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PLANS AND DOCUMENTS referred to in the ULDA APPROVAL dated 10/8/12

SITE BASED STORMWATER MANAGEMENT PLAN

PROPOSED DEVELOPMENT AT
TANNUM SANDS
FOR
ULDA

M2410 – Revision A
February 2012

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1.0 INTRODUCTION

JFP Urban Consultants Pty Ltd was commissioned by ULDA Pty Ltd to complete a Site Based Stormwater Management Plan (SBSMP) for the proposed development at Dahl Road, Tannum Sands – Stages 1A, 1B, 2 & 3. It is proposed to construct 141 lots with sealed driveways, stormwater drainage infrastructure, services and associated landscaping works (refer to Appendix A for layout).

Hydraulic modelling was undertaken to determine the peak stormwater flow rates, retention and flood management requirements for the development using the XP-SWMM program. The rational method was used to provide confirmation of the XP-SWMM results.

The SBSMP employs the principles of Water Sensitive Urban Design (WSUD), with a focus on reducing pollutant export and storm flows while improving the visual aesthetics of urban landscape as a part of the greater concept of Ecologically Sustainable Development (ESD). Stormwater quality modelling has been undertaken using the MUSIC 4.0 program as part of the WSUD treatment train selection process. The water quality improvement devices chosen for the development will significantly reduce post development stormwater pollutant loads in accordance with the compliance objectives in the *South East Queensland Regional Plan 2009 2031, Implementation Guideline No. 7.*

2.0 SITE CHARACTERISTICS

2.1 LOCATION

The subject site is located at Lot 6 and 7 on SP 228432 and Lot 133 on CTN 1953. A site Locality plan is shown in Figure 1. The total developable site comprises a total area of approximately 51ha with Stages 1A, 1B, 2 & 3 comprising approximately 10.7ha.

2.2 TOPOGRAPHY AND SITE DRAINAGE

The site exhibits gradients with slopes in the order of 3.0%-12.0%. A series of ridges run through the site dividing the catchment into five drainage areas. Stormwater from catchments A, B and C generally flow north to south, catchments D and E generally flow east to west. Refer Figure 2

Figure 1 – Site Location Plan

3.0 LOCAL CATCHMENT HYDRAULICS

3.1 INTRODUCTION

The Rational Method has been used to calculate the peak stormwater runoff flow rates for the pre and post development scenarios for all ARI (Annual Recurrence Interval) from the 1 year to 100 year event.

3.2 CATCHMENTS

The site has been divided into smaller sub-catchments for hydraulic analysis. Table 1 details the pre and post development catchment conditions.

Table 1 – Pre and Post Development Catchment Details

Catchment Area	Pre Development Area (ha)	Pre Development Fraction Impervious	Post Development Fraction Impervious
Catchment B			
B1	0.910	0.05	0.55
B2	5.055	0.05	0.55
B3	4.317	0.05	0.55
B4	1.934	0.05	0.55
Catchment C			
C1	1.883	0.05	0.55
C2	5.601	0.05	0.55
C3	3.025	0.05	0.55
C4	4.737	0.05	0.55
C5	2.547	0.05	0.55
C6	14.787	0.05	0.05*
C7	2.393	0.05	0.55
Total	47.189	-	-

*Catchment C6 is external to the site and currently bushland.

3.3 CO-EFFICIENT OF RUNOFF

For undeveloped areas a C_{10} value of 0.70 and has been calculated using a C^{I}_{10} value determined in the rational method analysis.

For developed areas a C_{10} value of 0.79 has been calculated using a C^{I}_{10} value determined in the rational method analysis.

3.4 TIME OF CONCENTRATION

For the pre developed site, the time of concentration was calculated using the Friend's Equation for overland sheet flow. The maximum length of overland flow was determined to be 50m after which channel flow (minor rills and gully erosion channels) will convey stormwater flows to the outfall. The time of concentration calculations undertaken for the 100yr ARI storm pre developed site are provided in Table 2.

For the post developed site, the time of concentration has been determined from standard inlet times and stream / pipe velocities for the remaining length of flow (Table 4.06.1, 4.06.5 & Section

4.06.9, QUDM, 2nd Edition, 2007). The time of concentration for the 100yr ARI storm post development site are provided in Table 3.

The time of concentration was calculated for Catchment Areas B tc1, C tc2 and C2 tc3 for the pre and post-developed scenarios. Refer Catchment Area plan in Appendix A.

Table 2 – Time of Concentration – Pre Development

Catchment	Overland Sheet Flow Length (m)	Mannings Co-efficient (Sheet Flow)	Slope % (Sheet Flow)	Stream flow length (m)	Stream velocity (m/s)	Stream Gradient (%)	Tc (min)
Pre Developed Catchment B tc1	50	0.035	1	482	0.7	2.5	25.0
Pre Developed Catchment C tc2	50	0.035	8	5.16	0.7	2.7	21.0
Pre Developed Catchment C tc3	50	0.035	12	650	0.7	3.5	24.0

Table 3 – Time of Concentration – Post Development

Catchment	Overland Sheet Flow Length (m)	Mannings Co-efficient (Sheet Flow)	Slope % (Sheet Flow)	Std Inlet Time (min)	Stream flow length (m)	Stream velocity (m/s)	Stream Gradient (%)	Pipe flow length (m)	Pipe velocity (m/s)	Tc (min)
Post Developed Catchment B tc1	-	-	-	10.0	370	0.7	2.5	131	2.0	20.0
Post Developed Catchment C tc2	-	-	-	8.0	315	0.7	2.5	346	3.0	17.0
Post Developed Catchment C tc3	50	0.035	12	-	650	0.7	3.5	-	-	24.0

3.5 RAINFALL INTENSITIES

The rainfall intensities for the pre and post developed scenarios were determined from the IFD chart for the Gladstone Region calculated from AR&R, 1987.

Table 4 – Rainfall Intensities

	Time of Conc (mins)	Q1 (mm/hr)	Q2 (mm/hr)	Q5 (mm/hr)	Q10 (mm/hr)	Q20 (mm/hr)	Q50 (mm/hr)	Q100 (mm/hr)
Pre Developed Catchment B tc1	25	57	74	94	106	123	146	163
Post Developed Catchment B tc1	20	64	82	105	119	137	163	183
Pre Developed Catchment C tc2	21	62	80	102	116	134	159	178
Post Developed Catchment C tc2	17	69	89	113	128	148	176	198
Pre Developed Catchment C tc3	24	58	75	96	108	125	149	167
Post Developed Catchment C tc3	25	57	74	94	106	123	146	163

3.6 PEAK FLOW RATES

Table 5 to Table 7 detail the results of the Rational Method analysis performed on the pre and post developed catchment.

Table 5 – Peak Stormwater Runoff for Various ARI - Catchment B tc1

Subject Site	Q1 (m^3/s)	Q2 (m^3/s)	Q5 (m^3/s)	Q10 (m^3/s)	Q20 (m^3/s)	Q50 (m^3/s)	Q100 (m^3/s)
Pre Developed	1.08	1.49	2.12	2.52	3.07	3.99	4.65
Post Developed	1.37	1.87	2.67	3.19	3.86	5.02	5.89
Increase	0.29	0.37	0.55	0.67	0.79	1.04	1.24

Table 6 – Peak Stormwater Runoff for Various ARI Events - Catchment C tc2

Subject Site	Q1 (m^3/s)	Q2 (m^3/s)	Q5 (m^3/s)	Q10 (m^3/s)	Q20 (m^3/s)	Q50 (m^3/s)	Q100 (m^3/s)
Pre Developed	1.66	2.26	3.22	3.84	4.65	6.05	7.06
Post Developed	2.16	2.95	4.19	5.00	6.07	7.90	9.28
Increase	0.49	0.69	0.97	1.16	1.42	1.86	2.22

Table 7 – Peak Stormwater Runoff for Various ARI Events - Catchment C tc3

Subject Site	Q1 (m^3/s)	Q2 (m^3/s)	Q5 (m^3/s)	Q10 (m^3/s)	Q20 (m^3/s)	Q50 (m^3/s)	Q100 (m^3/s)
Pre Developed	1.55	2.13	3.05	3.61	4.38	5.72	6.69
Post Developed	1.58	2.17	3.10	3.67	4.46	5.83	6.81
Increase	0.03	0.04	0.05	0.06	0.08	0.10	0.12

4.0 FLOOD ASSESSMENT

4.1 INTRODUCTION

This report provides detailed analysis of flows and predicted flood levels through the site. The hydraulic modelling package XP-SWMM was used to analyse flood levels within the site.

4.2 CATCHMENT DESCRIPTION

Levels through the catchment vary from approximately RL 32.5m AHD to 5.75m AHD.

Existing culverts occur at two locations along Dahl Road. Each culvert is a single 900mm RCP.

