NORTH TERRACE DRAINAGE DETAILED DESIGN

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DETAILED DESIGN STUDY

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1. Executive Summary

This report presents the Detailed Design study for the North Terrace Drainage Upgrade, located between Hackney Road and College Road, Kent Town. Hydro-Future Consulting has conducted a thorough feasibility study on the project area and has identified one solution utilising a combination of traditional stormwater management methods and water sensitive urban design (WSUD).

The main purpose of the detailed design is to alleviate significant flooding to the project area after heavy rainfall events. It aims to improve the quality of the stormwater entering the receiving environment, provide improved visual amenity and provide water harvesting options as well as being cost effective.

This detailed design study presents an analysis and evaluation for the effective flood mitigation of flood impact at North Terrace. Information within the detailed design study provides specific details on how Hydro-Future plans to complete the North Terrace Drainage Design Upgrade through a combination of WSUD technologies.

The fundamental components of the detailed design study included:

1. Conventional Stormwater Management Design
2. Gross Pollutant Trap Design
3. Bio-Retention Basin
4. Water Harvesting
5. Environmental Management Plan
6. Traffic Management Plan
7. Protect Culture and Heritage
8. Social acceptability
9. Community consultation
10. Retaining wall design
11. Structural analysis of a sandstone arch culvert
12. Safety
13. Construction and operation Issues
14. Detailed costing of the entire project
15. Detailed drawings of relevant aspects of the project
16. Detailed recreational amenities
17. Technical specifications

18. Construction of a project website

The majority of analysis and design covered throughout this project includes hand calculations with the aid of various engineering software including DRAINS, MUSIC, AutoCAD, STRAND 7 and others. All calculations and methodologies can be found in the appendices of this report. The results for the Water design have been summarised under section 4, DRAINS output is used to determine the required pipe sizes for the new stormwater drainage system which consists of 17 sub-catchments. The final drainage pipe sizes are 750mm and 600mm. The bio-retention system is designed to improve the water quality of the stormwater runoff and 21 rainwater tanks have been designed for water harvesting.

The total cost of the proposed design will be $1.2 Million.

2. Introduction

The City of Norwood Payneham and St. Peters is a metropolitan council, covering an area of 15.1km2, east of Adelaide's CBD. One of the primary services that the council provides for the 34,000 residents is the stormwater drainage network. The drainage network allows for the effective collection of surface water in the area and provides flood protection throughout the city. The majority of the system comprises stormwater pipes, pits, junction boxes and culverts, the stormwater makes its way to First Creek, the River Torrens and ultimately Gulf St Vincent.

North Terrace Kent Town has suffered significant flooding from College Road through to the Royal Hotel. The council would like to develop a stormwater solution to resolve these flooding events and future proof the existing system against any heavy rain events that may occur. The new solution aims to include water sensitive urban design (WSUD) technologies, be cost effective and to improve the quality of the water before it exits the system into First Creek.

North Terrace is a major arterial road on the outskirts of the CBD and has for many years experienced flooding issues as a result of major storm events, particularly between Hackney Road and College Road. The existing infrastructure has proven to be inefficient in providing quality flood mitigation along North Terrace and as a result requires an update in terms of new infrastructure and drainage solution options. These options will be presented in this detailed design study and cover detailed evaluation to mitigate current flood impact.

3. Deliverables

Hydro-Future Consulting has delivered the following items both within and with adjoined documentation for the detailed design:

- Detail Design Report
- Quality Management System manual
- Environmental Management Plan
- Technical Specifications
- Work, Health and Safety including Task Specific Safety Assessments
- Engineering CAD Drawings
- Bill of Quantities for each department
- Management Review Report

4. Water

4.1. Flood mitigation

4.1.1. Storm Water System Design

To prevent the flooding conditions of North Terrace, Kent Town a new stormwater drainage system along with water harvesting options and some water quality improvement methods have been suggested by Hydro-Future in the feasibility study. These combined design options will be carried out in the detail design stage. With the combined option, a new stormwater drainage system will be installed and will be the most highlighted design solution. It will be designed to contain the runoff from the total catchment area and will be designed for a worst case scenario of 1 in 20 year rain fall event.

4.1.2. Design Flow Determination

The total runoff of the catchment area was calculated using the equation below to determine the total flow rate of the storm water system to design the system.

$$Q = \frac{CIA}{360}$$

Where Q =the design flow rate (m^3/s)

C=Runoff coefficient

I= the rainfall intensity (mm/hr)

A= the catchment area (ha)

Therefore to calculate the design flow, above parameters were calculated first.

