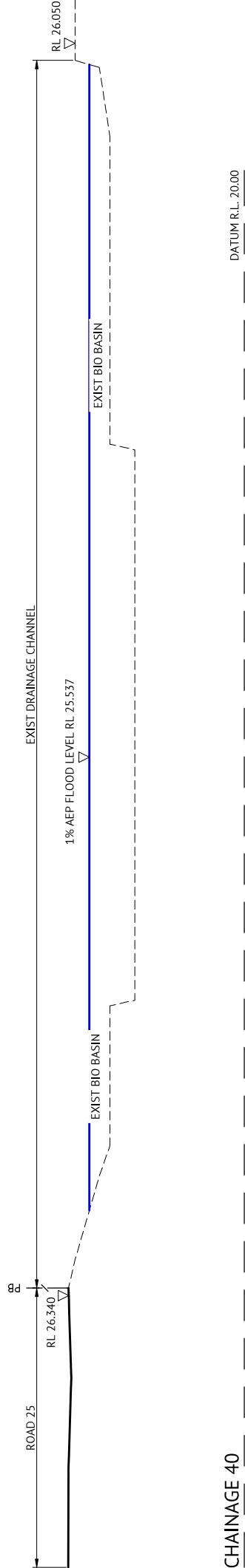
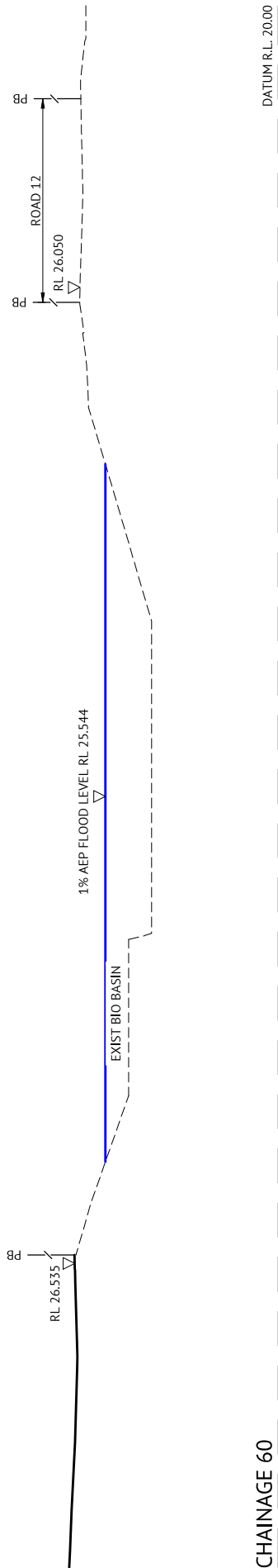
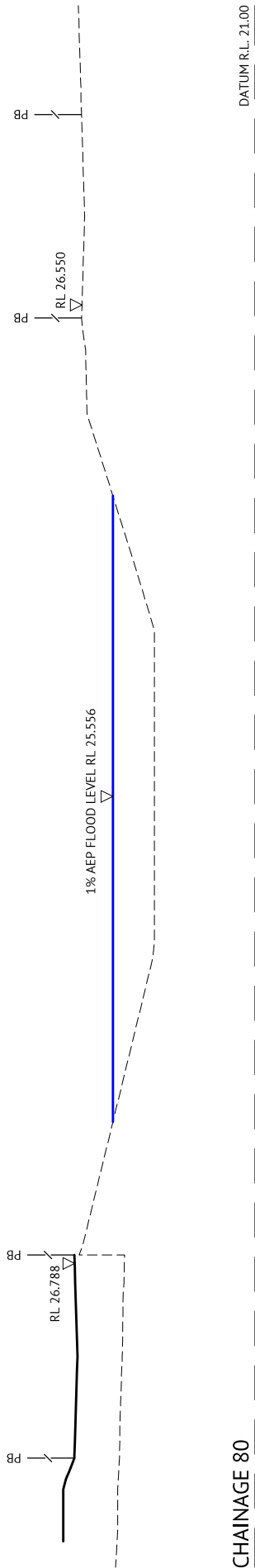
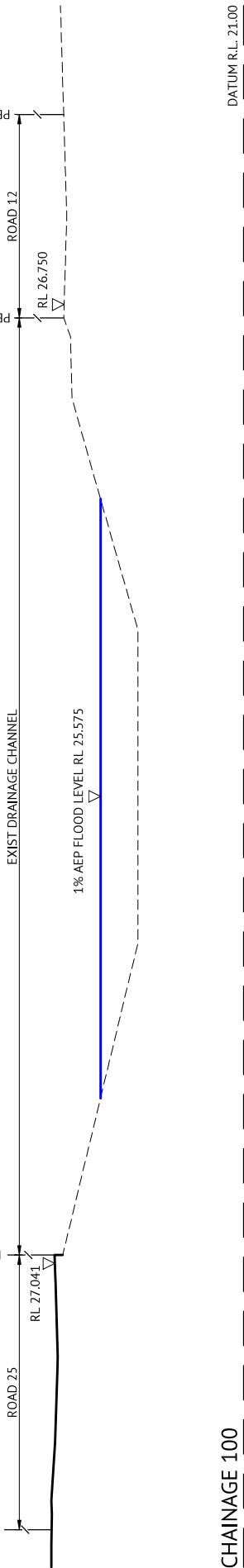
Pre Development Flows																								
Catchment Name	Area (ha)	t _c (min)	I ₁₀₀ (mm/hr)	C ₁₀₀	1% AEP (m ² /s)	I ₅₀ (mm/hr)	C ₅₀	2% AEP (m ² /s)	I ₃₀ (mm/hr)	C ₃₀	5% AEP (m ² /s)	I ₁₀ (mm/hr)	C ₁₀	10% AEP (m ² /s)	I ₅ (mm/hr)	C ₅	18% AEP (m ² /s)	I ₂ (mm/hr)	C ₂	39% AEP (m ² /s)	I ₁ (mm/hr)	C ₁	65% AEP (m ² /s)	Q _{3month} (m ² /s)
Ext1	0.89	50	141	0.84	0.29	127	0.81	0.25	109	0.74	0.20	95	0.70	0.16	85	0.67	0.14	68	0.60	0.10	53	0.56	0.07	0.04