4.3 CATCHMENT

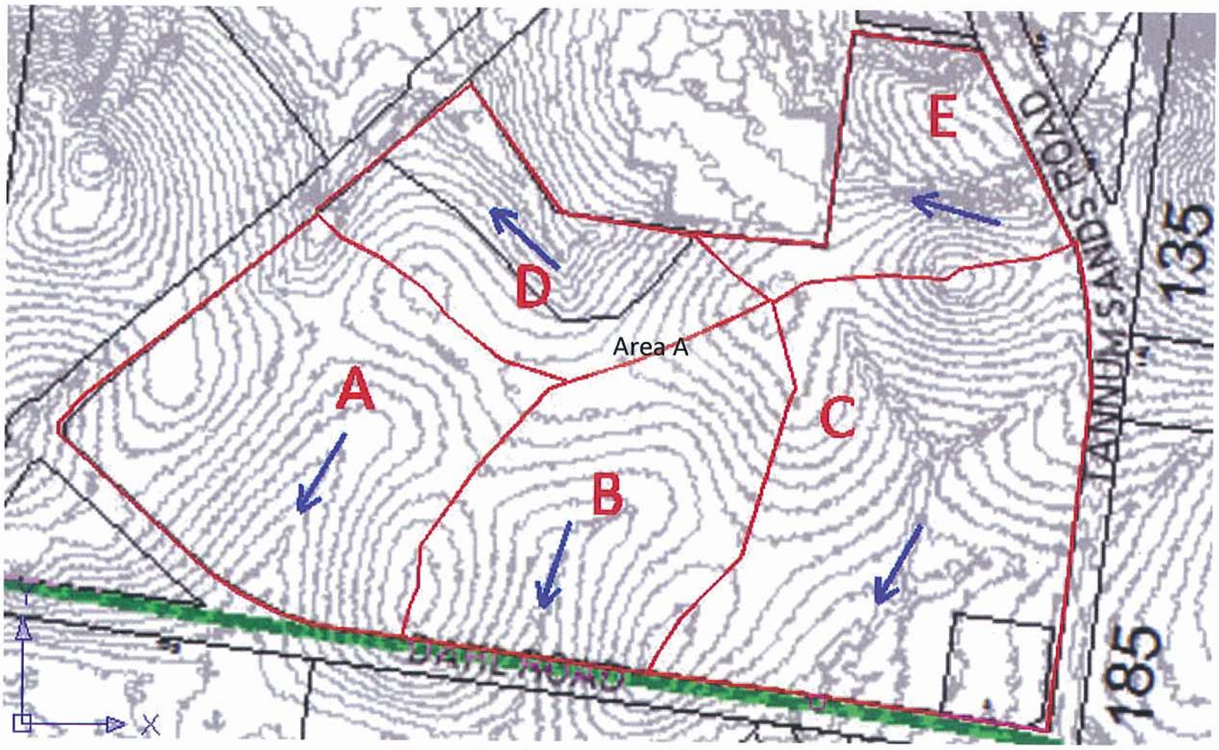
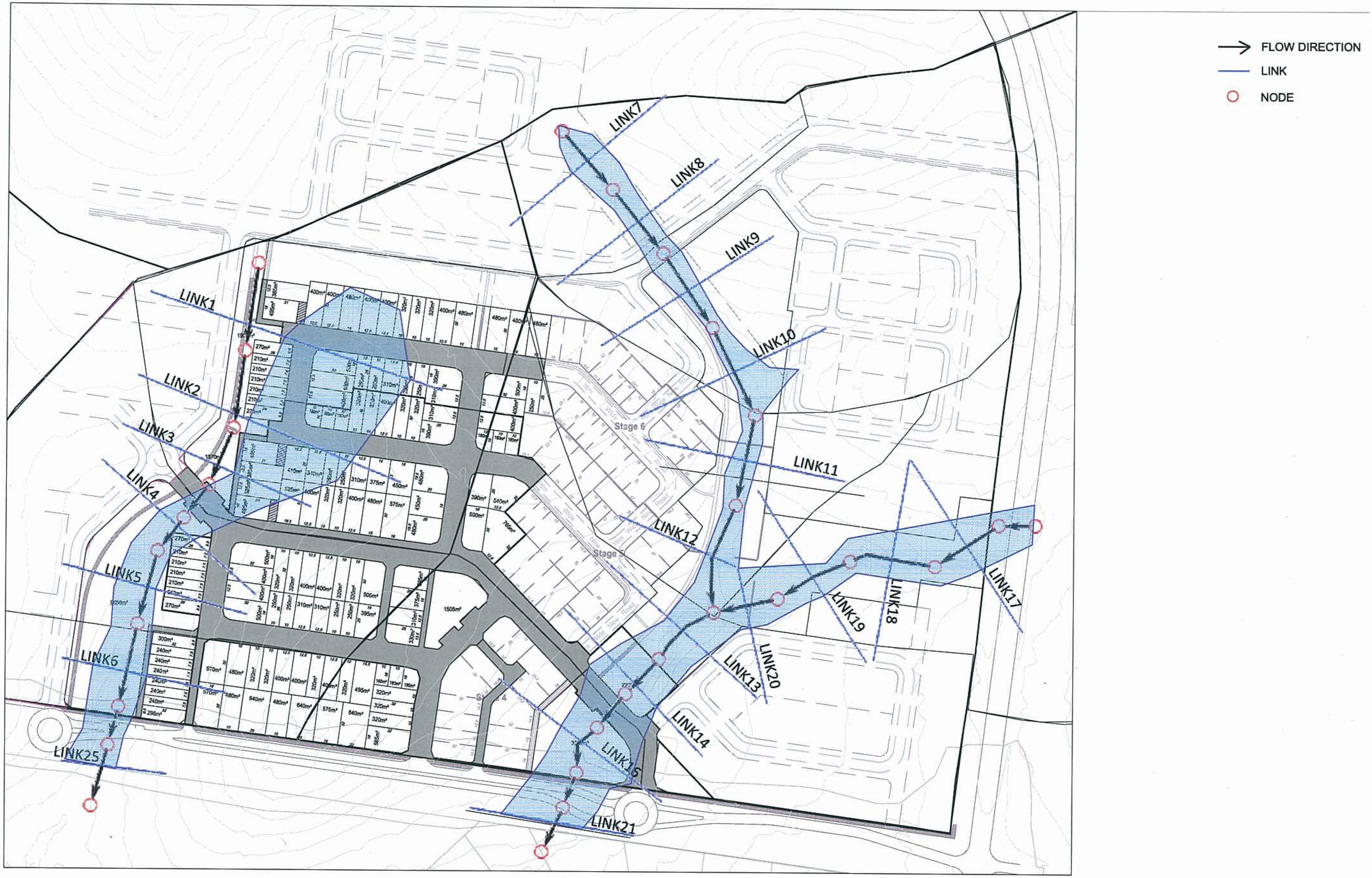


Figure 2 – Upstream Catchment

Figure 3 – 100yr ARI Pre Developed Flood Inundation



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SCALE : 1:2500 @ A3
DATE : FEBRUARY 2012
JOB No. : M2410
REV : A

PROJECT:
TANNUM SANDS
HYDRAULIC ASSESSMENT

FIGURE 3 - PRE DEVELOPED Q100 FLOOD INUNDATION

FILE NAME: M2410_SBSMP_PRE_INUNDATION_01.DWG

Figure 4 – 100yr ARI Post Developed Flood Inundation

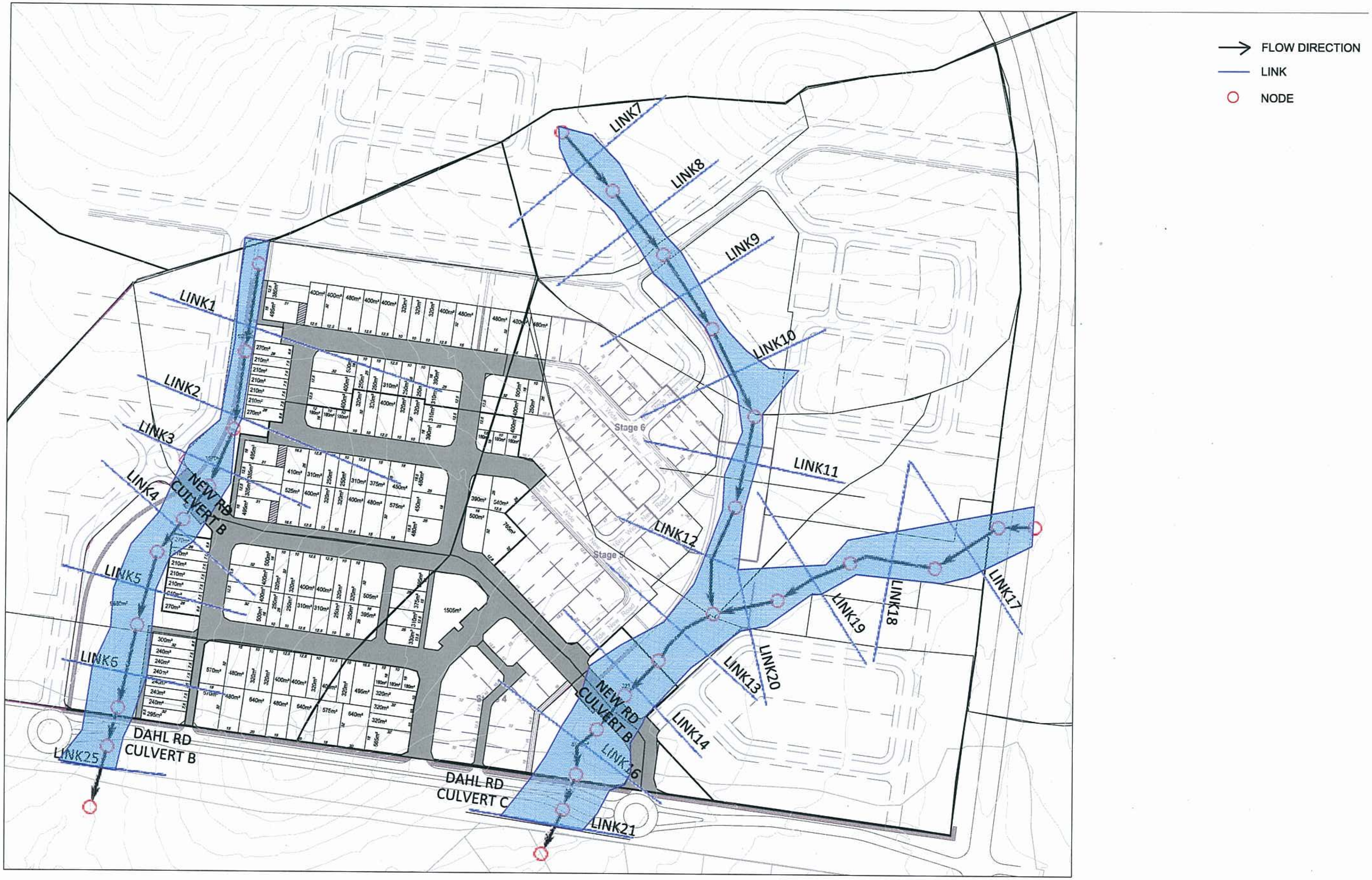

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FIGURE 4 - POST DEVELOPED Q100 INUNDATION

FILE NAME: M2410_SBSMP_POST_INUNDATION01.DWG

4.4 CATCHMENT FLOWS

Utilising the Rational Method flows calculated in Section 3.0, results of 100yrARI analysis for each sub-catchment are provided in Table 8.

Table 8 – 100yr ARI Catchment Flows (m^3/s)

		Pre Developed Site			Post Developed Site		
Catchment	**Area**	**Intensities (mm/hr)**	**Runoff Co-efficient**	**100yr ARI Flow (m^3/s)**	**Intensities (mm/hr)**	**Runoff Co-efficient**	**100yr ARI Flow (m^3/s)**
B1	0.910	163	0. 84	0.35	183	0. 95	0.44
B2	5.055	163	0. 84	1.92	183	0. 95	2.44
B3	4.317	163	0. 84	1.64	183	0. 95	2.08
B4	1.934	163	0. 84	0.74	183	0. 95	0.93
C1	1.883	170	0. 84	0.75	198	0. 95	0.98
C2	5.601	170	0. 84	2.22	198	0. 95	2.92
C3	3.025	170	0. 84	1.2	198	0. 95	1.58
C4	4.737	170	0. 84	1.88	198	0. 95	2.47
C5	2.547	170	0. 84	1.01	198	0. 95	1.33
C6	14.787	167	0. 84	5.76	167	0.84	5.76
C7	2.393	167	0. 84	0.93	167	0.95	1.05

4.5 MANNING'S COEFFICENT

Manning's coefficients used in the XPSWMM model are provided in Table 9.