4.1.3. Catchment area and Sub-Catchments

The total project catchment area of 8.085ha was divided into sub-catchments, considering the best locations for the pits and the properties which were selected for the rain water tank implementation. Figure 1 shows the total catchment area which is contributing to the flooding condition within North Terrace and Figure 2 presents the divided sub-catchment areas. The total catchment area was divided into 17 sub-catchments for the design of the stormwater system. Runoff from each sub-catchment will be entering the stormwater system via side entry pits along North Terrace.

Figure 1: Catchment Area for the Project (Tonkin Consulting, 2015)

Figure 2:Sub-catchment Areas for the project. (Google Maps, 2015)

The areas of the different sub-catchments were measured using the Google area calculator tool (Daft Logic 2015) and the selected roof areas which can be used for water harvesting were removed from the final catchment area as it does not contribute to the total runoff. Table 1 shows these measured areas.

Table 1: Total Sub-Catchment Areas

Sub-Catchment	Residential Area (ha)	Road Area (ha)	Removed Roof Area (ha)	Final Residential Area (ha)	Total Area (ha)
1	0.160	0.040	0.000	0.160	0.200
2	0.036	0.105	0.085	0.380	0.089
3	0.540	0.180	0.140	0.400	0.580
4	0.570	0.130	0.030	0.540	0.670
5	0.200	0.090	0.065	0.226	0.451
6	0.240	0.070	0.045	0.195	0.265
7	0.180	0.060	0.081	0.099	0.159
8	0.200	0.080	0	0.200	0.280
9	0.470	0.090	0	0.470	0.560
10	0.280	0.110	0	0.280	0.390
11	0.200	0.050	0	0.200	0.250
12	0.480	0.130	0	0.480	0.620
13	1.110	0.100	0.163	0.948	1.048
14	0.370	0.175	0.078	0.292	0.467
15	0.410	0.150	0.049	0.361	0.511
16	0.036	0.105	0.085	0.380	0.089
17	0.200	0.090	0.065	0.226	0.053
Total Areas	**6.33**	**1.755**	**0.416**	**5.446**	**8.085**

4.1.4. Determining the Pervious and Impervious Area

After the total sub-catchment areas were determined, the total impervious (paved) and pervious (Green fill) areas for each sub-catchment were determined using Google images. The total impervious area and pervious areas were determined as 90% and 10% respectively for most of the sub-catchments, as most of the catchment areas consist of a similar distribution of pervious and impervious areas. Only sub-catchment number 13, near Osborne Road observed consist of around 80% paved and 20% pervious area according to site investigations. Therefore for all sub-catchments 90% of impervious area were assumed except sub-catchment 13 was assumed to be 80% impervious.

The total impervious area for each sub-catchment is calculated using the following equation with the figures from Table 2: Detailed calculations are included in Appendix 1 section 1.1.1.

$$Total\ \%\ Impervious\ Area = \frac{[(0.9)(Residential\ Area) + (Road\ Area)]}{Total\ Area} * 100$$

Table 2: Total Paved and Pervious Areas

Sub-Catchment	Total % Paved Area	Total % Pervious Area
1	92.0	8.0
2	92.8	7.2
3	93.1	6.9
4	91.9	8.1
5	94.0	6.0
6	92.6	7.4
7	93.8	6.2
8	92.9	7.1
9	91.6	8.4
10	92.8	7.2
11	92.0	8.0
12	90.6	9.4
13	81.9	18.1
14	93.7	6.3
15	92.9	7.1
16	92.8	7.2
17	94.0	6.0

4.1.5. Time of Concentration

The most common practice used in Australia to find travel time is to use the charts offered in Argue (1986). To use these charts, the average slope of the catchment surface is required. As we were not provided with a contour map, this method for travel time calculation was dismissed. Likewise other methods that require the average slope of the surface such as the Kinematic Wave Equation and the Bransby William Equation were also dismissed. (Ahammed 2014)

Though it was possible for us to visit site to find the RL levels physically but due to time restrictions, it was ruled out. This option was also not practicable due to the project area being made up of residential and commercial properties, meaning catchment areas would be hard to gain access to.

Due to the lack of information regarding the RL levels of the project site, runoff calculations have been made using the Probabilistic Rational Method. Using the equation below, the travel times for each sub-catchments were calculated.

$$t_c = 0.76 \times A^{0.38}$$

Where;

A = Area of the Sub-Catchment with roof areas (km^2)

$t_c = Time\ of\ Concentration$ (hr)

The longest distance within the sub-catchment was considered without excluding the roof areas which contribute to the rainwater harvesting. Once the travel time was obtained from the equation, standard 5minute of gutter to pit time was added to obtain the complete travel time for the runoff. The calculated travel time for all 17 Sub-Catchments are included in Table 90.