Ext2	1.69	31	139	0.84	0.55	125	0.81	0.47	107	0.74	0.37	94	0.70	0.31	84	0.67	0.26	67	0.60	0.19	52	0.56	0.14	0.07
Ext3	0.85	29	143	0.84	0.28	129	0.81	0.24	111	0.74	0.19	97	0.70	0.16	87	0.67	0.14	69	0.60	0.10	54	0.56	0.07	0.04
Ext4	11.07	28	145	0.84	3.74	130	0.81	3.22	112	0.74	2.54	98	0.70	2.11	88	0.67	1.79	70	0.60	1.27	55	0.56	0.94	0.47
Ext5	0.69	27	149	0.84	0.24	135	0.81	0.21	116	0.74	0.16	101	0.70	0.13	91	0.67	0.12	72	0.60	0.08	56	0.56	0.06	0.03
Ext6	4.36	28	147	0.84	1.49	132	0.81	1.29	114	0.74	1.01	99	0.70	0.86	89	0.67	0.71	71	0.60	0.51	55	0.56	0.38	0.19
C01	1.45	6	281	1.00	1.13	254	0.99	1.01	219	0.90	0.80	192	0.86	0.66	173	0.82	0.57	139	0.73	0.43	109	0.69	0.30	0.15
C02	1.53	6	279	1.00	1.19	252	0.99	1.06	217	0.90	0.83	191	0.86	0.70	172	0.82	0.60	138	0.73	0.43	108	0.69	0.32	0.16
C03	1.15	6	282	1.00	0.90	255	0.99	0.81	220	0.90	0.64	193	0.86	0.53	174	0.82	0.45	140	0.73	0.33	110	0.69	0.24	0.12
C04	1.24	6	281	1.00	0.97	254	0.99	0.86	219	0.90	0.68	192	0.86	0.57	173	0.82	0.49	139	0.73	0.35	109	0.69	0.26	0.13
C05	0.33	8	254	0.84	0.19	230	0.81	0.17	198	0.74	0.13	174	0.70	0.11	156	0.67	0.09	126	0.60	0.07	98	0.56	0.05	0.03
C06	1.37	15	194	0.84	0.62	175	0.81	0.54	151	0.74	0.42	132	0.70	0.35	118	0.67	0.30	95	0.60	0.22	75	0.56	0.16	0.08
C07	0.94	13	206	0.84	0.45	186	0.81	0.39	160	0.74	0.31	140	0.70	0.26	126	0.67	0.22	101	0.60	0.16	79	0.56	0.12	0.06