Table 9 – Reach Data

Area Type	Manning's n
Heavily Vegetated Areas	0.080
Road	0.015

4.6 TAILWATER LEVELS

A mannings formula calculation was undertaken for the 100yr ARI storm at the outlet of the pipe from Catchment B to estimate a suitable tailwater level. Results show the 100yr ARI flood level at the outlet to be lower than the obvert of the existing culvert. The obvert of the existing pipe at 6.64mAHD was adopted to provide a conservative estimate of the tailwater level.

The 100yr ARI flood level is below the outlet from Catchment C. The obvert of the pipe was adopted at 8.28 mAHD for the tailwater level at this location. This is considered a conservative estimate of downstream flood levels.

4.7 CULVERT SUMMARY

Table 10 provides a summary of the culverts used in the XP SWMM Model.

Table 10 – Culvert Summary

Culvert	RCP/Box Culvert (mm)	Inlet Invert (m)	Outlet Invert (m)	Slope (%)
Pre Developed Case				
Tannum Sands Rd Culverts*	3/750	12.25	12.15	1.0
Dahl Road Culvert B	1/900 RCP	7.31	7.38	-1.1
Dahl Road Culvert C	1/900 RCP	5.94	5.74	3.1
Post Developed Case				
Tannum Sands Rd Culverts*	3/750	12.25	12.15	1.0
Dahl Road Culvert B	525 RCP	7.45	7.25	2.0
	450 RCP	5.94	5.74	2.0
Dahl Road Culvert C**	1/900x900**	7.31	7.38	-1.0
	1/900 RCP	7.31	7.38	-1.0
New Road Catchment B	3/525 RCP	8.96	8.54	1.9
New Road Catchment C	6/1050 RCP	8.50	8.30	1.3

*Existing culverts under Tannum Sands Road were assumed to convey a 50yrARI storm event.

**The proposed 900x900 box culvert under Dahl Road (Catchment C) has been modelled with a fauna passage (ledge) incorporated within the pipe work. The fauna passage (ledge) is 400mm high and 300mm wide and allows small animals to travel through the pipe-work during minor storm events.

The proposed culvert design is preliminary and will be reassessed as part of the engineering design process.

4.8 RESULTS

Table 11 and Table 12 provide a summary of predicated flood levels through the site.

Table 11 – Summary of Predicted Flood Levels Catchment B

Link	Pre Developed Site 100yrARI Flood Levels	Post Developed Site 100yrARI Flood Levels	Difference (neg value decrease)
Catchment B			
Link1	13.36	13.33	-0.02
Link2	11.09	10.99	-0.10
Link3	9.28	9.63	0.35
Link4	8.53	8.55	0.02
Link5	8.20	8.29	0.09
Link6	7.64	8.21	0.57
Link25	6.80	6.77	-0.03

Flow over the internal road "New Road B" occurs at a depth of 90mm during a 100yrARI storm event with a road level of 9.5 mAHD (between Link 3 and Link 4).

Flow over Dahl Road (Catchment B) occurs at a depth of 130mm during a 100yrARI storm event with a road level of 8.05 mAHD.

Table 12– **Summary of Predicted Flood Levels Catchment C**

Link	Pre Developed Site 100yrARI Flood Levels	Post Developed Site 100yrARI Flood Levels	Difference (neg value decrease)
Catchment C			
Link7	17.98	18.00	0.02
Link8	15.72	15.77	0.04
Link9	13.16	13.17	0.01
Link10	11.52	11.55	0.04
Link11	10.59	10.63	0.04
Link12	9.97	9.98	0.01
Link13	9.69	9.70	0.01
Link14	9.19	9.67	0.48
Link16	8.72	9.19	0.47
Link17	12.30	12.07	-0.23
Link18	10.90	10.78	-0.12
Link19	10.43	10.44	0.01
Link20	9.97	9.98	0.01
Link21	8.32	8.32	0.00

Results indicate that increased flood levels occur at various locations throughout the site. Lots within these areas are to be filled to ensure flood immunity for the 100yr ARI storm event. Appropriate freeboard will be applied to finished "filled" levels through the site.

Stormwater does not flow over the internal road "New Road C". Flow over Dahl Road (Catchment C) occurs at a depth of 160mm during a 100yrARI storm event with a road level of 9.0mAHD.

These flows levels over roads should be viewed as very preliminary and subject to change during the engineering design process.

5.0 STORMWATER QUANTITY MANAGEMENT

5.1 INTRODUCTION

XP-SWMM Version 13.00 was utilised to analyse the pre developed and post developed site flows through the creek to determine a suitable outlet arrangements under Dahl Road to ensure a non-worsening of flows from the site for a range of storm events.

5.2 SITE OUTFLOW

The pre and post developed site outflow for a range of ARI storm events is detailed in Table 13 and Table 14. Detailed XP-SWMM results for the 100 year ARI storm event are attached in Appendix B.

Table 13 – Outflow from Catchment B

Scenario	Q1 (m^3/s)	Q2 (m^3/s)	Q5 (m^3/s)	Q10 (m^3/s)	Q20 (m^3/s)	Q50 (m^3/s)	Q100 (m^3/s)
Allowable Discharge (Pre Developed)	0.80	1.25	1.67	2.32	2.99	3.95	4.41
Outflow (Post Developed Mitigated)	0.74	1.04	1.67	2.09	2.69	3.73	4.07
Non- Worsening	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*

Table 14 – Outflow from Catchment C

Scenario	Q1 (m^3/s)	Q2 (m^3/s)	Q5 (m^3/s)	Q10 (m^3/s)	Q20 (m^3/s)	Q50 (m^3/s)	Q100 (m^3/s)
Allowable Discharge (Pre Developed)	2.31	3.85	5.73	6.93	8.80	11.45	12.73
Outflow (Post Developed Mitigated)	2.10	3.24	5.26	6.35	8.11	10.68	12.09
Non- Worsening	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*

The proposed outlet systems provide sufficient flow retention to attenuate post development peak flow rates to below pre development rates.

Detention storage available within rainwater tanks will further reduce inflows to the stormwater system.

6.0 CONSTRUCTION PHASE WATER QUALITY MANAGEMENT

6.1 POLLUTANTS

Typical pollutants generated during the construction phase of the developments are detailed in Table 15 (Gold Coast City Council, 2005).

Table 15 – Construction Phase Pollutants

Pollutant	Potential Source	Priority
Litter	Paper, construction packaging, food packaging, cement bags, off-cuts	High
Sediment	Unprotected exposed soils and stockpiles during earthworks and building works	High
Hydrocarbons	Fuel and Oil spills, leaks from construction equipment and temporary carpark areas	High
Toxic Materials	Cement Slurry, asphalt primer, solvents, cleaning agents, washwaters (eg. from tile works)	Medium

6.2 POTENTIAL IMPACTS

The addition of coarse / fine sediments and other pollutants to stormwater runoff from the development sites may increase nutrient loads of receiving water, potentially resulting in eutrophication. Impacts on the surrounding environment will be minimised during the construction phase with measures as outlined in this *Site Based Stormwater Management Plan* and *Erosion and Sediment Control Plans* to be provided at the operational works stage.

6.3 PERFORMANCE OBJECTIVES AND INDICATORS

The amount of runoff crossing and leaving the sites should be kept to a minimum during the construction phase. This will restrict soil erosion and mobilisation of sediments and pollutants through and off the site.

Stormwater runoff at discharge points during the construction phase shall conform to the performance criteria as specified in Table 15 (Gold Coast City Council, 2005).

Table 16 – Construction Phase Performance Criteria

Pollutant	Potential Source
Total Suspended Solids	90^{th} Percentile < 50mg/L
pH	6.5 – 9.0
Dissolved Oxygen	90^{th} Percentile > 80% saturation or 6mg/L
Hydrocarbons	No visible sheen on receiving waters
Litter	No visible litter washed from site

6.4 MONITORING AND MAINTENANCE

The general requirement of monitoring during the construction phase will be:

- Work activities are restricted to designated construction areas
- Earthworks and site clearing are undertaken in accordance with the *Erosion and Sediment Control Plans*
- Erosion and sediment control devices are to be constructed/installed in accordance with the *Erosion and Sediment Control Plans*
- Inspection of sediment fences and erosion and sediment control structures/devices on a weekly basis as well as after any rain event exceeding 25mm in 24hrs
- Stormwater discharges from the site are not having any adverse affect on the downstream environment
- Monitoring and recording of the performance of the drainage control devices including water quality testing where required
- Any failure in the stormwater system shall be immediately rectified to prevent uncontrolled discharge from the site
- Any failure to the stormwater system causing damage to surroundings should implement immediate remedial work to the damaged area

6.5 RESPONSIBILITY AND REPORTING

- The contractor shall be responsible for monitoring the performance of all drainage control and erosion and sediment control devices
- Records of any failures to devices should be kept and reported to the Construction Manager
- Regular inspections of the devices shall be reported to the Construction Manager
- Inspections of the devices after heavy rainfall shall be reported to the Construction Manager

7.0 OPERATIONAL PHASEWATER QUALITY MANAGEMENT

7.1 INTRODUCTION

In recent years there have been an increasing number of initiatives to manage the urban water cycle in a more sustainable way. The integration of the urban water cycle within the concept of Ecologically Sustainable Development (ESD) is termed Water Sensitive Urban Design (WSUD). A WSUD treatment train consisting of Bio-Retention Areas and Rainwater Tanks has been proposed to treat all stormwater flows from the operational phase of the development.

7.2 POLLUTANTS

Pollutants typically generated by during the operational phase of the development are detailed in Table 17 (Gold Coast City Council, 2005).