Example Calculation for Sub-Catchment 1;

$$t_c = 0.76 \times A^{0.38}$$

$$t_c = 0.76 \times (Residential\ area)^{0.38}$$

$$t_c = (0.76 \times 0.0016^{0.38}) \times 60$$

$$t_c = 3.9 \text{ Minutes}$$

Total Travel time for the Sub-Catchment 1 = 3.9 +5 = 9 Minutes

4.1.6. Rainfall Intensity

The travel time or time of concentration (tc) is defined as the longest duration for water to flow out from the catchment outlet (Argue, 1986). The travel time for each sub-catchment is defined as the critical storm duration and ultimately plays a major role in calculating the rain fall intensity. According to the calculated travel times, the intensity frequency duration (IFD) for each sub catchment was measured using IFD graph from Bureau of Meteorology data which is included in Appendix A.1.1.3. The stormwater system was designed for a 1 in 20 year rain event as it is the worst case event according to client's requirements as mentioned below.

The client requires that project be design to the following criteria:

- Local / Arterial roadway – 1 in 5 year Average Recurrence Interval standard,
- Trapped low point in roadway – 1 in 20 year Average Recurrence Interval standard,
- Creeks – 1 in 100 year Average Recurrence Interval standard.
- the North Terrace low point adjacent to First Creek shall be designed to an ARI of 1 in 20 years

4.1.7. Runoff Coefficient

The run-off coefficient (C10) for the pervious and paved area within the catchment is calculated using the information from Argue (1986). According to Argue for road ways and roofs (Paved areas) the runoff coefficient is assumed to be C10 = 0.9 while for residential land use C10 = 0.1 is assumed for Southern Australian region as shown in Figure 242, Appendix A section 0. The aforementioned runoff coefficient has to be multiplied by a frequency conversion factor Fy, as the design ARI is higher than 10 years. (Argue, 1986). The frequency factors are included in Figure 241, Appendix A section 0.

$$C_y = F_y * C_{10}$$

$$C_{20} = F_{20} * C_{10}$$

Using designated values from Figure 242 and Figure 241, the runoff coefficients for the pervious and impervious areas within the whole sub-catchments were calculated. As both the pervious and impervious area contributes to the runoff, the weighted runoff coefficient for the total runoff of the catchment was calculated using the equation mentioned below.

$$C = \frac{C_i\, A_i + C_p\, A_p}{\sum A}$$

Example calculation for Sub-Catchment 1;

Run-off coefficient for paved area:

$$C_{20} = 1.05 * 0.9 = 0.945$$

Run-off coefficient for pervious area:

$$C_{20} = 1.05 * 0.1 = 0.105$$

Using the calculated Runoff coefficient values for both pervious and impervious area the weighted runoff coefficient for sub-catchment 1 was calculated.

The Runoff coefficient for the sub-catchment 1:

$$C_1 = \frac{C1A1 + C2A2}{\sum A}$$

$$C_1 = \frac{(92.6 * 0.945) + (7.4 * 0.105)}{100} = 0.878$$

4.1.8. Runoff / Design Flow (Q)

Based on the previously mentioned data, the flow rates for all sub-catchments were calculated. Therefore, the total design runoff flow rate for each sub catchment and the total runoff for the total catchment area have been calculated as below example using Excel spread sheets. Obtained results are included in

Table 3.

Example calculation for sub-catchment 1;

$$Q = \frac{CIA}{360}$$

$$Q = \frac{0.878 \times 95\left(\frac{mm}{hr}\right) \times 0.2(ha)}{360}$$

$$Q = 0.0463\ m^3/s$$

Table 3: Flow Rates of each Sub-Catchment

Sub-Catchment	Flow rate (1:20)
1	0.046
2	0.089
3	0.136
4	0.155
5	0.053
6	0.058
7	0.038
8	0.065
9	0.116
10	0.086
11	0.058
12	0.125
13	0.185
14	0.104
15	0.107
16	0.085
17	0.053
Total Flow rate	**1.56 m^3/s**

4.1.9. DRAINS Model

4.1.9.1. DRAINS Model Considerations

Analysis of the stormwater system upgrade in DRAINS involved both the use of the rational method and the ILSAX method. The rational method assumes that the recurrence interval of the peak discharge is equal to that of the rainfall intensity and assumes that there is no loss of runoff in the system. The ILSAX method on the other hand considers losses from rainfall that don't contribute to the runoff, losses occur due to depression storages and runoff lost over pervious surfaces (Ahammed 2014). For these reasons the ILSAX model is taken as the most accurate but least conservative of the models. Results of the DRAINS analysis can be found in Appendix 1 section 1.2.