C08	1.24	15	194	0.84	0.56	175	0.81	0.49	151	0.74	0.38	132	0.70	0.32	118	0.67	0.27	95	0.60	0.20	75	0.56	0.14	0.07
C09	1.51	14	202	0.84	0.71	183	0.81	0.62	157	0.74	0.48	137	0.70	0.40	123	0.67	0.34	99	0.60	0.25	77	0.56	0.18	0.09
C10	5.62	19	174	0.84	2.28	157	0.81	1.97	135	0.74	1.55	118	0.70	1.29	106	0.67	1.10	85	0.60	0.79	66	0.56	0.58	0.29
C11	1.33	18	182	0.84	0.57	164	0.81	0.49	141	0.74	0.38	123	0.70	0.32	111	0.67	0.27	89	0.60	0.20	70	0.56	0.14	0.07
C12	2.17	16	191	0.84	0.97	172	0.81	0.83	148	0.74	0.66	130	0.70	0.55	116	0.67	0.47	93	0.60	0.34	73	0.56	0.25	0.12
Total	39.43				17.13			14.92			11.73			9.78			8.33			5.97				4.40

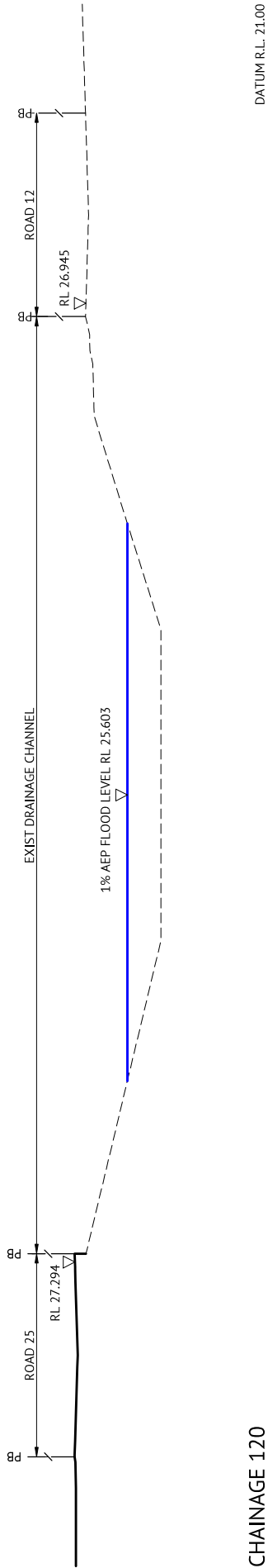
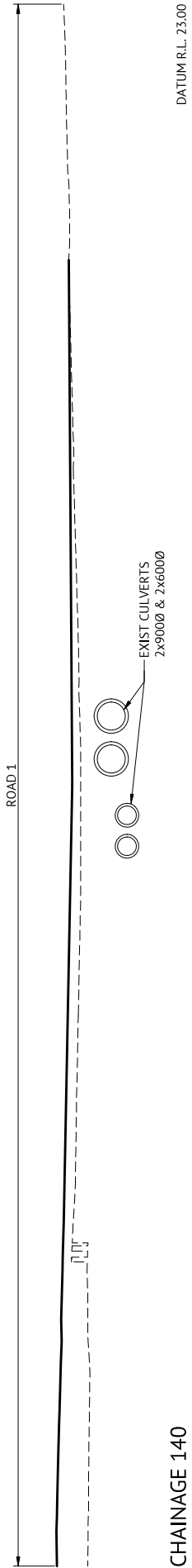