Table 17 – Operational Phase Pollutants

Pollutant	Potential Source
Litter	Construction packaging, construction waste materials, paper, food packaging
Sediment	Exposed soils and stockpiles
Oxygen demanding substances (organic & chemical matter)	Organic or chemical matter
Nutrients	Nitrogen, phosphorus
Pathogens / Faecal coliforms (bacteria & viruses)	Sewage
Heavy metals (often associated with fine sediment)	Sediment runoff
Surfactants (eg. detergents from car washing)	Detergents, cleansing agents

7.3 OBJECTIVES

Table 18 outlines the stormwater pollutant reductions required for the subject site as stipulated within the Department of Environment and Resource Management Draft Urban Stormwater Queensland Best Management Practice Environmental Management Guidelines 2009. These objectives accord with current Gladstone Regional Council guidelines and the provisions of Healthy Waterways.

Table 18 – Operational Phase Water Quality Objectives

Pollutant	Pollutant Reduction Requirements
Suspended Solids	85%
Total Phosphorus	70%
Total Nitrogen	45%
Gross Pollutants	90%

7.4 STORMWATER QUALITY TREATMENT TRAIN

A WSUD treatment train has been developed to meet the reduction criteria established in Table 18. Bio-Retention areas (within garden areas) with a surface / treatment area of 1675m^2 will be utilised to capture and treat overland flows from pervious and impervious areas. Healthy Waterways *'WSUD Technical Guidelines'*, states that to achieve adequate load reduction targets, bio-retention areas are to be sized at approximately 1-2% of the catchment area. Table 19 provides a catchment and bio-filtration area details.

Table 19 – Bio-Retention Area Details

	Catchment Area (ha)	Bio-retention Area (m^2)	Percentage of Catchment Area (%)
Catch B (east)	8.190	1200	1.5
Catch C (west)	3.581	475	1.3

The proposed development of Stages 1A, 1B, 2 & 3 of the site encompasses an area of approximately 10.7 ha with 141 proposed lots. Additional areas have been included within this assessment to provide optimum water quality treatment for subsequent stages of the development. The stormwater quality treatment area modelled in this assessment is 11.77ha with approximately 180 proposed lots.

3kL and 5kL tanks are proposed for units/townhouses and allotments respectively. The rainwater tanks will aid in the reduction of pollutant loadings output from the site, primary treatment will occur within the bio-retention areas.

7.5 MAINTENANCE

A bio-retention filter maintenance plan will be provided at the engineering design stage.

7.6 MUSIC CATCHMENT PARAMETERS

Stormwater quality modelling has been undertaken for the proposed development using the MUSIC 4.0 software program. MUSIC is widely accepted as the industry standard for WSUD modelling and is an excellent tool for comparing treatment train effectiveness. Input parameters for source nodes and treatment devices have been obtained from the MUSIC Modelling guidelines for SEQ Version 1. 50% of the roof area was assumed to discharge to the rainwater tank in the MUSIC analysis. The model layout is provided in Figure 4. Details on each catchment (Source Node) are detailed in Table 20.

Table 20 – MUSIC Catchment Parameters

Catchment	Area (ha)	Percentage Impervious	MUSIC Source Node
Catchment B (east)			
Ground Level B	1.445	25	GROUND LEVEL
Roof to Ground B	1.425	100	ROOF
Roof to Tank B	1.425	100	ROOF
Townhouse Ground Level B	0.195	30	GROUND LEVEL
Townhouse Roof to Ground B	0.188	100	ROOF
Townhouse Roof to Tank B	0.188	100	ROOF
Road/Pathway B	2.510	60	ROAD
Park B*	0.814	5	PARK
Sub Total	8.190	-	-
Catchment C (west)			
Ground Level C	1.003	25	GROUND LEVEL
Roof to Ground C	0.513	100	ROOF
Roof to Tank C	0.513	100	ROOF
Road/Pathway C	1.052	60	ROAD
Park C	0.500	5	PARK
Sub Total	3.581	-	-
TOTAL	11.771	-	-

*Park B was divided evenly between the Catchment B (east) and Catchment B (west).

7.7 MUSIC SOURCE NODE PARAMETERS

The MUSIC source node parameters were taken from the Music Modelling Guidelines *Version 1.0-2010.*

Table 21 – MUSIC Base and Storm Flow Concentration Parameters

Land Use	Parameter	Total Suspended Solids (Log_{10} mg/L)		Total Phosphorus (Log_{10} mg/L)		Total Nitrogen (Log_{10} mg/L)	
		Base Flow	Storm Flow	Base Flow	Storm Flow	Base Flow	Storm Flow
Urban Residential	Mean	1.00	2.18	-0.97	-0.47	0.20	0.26
	Std Deviation	0.34	0.39	0.31	0.31	0.20	0.23

Table 22 – MUSIC Runoff Generation Parameters

Parameter	Urban Residential
Rainfall Threshold (mm)	1
Soil Storage Capacity (mm)	500
Initial Storage (%)	10
Field Capacity (mm)	200
Infiltration Capacity Coefficient a	211
Infiltration Capacity Coefficient b	5
Daily Recharge Rate (%)	28
Daily Base-flow Rate (%)	27
Daily Seepage Rate (%)	0

7.8 MUSIC RAINFALL PARAMETERS

Gladstone meteorological data was utilised for the MUSIC analysis. Modelling of the site was analysed for a period of 10 years (from 1st January 1980 to 31st December 1989) using a 6 minute time-step period. Average potential evapo-transpiration data has been taken from the MUSIC software package.

7.9 MUSIC TREATMENT NODE PARAMETERS

Rainwater Tank:

3kL and 5kL tanks are proposed for units/townhouses and allotments respectively. The water re-use rate for the site was assumed to be 0.117kL/day/house as per SEQ guidelines. Rainwater tanks have been modelled as "grouped" rather than for each individual allotment. The overflow pipe diameter is the product of the standard 90mm overflow pipe multiplied by the square root of the number of contributing tanks. This technique is in accordance with MUSIC version 4.0 guidelines by 'Water by Design' for SEQ. Table 23 details the input parameters used for the rainwater tanks.

Table 23 – MUSIC Rainwater Tank Parameters

	Rain Tanks Catch B (east)		**Rain Tanks Catch C (west)**
Parameter	**Lots**	**Units**	**Lots**
Volume below Overflow Pipe (kL)	570	75	205
Depth above Overflow (m)	0.2	0.2	0.2
Surface Area (m^2)	285	37.5	103
Overflow Pipe Diameter (mm)	961	450	576
Re-use Properties: Daily Demand (kL/day)	5939	801	4122
Re-use Properties Annual Demand (kL/yr)	13.40	2.94	4.82

Bio-Retention Basin:

Details of the proposed bio-retention area are shown in Table 24.

Table 24 – MUSIC Bio-Retention Area Parameters

Parameter	**Bio-retention Catch B (east)**	**Bio-retention Catch C (west)**
Extended Detention Depth (m)	0.3	0.3
Surface Area (m^2)	1200	475
Seepage Loss (mm/hr)	0.0	0.0
Filter Area (m^2)	1200	475
Filter Depth (m)	0.4	0.4
Filter Median Particle Diameter (mm)	0.45	0.45

7.10 MUSIC MODEL

The MUSIC model analysed for the proposed development site is shown in Figure 4.

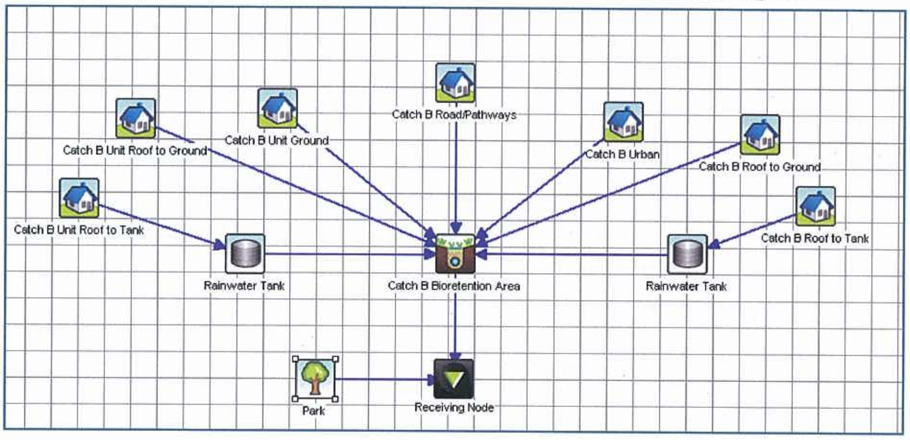


Figure 5 –Music Model Layout Catchment B (east)

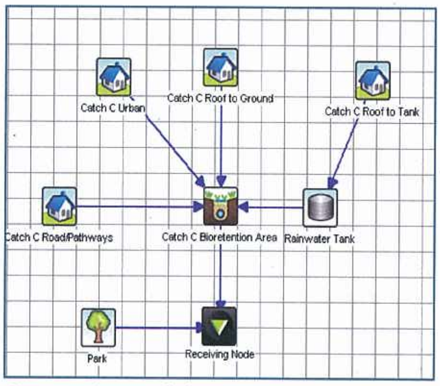


Figure 6 - Music Model Layout Catchment C (west)

7.11 MUSIC RESULTS

The MUSIC pollutant load reductions for the site are detailed in Table 25.

Table 25 – MUSIC Results

		Catch B (east)		Catch C (west)	
Parameter	Required Pollutant reduction (%)	MUSIC Results Achieved	Objective Achieved	MUSIC Results Achieved	Objective Achieved
Total Suspended Solids	85	85.2	Yes	85.2	Yes
Total Phosphorus	70	72.4	Yes	76.2	Yes
Total Nitrogen	45	48.7	Yes	48.7	Yes

Table 25 demonstrates that the load based pollutant objectives for the site have been achieved. The water quality devices proposed for the site are adequate in achieving satisfactory pollutant levels output from the site.

8.0 CONCLUSION

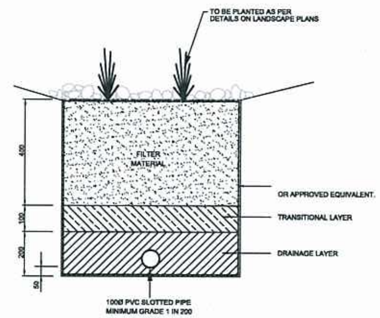
The SBSMP developed by JFP Urban Consultants Pty Ltd indicates significant reductions in post development stormwater runoff pollutant concentrations and peak flow rates discharging to downstream receiving waters.

XP SWMM modelling and rational method analyses of pre and post development catchment areas have enabled retention requirements to be determined. Modelling indicates downstream flood levels are not increased by the proposed development.

MUSIC modelling was undertaken and the results confirmed that the proposed rainwater tanks and bio-filtration areas provide sufficient water quality improvement to meet current Gladstone Regional Council and Healthy Waterways water quality objectives for the site.

LEGEND

- PROPERTY BOUNDARY
- FLOW PATH
- CATCHMENT B (EAST)
- CATCHMENT C (WEST)
- PARK
- BIO RETENTION AREA (INDICATIVE ONLY)
- CONTOURS



BIOFILTER MATERIAL DETAIL
N.T.S.

BIOFILTER MATERIAL GRADING

FILTER MATERIAL SHALL BE GRADED AND CAN BE SILICEOUS OR CALCAREOUS IN ORIGIN. THE FILTER MATERIL SHALL BE EVENLY PLACED AND LIGHTLY COMPACTED. THE FILTER MATERIAL SHALL COMPLY WITH THE FOLLOWING CRITERIA:

MATERIAL:
SANDY LOAM OR APPROVED EQUIVALENT WITH A HYDRAULIC CONDUCTIVITY OF 200mm/hr AND BE FREE FROM RUBBISH AND DETERIOUS MATERIAL.

PARTICLE SIZE DISTRIBUTION:
TYPICAL SOIL COMPOSITION WOULD BE:
CLAY & SILT < 3% (<0.05mm)
VERY FINE SAND 5 - 30% (0.05-0.15mm)
FINE SAND 10%-30% (0.15mm-0.25mm)
MEDIUM TO COARSE SAND 40%-60% (0.25mm-1.0mm)
COARSE SAND 7%-10% (1.0mm-2.0mm)
FINE GRAVEL < 3% (2.0mm-3.4mm)

ORGANIC MATTER CONTENT
LESS THAN 5% (w/w). AN ORGANIC CONTENT OF HIGHER THAN 5% IS LIKELY TO RESULT IN LEACHING OF NUTRIENTS. REFER TO AS4419 - 2003.

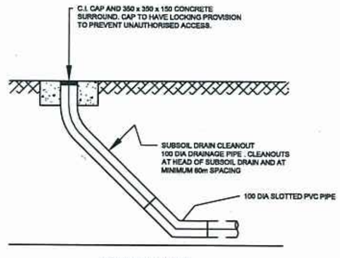
pH
AS SPECIFIED FOR 'NATURAL SOILS AND SOIL BLENDS' 5.5 - 7.5 (pH 1:5 IN WATER). REFER TO AS4419-2003.

ELECTRICAL CONDUCTIVITY
AS SPECIFIED FOR 'NATURAL SOILS AND SOIL BLENDS' < 1.25dS/m. REFER TO AS4419-2003.

PHOSPHORUS
< 100 mg/kg. SOILS WITH PHOSPHORUS CONCENTRATIONS > 100 mg/kg SHOULD BE TESTED FOR POTENTIAL LEACHING. WHERE PLANTS WITH MODERATE PHOSPHORUS SENSITIVITY ARE TO BE USED, PHOSPHORUS CONCENTRATIONS SHOULD BE < 20 mg/kg.

TRANSITIONAL LAYER
SHALL BE CLEAN WELL GRADED SAND OR COURSE SAND MATERIAL CONTAING LITTLE OR NO FINES. THE GRADING OF THE TRANISIOTNAL LAYER SHALL COMPLY WITH THE FOLLOWING CRITERIA:
% PASSING 1.4mm 100%
1.0mm 80%
0.7mm 44%
0.5mm 8.4%
THE TRANSITIONAL LAYER SHALL HAVE A HYDRAULIC CONDUCTIVITY OF 1800mm/hr

DRAINAGE LAYER
SHALL BE CLEAN FINE GRAVEL (2mm - 5mm WASHED SCREENINGS)
MINIMUM HYDRAULIC CONDUCTIVITY OF 3600mm/hr.



BIO-BASIN SUBSOIL CLEANOUT
NOT TO SCALE

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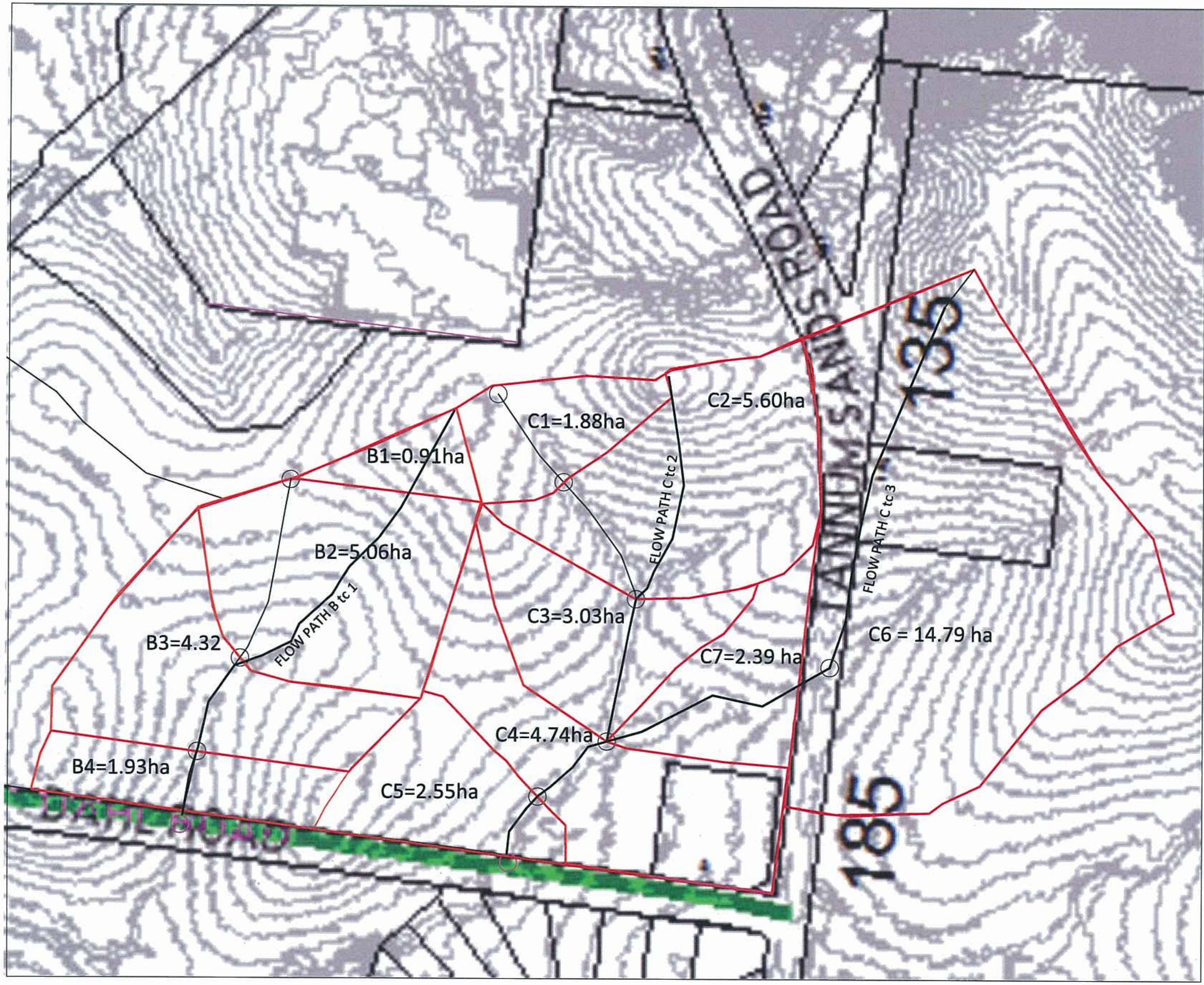
PLANNERS
URBAN DESIGNERS
SURVEYORS
ENGINEERS
LANDSCAPE ARCHITECTS

N

SCALE : 1:2000 @ A3
DATE : FEBRUARY 2012
JOB No. : M2410
REV : A

PROJECT:
TANNUM SANDS
HYDRAULIC ASSESSMENT
STORMWATER QUALITY MANAGEMENT PLAN
FILE NAME: SQMP.DWG



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PLANNERS
URBAN DESIGNERS
SURVEYORS
ENGINEERS
LANDSCAPE ARCHITECTS

N

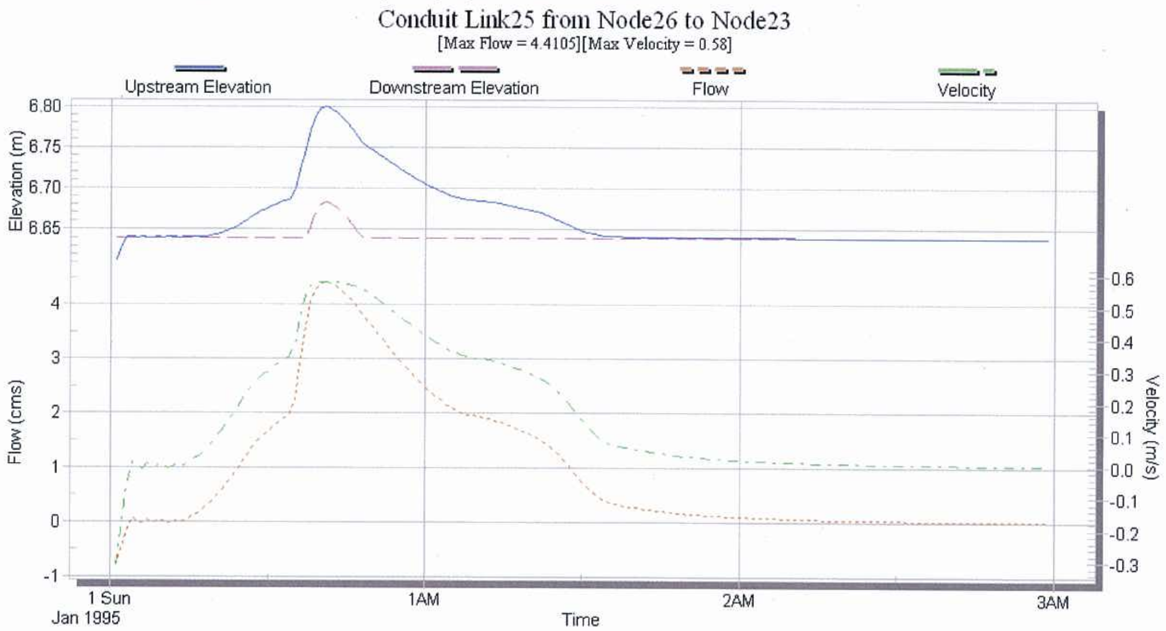
SCALE : 1:4000 @ A3
DATE : FEBRUARY 2012
JOB No. : M2410
REV : A