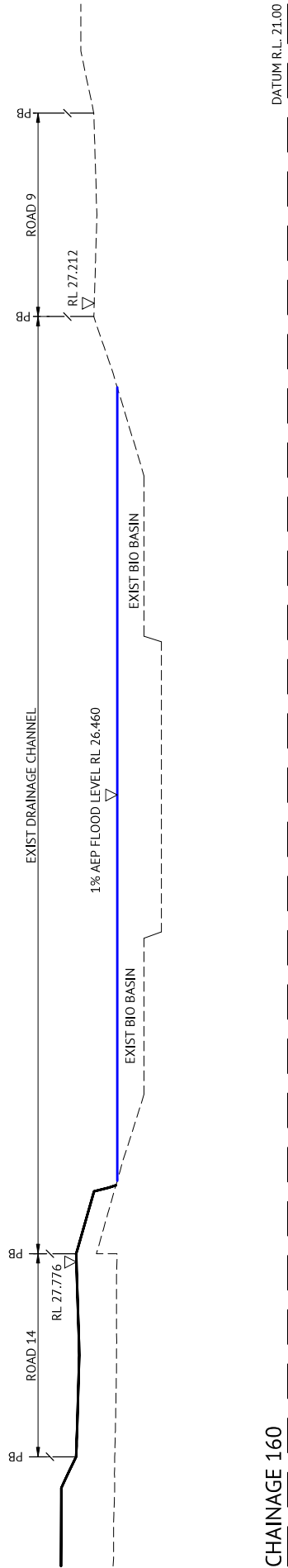
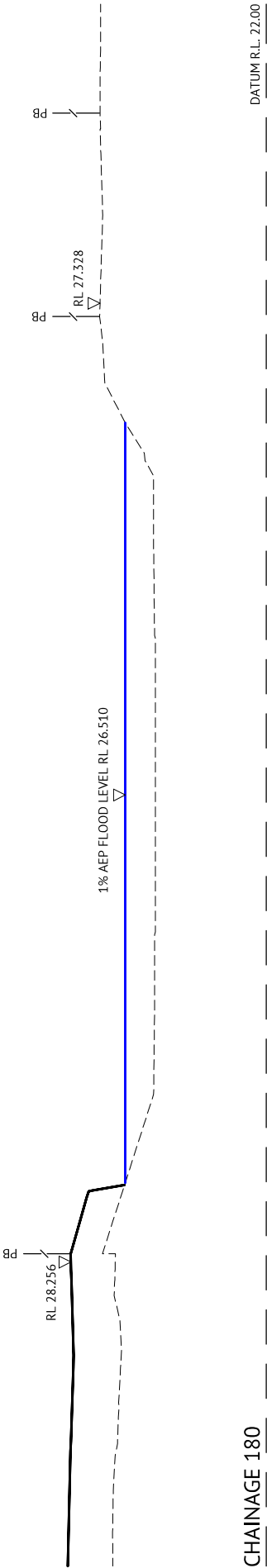
Note: Q3month is calculated as 50% of 65% AEP

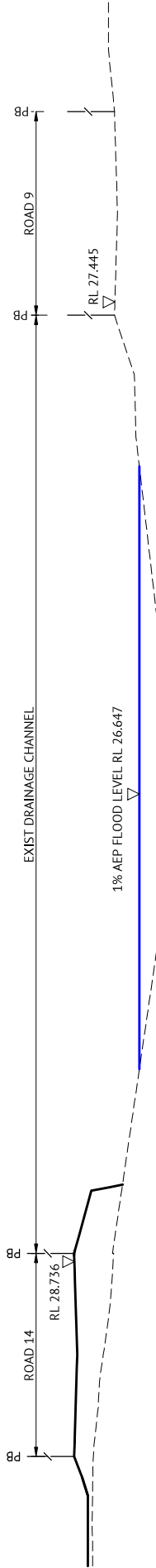
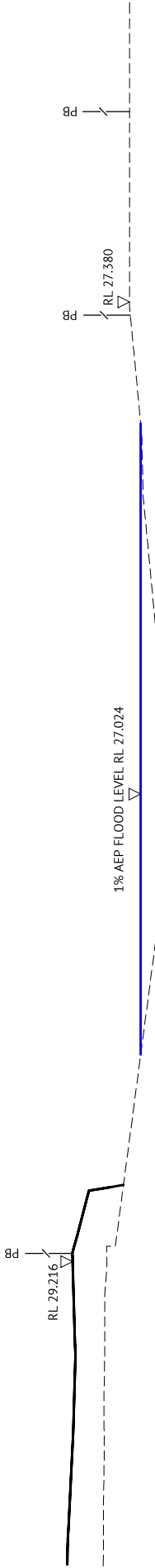
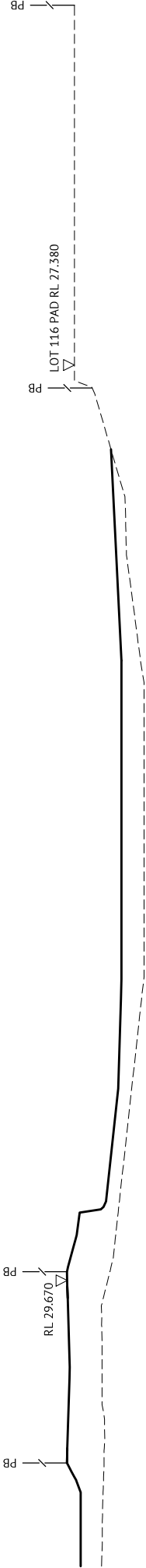
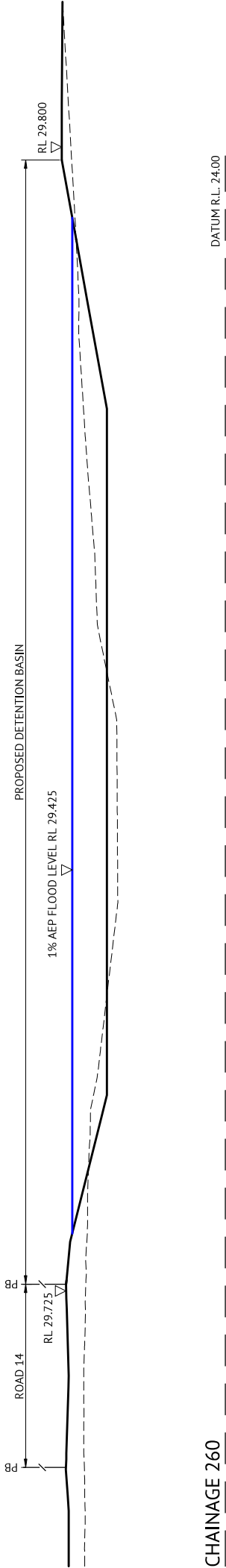
Post Development Flows																								
Catchment Name	Area (ha)	t _c (min)	I ₁₀₀ (mm/hr)	C ₁₀₀	1% AEP (m ² /s)	I ₅₀ (mm/hr)	C ₅₀	2% AEP (m ² /s)	I ₃₀ (mm/hr)	C ₃₀	5% AEP (m ² /s)	I ₁₀ (mm/hr)	C ₁₀	10% AEP (m ² /s)	I ₅ (mm/hr)	C ₅	18% AEP (m ² /s)	I ₂ (mm/hr)	C ₂	39% AEP (m ² /s)	I ₁ (mm/hr)	C ₁	65% AEP (m ² /s)	Q3month (m ² /s)