PROJECT:
TANNUM SANDS
HYDRAULIC ASSESSMENT
CATCHMENT AREAS
FILE NAME: M2410_SBSMP_CATCH_AREAS_REVA.DWG

APPENDIX B – XP SWMM Q100 MODELLING RESULTS FOR RETENTION

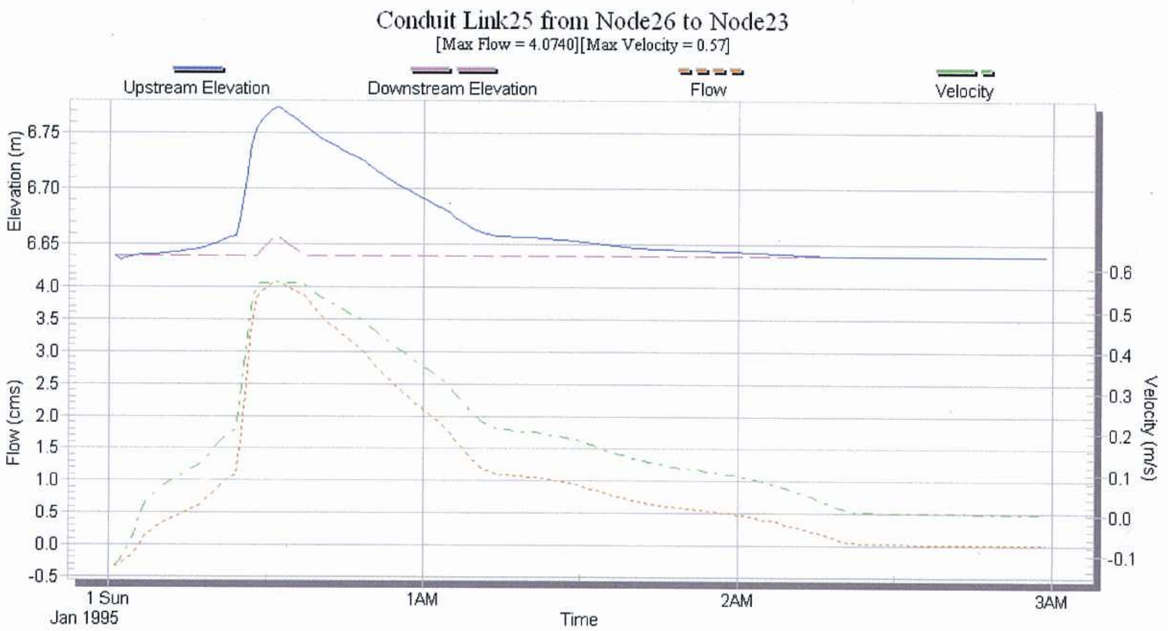
Catchment B

Q100 Pre Development Runoff Hydrograph and Elevation



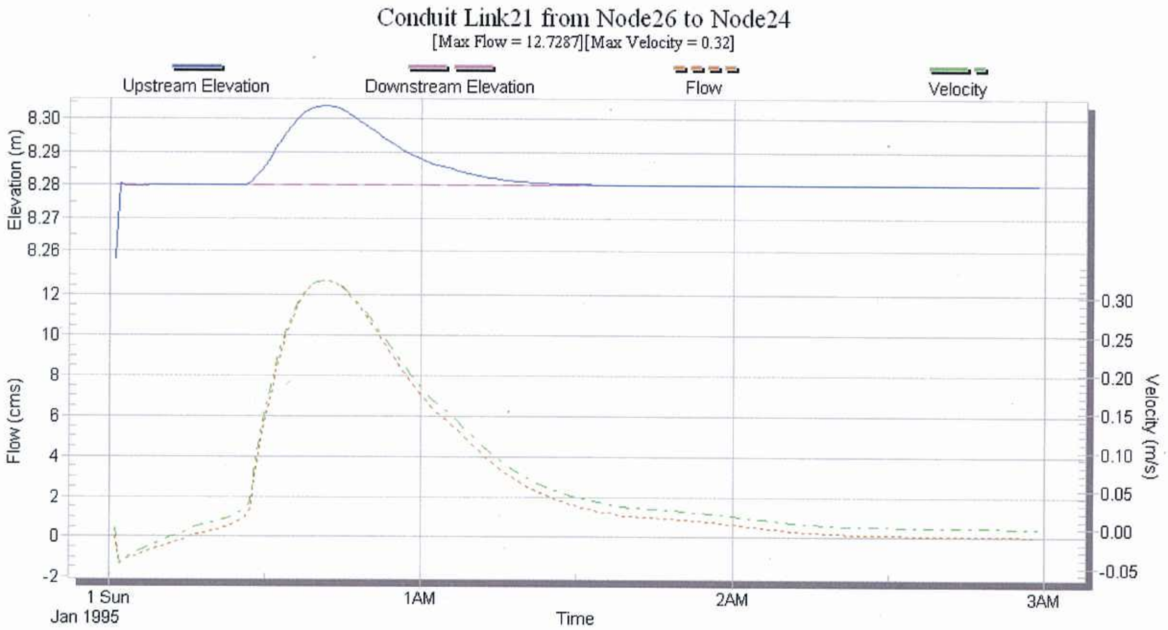
Catchment B

Q100 Post Development Runoff Hydrograph and Elevation



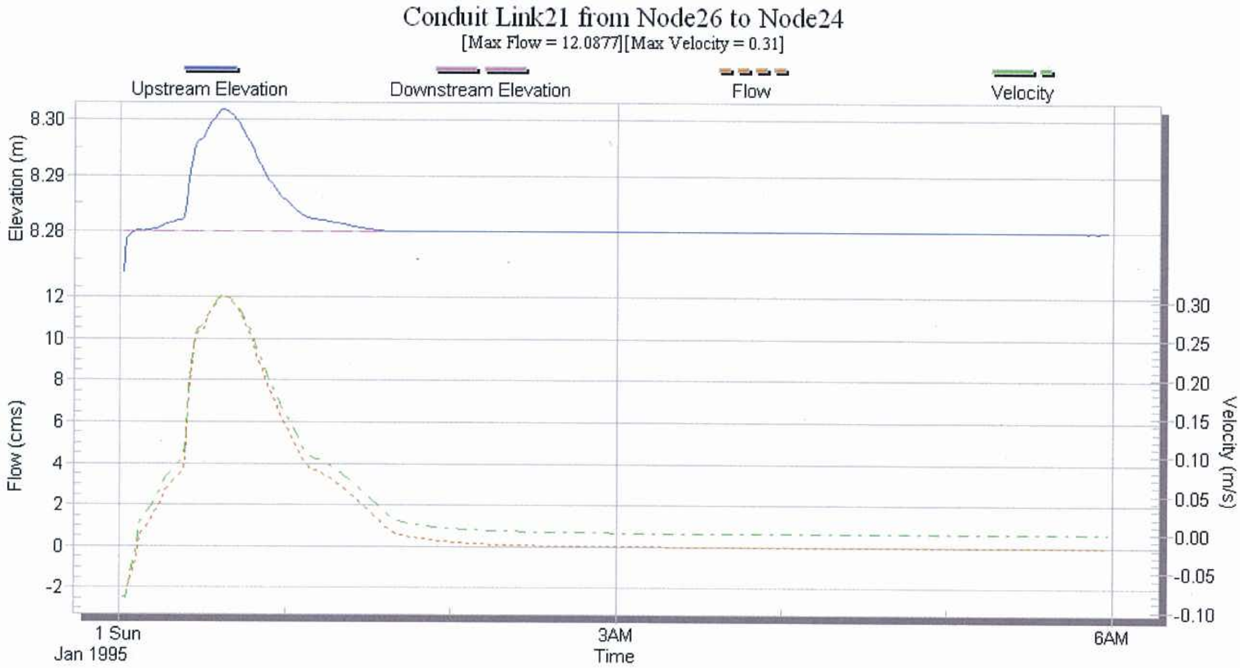
Catchment C

Q100 Pre Development Runoff Hydrograph and Elevation



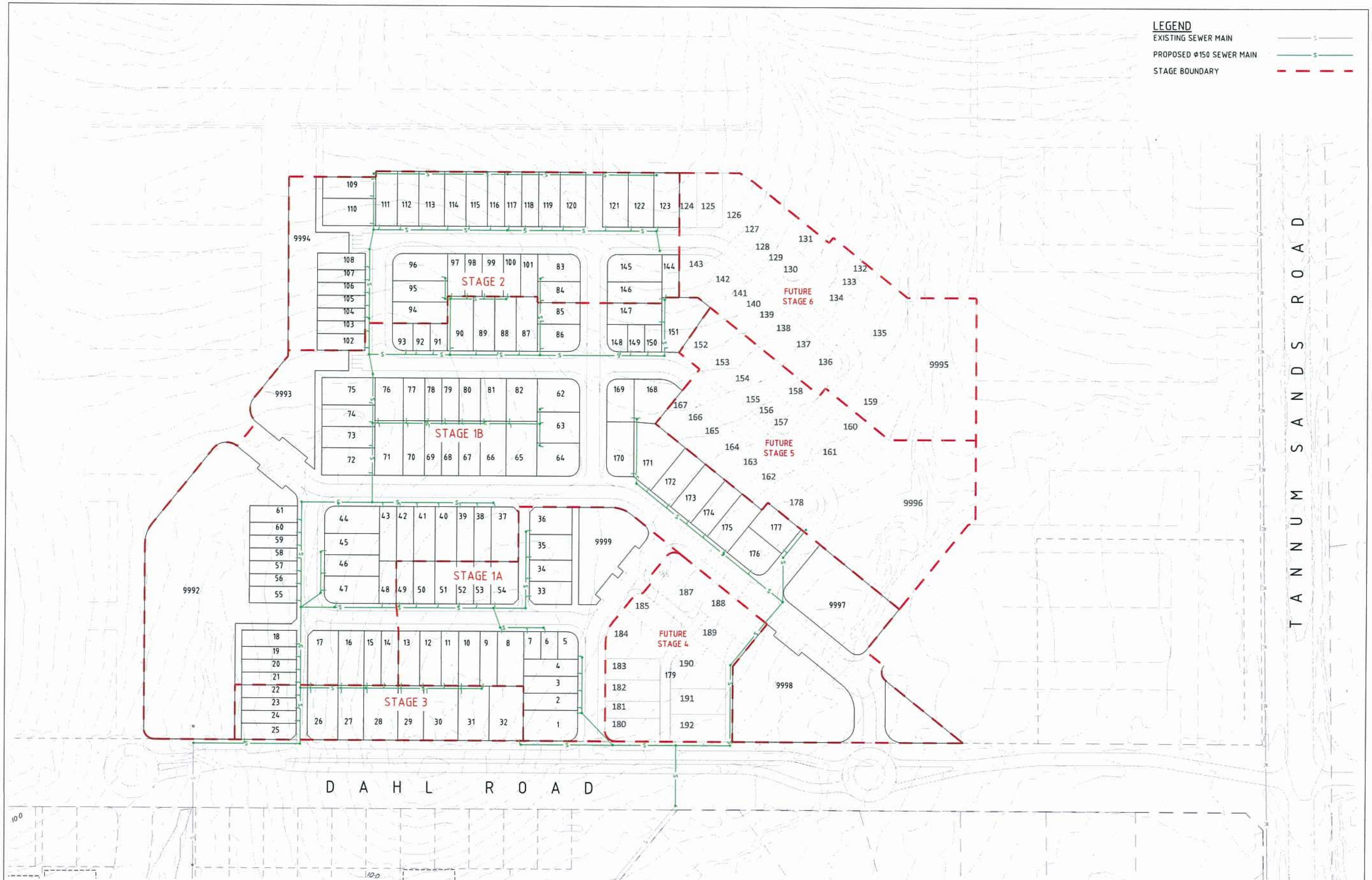
Catchment C

Q100 Post Development Runoff Hydrograph and Elevation



APPENDIX A –SITE PLANS

APPENDIX E – JFP SEWERAGE LAYOUT DRAWINGS



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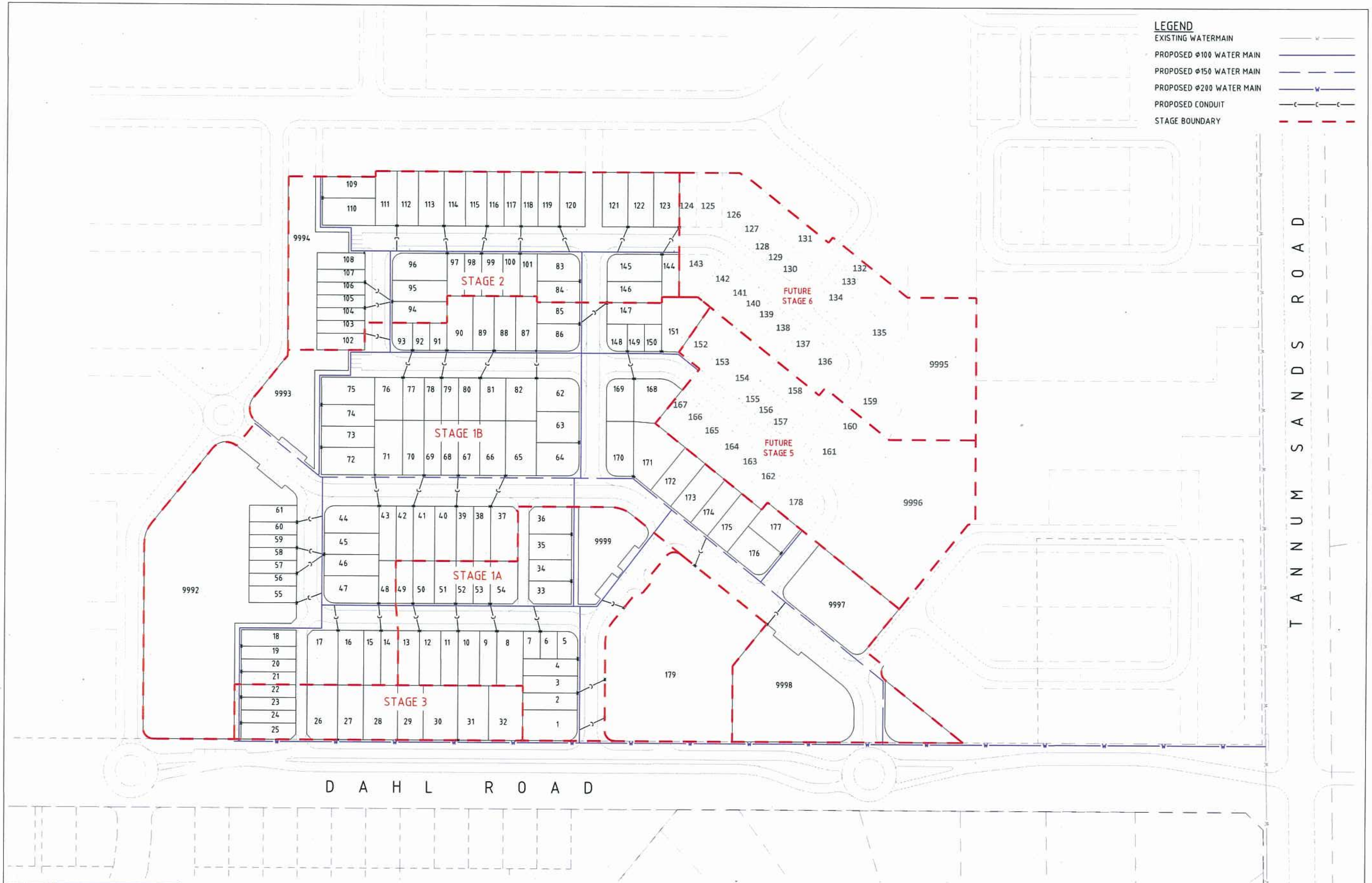
ENGINEERING CONCEPT - PRELIMINARY SEWERAGE LAYOUT

STAGES 1 - 3 TANNUM SANDS UDA
URBAN LAND DEVELOPMENT AUTHORITY

PROJECT:	PLAN:	ISSUE:	NORTH:
M2410	PS01	A	
SCALE:	1:1000 @ A1		
DATE:	FEBRUARY 2012		
FILE NAME:	M2410 PS01A PRELIM SEWER.DWG		

APPENDIX F – JFP WATER SUPPLY LAYOUT DRAWINGS



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ENGINEERING CONCEPT - PRELIMINARY WATER RETICULATION LAYOUT

STAGES 1 - 3 TANNUM SANDS UDA
URBAN LAND DEVELOPMENT AUTHORITY

PROJECT:	PLAN:	ISSUE:	NORTH:
M2410	PW01	A	
SCALE:	1:1000 @ A1		
DATE:	FEBRUARY 2012		
FILE NAME:	M2410 PW01A PRELIM WATER.DWG		

APPENDIX G – JFP PRELIMINARY SEWERAGE ASSESSMENT REPORT

Creativity/PRECISION

JFP
URBANCONSULTANTS

PRELIMINARY SEWERAGE ASSESSMENT

PROPOSED DEVELOPMENT AT
Tannum Sands UDA
FOR
ULDA

PLANNERS
URBAN DESIGNERS
SURVEYORS
ENGINEERS
LANDSCAPE ARCHITECTS

Creativity/PRECISION

JFP URBAN CONSULTANTS

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URBAN DESIGNERS
SURVEYORS
ENGINEERS
LANDSCAPE ARCHITECTS

PRELIMINARY SEWERAGE ASSESSMENT

PROPOSED DEVELOPMENT AT
Tannum Sands UDA
FOR
ULDA

M2410E – Revision E
February 2012

JFP Urban Consultants Pty Ltd
Prepared by: Andrew Fraser
Approved by: Andrew Fraser

TABLE OF CONTENTS

DRAWINGS

1.0 INTRODUCTION

JFP Urban Consultants Pty Ltd (JFP) has been commissioned by ULDA to undertake an investigation into the provision of sewerage for the proposed Tannum Sands UDA.

It is proposed to provide a "regional solution" with the implementation of a major pump station that services the ultimate catchment from ULDA controlled and external developments.

The sewerage assessment has been undertaken to determine sewerage infrastructure required to service the interim and ultimate catchments.

JFP Drawing M2410 PS03 (attached to this assessment) provides indicative interim and ultimate catchments.

2.0 CONSTRUCTION

2.1 CONSTRUCTION ACTIVITIES

The main components of the construction program include:

- Construction of trunk gravity sewer mains (225mm & 300mm dia) upstream of the proposed sewerage pump station
- Construction of a major sewerage pump station and associated access works
- Construction of a sewerage rising main (225mm dia – interim) from the proposed sewerage pump station to the existing Gladstone Regional Council (GRC) Tannum Sands Sewerage Treatment Plant (approximate length 2,695m). The alignment of the proposed sewerage rising main will generally run parallel to the existing GRC 300mm dia sewerage rising main

JFP Drawings M2410 PS01 & M2410 PS02 (attached to this assessment) provide indicative locations of the main components to be constructed.

Indicative timing for these construction activities is provided in Table 1. The actual commencement date for construction is dependent upon the timing of approvals, and as such these dates are subject to change.

Table 1

Construction Activity	Timeframe	Indicative Dates
Construction of trunk gravity sewer mains	1 month	Commence in mid 2012
Construction of major sewerage pump station	1 month	Commence in mid 2012
Construction of sewerage rising main	2 months	Commence in mid 2012

2.2 SITE PREPARATION AND CLEARANCE

At the start of the site preparation phase surveyors will mark the location of the following construction components:

- Trunk gravity sewer main centrelines
- Sewerage pump station location
- Sewerage pump station access centreline
- Sewerage rising main centreline

The width of the construction corridor will be pegged to identify areas of particular significance that need to be avoided or protected during construction.

A qualified ecologist will inspect the site to ensure that the ecological constraints and mitigation measures are correctly implemented.

Vegetation to be retained, areas that have been identified as environmentally sensitive or other areas that require protection from potential damage will be fenced for the duration of the construction period.

A geotechnical engineering consultant with expertise in acid sulfate soils will be commissioned to undertake field testing and reporting to establish the presence and best practice management strategy for works below RL 5m AHD. Works to be undertaken below RL 5m AHD will include sewer trench excavation, sewer pump station well excavation and sewer rising main trench excavation works. The geotechnical engineer commissioned to undertake these works will prepare an Acid Sulfate Management Plan which will incorporate best practice for the identification and management of acid sulfate soils.

Erosion and transportation of sediments off-site will be controlled by the use of temporary sediment basins / barriers. Stockpiles will be located within existing cleared areas and contained within individual sediment barriers / bunds.

All sedimentation and erosion control measures will be carried out in accordance with the Institution of Engineers Australia (Queensland Division) Soil Erosion and Sediment Control – Guidelines for Queensland Construction Sites.