Ext 68	0.73	1.79	180	0.84	0.31	162	0.81	0.26	140	0.74	0.21	122	0.70	0.17	110	0.67	0.15	88	0.60	0.11	69	0.56	0.08	0.04
P05	1.70	5.6	287	1.00	1.36	260	0.99	1.22	224	0.90	0.96	197	0.86	0.80	177	0.82	0.68	143	0.73	0.49	112	0.69	0.36	0.18
P06	0.92	5.4	291	1.00	0.74	263	0.99	0.66	227	0.90	0.52	200	0.86	0.44	179	0.82	0.37	145	0.73	0.27	114	0.69	0.20	0.10
P07	2.00	5.8	284	1.00	1.57	257	0.99	1.41	221	0.90	1.11	194	0.86	0.93	175	0.82	0.79	141	0.73	0.57	110	0.69	0.42	0.21
P08	0.71	5.4	291	1.00	0.58	263	0.99	0.52	227	0.90	0.41	200	0.86	0.34	179	0.82	0.29	145	0.73	0.21	114	0.69	0.15	0.08
P09	2.19	6.3	277	1.00	1.69	251	0.99	1.51	216	0.90	1.19	190	0.86	0.99	171	0.82	0.85	137	0.73	0.61	108	0.69	0.45	0.22