Impacts on the surrounding environment will be minimised during the construction phase with measures outlined in the approved Erosion and Sediment Control Plans and Construction Management Plan to be provided at the operational works design stage.

2.3 LAYDOWN AREAS

Although the majority of construction activities will occur along the route of the sewerage rising main alignment, the establishment of temporary construction compounds and facilities will be required during the construction period. These compounds will be used for temporary storage of construction equipment and materials.

Where practical these laydown areas will be located in previously cleared or disturbed areas.

2.4 CIVIL WORKS

Trenching will be used to construct the sewer trunk gravity and sewerage rising mains. It is expected that an excavator will be used to dig the trench for the majority of the route. In some areas (deeper excavation associated with the sewer trunk gravity main) hard rock may be encountered a hydraulic rock breaking equipment may be required.

Typical trench dimensions for the sewerage rising main will be approximately 0.9m wide and generally 1.3m deep, allowing for a cove depth to top of pipe of approximately 1m. The actual depth profile will be determined during the detailed operational works design phase.

An indicative construction layout is detailed on JFP Drawings M2410 PS01 & M2410 PS02 (attached to this assessment) and provides indicative locations of the main components to be constructed. To manage the integrity of the sewer trunk gravity and sewerage rising mains no trees will be planted on the pipe alignments.

The distance covered daily by trenching will be dependent on terrain, equipment and weather conditions.

Breaks in the trench can be left to facilitate wildlife crossing. In addition methods to prevent fauna entrapment such as trench breakers, ramped ends of trench and fauna ladders may also be implemented.

3.0 DESIGN CRITERIA

3.1 GRAVITY SEWER

Per Capita Flow:

- Average Dry Weather Flow (ADWF) = 225 L/EP/d

Peaking Factor:

- Peak Wet Weather Flow (PWWF) = Greater of 5 x ADWF or C_1 x ADWF
  Where $C_1 = 15 \times (EP)^{-0.1587}$
  Minimum value of C_1 to be 5

Depth of flow at PWWF ≤ 0.75d

3.2 PUMP STATIONS

Pump Operating Storage in Wet Well (m^3) $= \dfrac{(0.9 \times \text{Single Pump Capacity L/s})\ \text{kL}}{N}$

Where N = 12 starts per hour for motors < 15 kW
= 8 starts per hour for motors < 50 kW
= 5 starts per hour for motors > 50 kW

Single Pump Capacity (duty & standby) = PWWF

Pump Station Level Information:

Top of Pump Station	RL 6.500m AHD
Finished Ground Surface	RL 6.300m AHD
Overflow	RL 5.700m AHD
Outgoing Rising Main	RL 4.700m AHD

3.3 RISING MAINS

For planning purposes a maximum velocity of 1.5 m/s was adopted for rising mains during single pump flow (C_1 factored flows). This velocity results in an economic balance between capital and operational costs.

Maximum Velocity ≤ 1.5 m/s

The rising main will be design to achieve a cleansing velocity of 0.75 m/s. To ensure that the cleansing velocities are achieved velocities greater than 1.5 m/s may be required for ultimate flow scenarios.

Minimum Velocity ≥ 0.75 m/s

4.0 DEVELOPMENT POPULATION AND FLOW

The sewerage equivalent person (EP) populations associated with the ULDA and external catchments was determined from allotment yield projections and GRC design standards.

The following occupancy rates were adopted:

Table 2

Use	Basis	Sewerage Demand Density (EP)
Dwelling House	Per Lot	2.8
Retirement Village	Per Unit	1.8

SCENARIO A – INTERIM CATCHMENT

Interim catchment comprises ULDA development (approximately 820 Lots), the Silverton Drive development (44 Lots – 38 to be provided with reticulated sewer) located on the southern side of Dahl Road and the proposed retirement village (158 Units) located on the eastern side of Tannum Sands Road north of the ULDA site.

Table 3A

No. of Lots/Units (ET)	Demand (EP per ET)	Population (EP)	ADWF (L/s)	5 x ADWF (L/s)	C_1	C_1 x ADWF (L/s)	PWWF (L/s)
820	2.8	2,296	5.98	29.90	-	-	29.90
38	2.8	106	0.28	1.40	-	-	1.40
158	1.8	284	0.74	3.70	-	-	3.70
	Total:	2,686	7.00	35.00	4.28	29.96	35.00

Pump Well Diameter, Well Levels and Storage

A 3.2m diameter wet well has been adopted.

The preliminary pump well levels (subject to detail design) are detailed below in accordance with GRC guidelines.

Incoming Sewer IL	0.000m AHD
Alarm Level	-0.150m AHD
Standby Pump Start	-0.450m AHD
Duty Pump Start (interim)	-1.350m AHD
Duty Pump Stop	-2.150m AHD
Pump Floor	-2.550m AHD

Minimum Operational Volume of Wet Well (m^3) $= \dfrac{\text{(0.9 x Single Pump Capacity L/s) kL}}{N}$

$= \dfrac{\text{(0.9 x 35.00 L/s) kL}}{5}$

$= 6.30\ m^3$

Storage height required = $\frac{\text{Volume}}{\pi \times 1.6^2}$

= $\frac{6.30}{\pi \times 1.6^2}$

= 0.78m

Storage volume available is 1.40m (ultimate) – therefore storage volume is ok.

Emergency Storage

Emergency storage has been provided as four (4) hours x ADWF above standby pump start level in wet well.

Assume Surcharge Level (overflow RL)	=	5.700m AHD
Standby Pump Start Level	=	-0.450m AHD
Volume above Standby Pump Start Level	=	$(5.700 + 0.450) \times 1.6^2$
	=	$15.74m^3$

Volume in upstream sewerage reticulation pipes and manholes at ultimate development has not been included in emergency storage calculations.

The available emergency storage time in the system is calculated as follows:

Emergency Storage Time = $\frac{\text{Volume in System}}{\text{ADWF}}$

= $\frac{15.74 \times 1000}{7.00 \times 3600}$

= 0.63 hrs (Interim)

Emergency storage = 4 hrs ADWF + 50% of emergency storage of upstream pump stations (GRC Requirement). For the purposes of this preliminary assessment we have adopted a storage required for four (4) hours reaction time.

Therefore additional storage is required for ultimate scenario.

Storage required for six (6) hours reaction time = $\frac{4hrs \times 7.00L/s \times 3600}{1000}$

= $100.80m^3$

Additional storage required = $100.80m^3 - 15.74m^3$

= $85m^3$

Additional storage to be a well as follows:

Top of well to be 5.700m AHD

Bottom of well to be -0.450m AHD

Diameter of Additional Storage Well (D)
Adopt four (4) additional storage wells

$D = \sqrt[2]{\frac{4 \times V}{\pi \times h}}$ V = $85m^3$
h = 6.150m

D = 4.20m

SCENARIO B – ULTIMATE CATCHMENT

Ultimate catchment comprises ULDA development (approximately 820 Lots), the Rio Tinto Land on the western side of Coronation Drive (approximately 750 Lots), the Silverton Drive development (44 Lots – 38 to be provided with reticulated sewer) located on the southern side of Dahl Road and the proposed retirement village (158 Units) located on the eastern side of Tannum Sands Road north of the ULDA site.

The interim rising main (225mm dia) will require augmentation and additional storage is required for the ultimate scenario.

Table 3B

No. of Lots/Units (ET)	Demand (EP per ET)	Population (EP)	ADWF (L/s)	5 x ADWF (L/s)	C_1	C_1 x ADWF (L/s)	PWWF (L/s)
820	2.8	2,296	5.98	29.90	-	-	29.90
750	2.8	2,100	5.47	27.35	-	-	27.35
38	2.8	106	0.28	1.40	-	-	1.40
158	1.8	284	0.74	3.70	-	-	3.70
	Total:	4,786	12.47	62.35	3.91	48.76	62.35

Pump Well Diameter, Well Levels and Storage

A 3.2m diameter wet well has been adopted.

The preliminary pump well levels (subject to detail design) are detailed below in accordance with GRC guidelines.

Incoming Sewer IL	0.000m AHD
Alarm Level	-0.150m AHD
Standby Pump Start	-0.450m AHD
Duty Pump Start (interim)	-1.350m AHD
Duty Pump Stop	-2.150m AHD
Pump Floor	-2.550m AHD

Minimum Operational Volume of Wet Well (m^3) $= \frac{(0.9 \times \text{Single Pump Capacity L/s})\ \text{kL}}{N}$

$= \underline{(0.9 \times 62.35\ \text{L/s})\ \text{kL}}$

5

	=	$11.22\ m^3$
Storage height required	=	$\frac{\text{Volume}}{\pi \times 1.6^2}$
	=	$\frac{11.22}{\pi \times 1.6^2}$
	=	1.40m

Storage volume available is 1.40m (ultimate) – therefore storage volume is ok.

Emergency Storage

Emergency storage has been provided as six (6) hours x ADWF above standby pump start level in wet well.

Assume Surcharge Level (overflow RL)	=	5.700m AHD
Standby Pump Start Level	=	-0.450m AHD
Volume above Standby Pump Start Level	=	$(5.700 + 0.450) \times 1.6^2$
	=	$15.74m^3$

Volume in upstream sewerage reticulation pipes and manholes at ultimate development has not been included in emergency storage calculations.

The available emergency storage time in the pump station is calculated as follows:

Emergency Storage Time	=	$\frac{\text{Volume in Pump Station}}{\text{ADWF}}$
	=	$\frac{15.74 \times 1000}{12.47 \times 3600}$
	=	0.35 hrs (Ultimate)

Emergency storage = 4 hrs ADWF + 50% of emergency storage of upstream pump stations (GRC Requirement). For the purposes of this preliminary assessment we have adopted a storage required for six (6) hours reaction time.

Therefore additional storage is required for ultimate scenario.

Storage required for six (6) hours reaction time	=	$\frac{6hrs \times 12.47L/s \times 3600}{1000}$
	=	$269.35m^3$
Additional storage required	=	$269.35m^3 - 15.74m^3$
	=	$254m^3$

Additional storage to be multiple wells as follows:

Top of well to be 5.700m AHD
Bottom of well to be -0.450m AHD

Diameter of Additional Storage Wells (D)
Adopt three (3) additional storage wells

$$D = \sqrt[2]{\frac{4 \times V}{\pi \times h}}$$

V	=	$254m^3 / 3 = 85m^3$
h	=	6.150m

D = 4.20m (3 Storage Wells)