P10	2.75	6.3	277	1.00	2.12	251	0.99	1.89	216	0.90	1.49	190	0.86	1.24	171	0.82	1.06	137	0.73	0.77	108	0.69	0.56	0.28
P11	4.41	7.1	265	1.00	3.25	239	0.99	2.90	206	0.90	2.28	181	0.86	1.91	162	0.82	1.63	131	0.73	1.18	103	0.69	0.87	0.43
P12	3.00	7.9	254	1.00	2.12	230	0.99	1.89	198	0.90	1.49	174	0.86	1.25	156	0.82	1.06	126	0.73	0.77	98	0.69	0.56	0.28
Total	15.41				7.63			6.81			5.36			4.49			3.83			2.76			2.03	

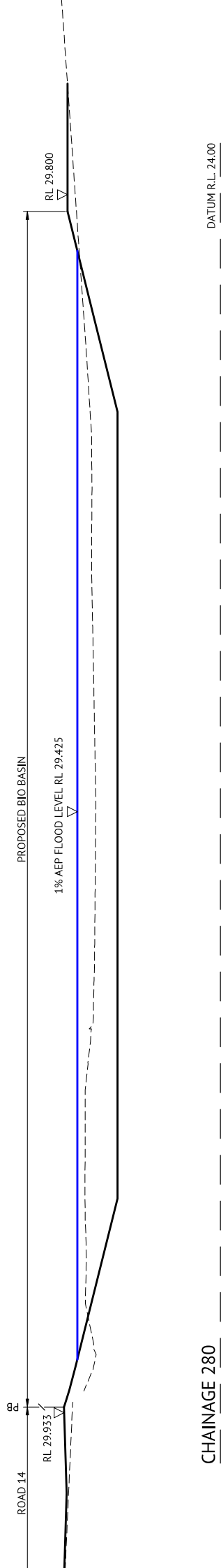
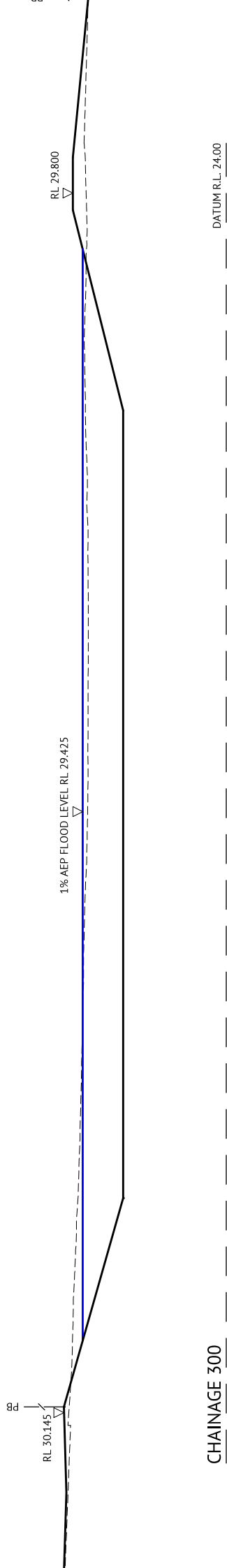
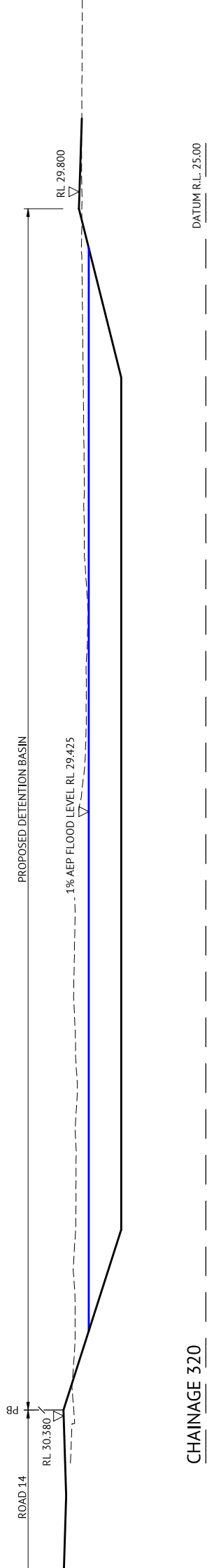
APPENDIX D
CIVIL DRAINAGE PLANS

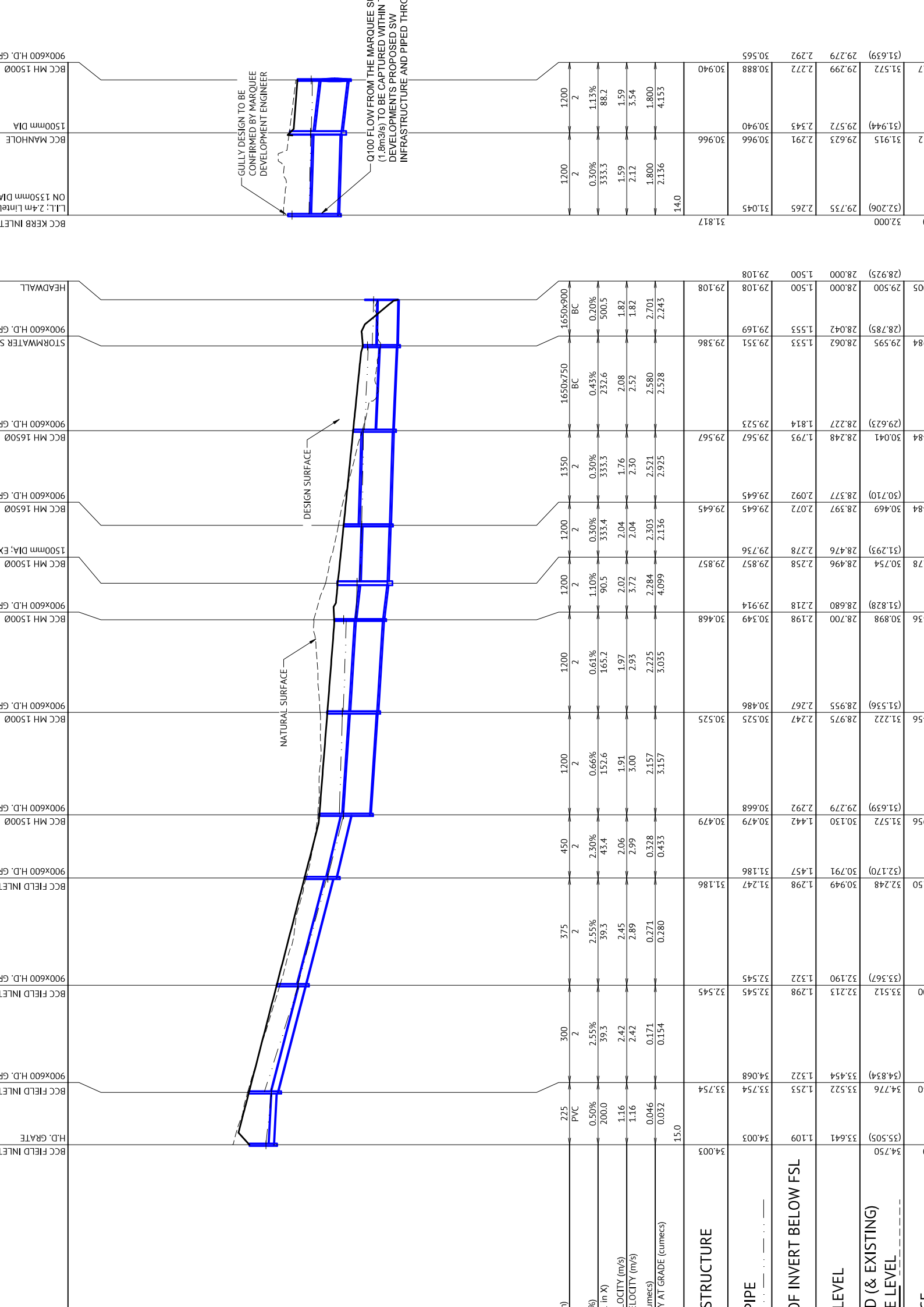














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