4.1.9.2. Design Parameters

4.1.9.3. Rainfall Data / Storm Duration

In the analysis of our stormwater system we looked at large number of different rainfall events. We considered ARI's of 1, 5, 10, 20 and 50 years and looked at storm durations of 5, 10, 30 mins, and 1 and 2 hours. In total there 26 storms used in the analysis. For our analysis we are using a 1,5,10 year ARI as the minor storm consideration and a 20 and 50 year ARI for the major storm consideration. Designing for both of these ARI's will be a conservative approach and will ensure that the storm water system is more than adequate in the future.

4.1.9.4. Runoff Travel Time and coefficient

Due to lack of information regarding the RL levels of the project site, runoff calculations have been made using the Probabilistic Rational Method.

$$t_c = 0.76 \times A^{0.38}$$

Flow calculations using the Probabilistic Rational Method can be found in section 4.1.5.

4.1.9.5. Runoff Coefficient

Runoff coefficient used in the design calculations were taken from Table 5.3 of Argue (1986) as shown in Figure 242 in Appendix 1.1.4. Then the weighted runoff coefficient were calculated and included in

Table 92 Appendix 1 section 1.1.4.

4.1.9.6. Invert Level Calculations

Invert level calculations of the pipes were used using the following formula (Ahammed, 2014)

$$IL = FSL - Pipe\ Diameter - Cover$$

The FSL levels were taken from Tonkin Consulting's long section drawing, filename 20101355_SWPLAN. From this drawing, FSL's were known from First Creek to just past College Road (Chainage 320.00). As the length of the pipe extends to a length of 720m the FSL's up to this point were needed. Due to the lack of information regarding the levels in the project area projected FSL's from Chainage 320 to Chainage 720 were produced using Microsoft Excel to reflect the steady rise of the road over this span (Ahammed, 2015).

As shown in

Table 4, the length of the system has been split into a number of segments that are determined from the distance between pits. In order to meet the slope and velocity requirements the pipe cover was changed from Chainage 0.00 to Chainage 50.00. According to WSA standards it should also be noted that slope requirements of greater than 0.005 and velocity requirements of greater than $1m^2/s$ but less than $4m^2/s$ have been achieved.

To check the velocity requirements the following formula was used:

$$Velocity = \frac{1}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

Where: n = Manning's roughness coefficient (For concrete 0.015)

R = Hydraulic Radius $\left(\frac{Pipe\ Diameter}{4}\right)$

S = slope $\left(\frac{Finish\ IL - Starting\ IL}{Length\ of\ Pipe}\right)$

The calculations for the segment of First Creek to Pit 1 have been shown as an example to demonstrate how the hydraulic radius, slope, and velocity were calculated.

R = Hydraulic Radius $\left(\frac{Pipe\ Diameter}{4}\right)$ = $\left(\frac{0.75}{4}\right)$ = 0.1875

$$S = \text{slope}\left(\frac{Finish\ IL - Starting\ IL}{Length\ of\ Pipe}\right) = \left(\frac{34.62 - 34.4}{20}\right) = 0.01$$

$$Velocity\ = \frac{1}{n} \times\ R^{\frac{2}{3}}\ \times S^{\frac{1}{2}} = \frac{1}{0.015} \times\ 0.1875^{\frac{2}{3}}\ \times 0.01^{\frac{1}{2}} = 2.29 m^2/s$$

For the remainder of the calculations Microsoft Excel was used, final IL design levels, slope and velocity checks, and pipe lengths can be found in

Table 4.

Table 4: Pipe Design Details

Segment	Chainage (m)	IL's at starting point (m)	Length (m)	Slope	IL's at finishing point (m)	Check the Velocity (m^2/s)
First creek to Pit 1	0-20	34.4	20	0.01	34.62	2.29
Pit1-Pit2	20-50	34.62	30	0.02	35.15	2.90
Pit2-Pit3	50-150	35.15	100	0.01	36.061	1.80
Pit3-Pit4	150-245	36.06	95	0.02	38.071	2.74
Pit4-Pit16	245-305	38.07	60	0.02	39.42	2.82
Pit16-Pit5	305-360	39.42	55	0.02	40.65	2.81
Pit5-Pit6	360-395	40.65	35	0.02	41.256	2.48
Pit6-Pit7	395-475	41.26	80	0.01	42.4	2.25
Pit7-Pit17	475-555	42.40	80	0.01	43.544	2.25
Pit17-Pit8	555-625	43.54	70	0.01	44.545	2.25
Pit8-Pit9	625-720	44.55	95	0.01	45.832	2.19

4.1.9.7. Pipe Type & Details

All stormwater pipes used in the upgrade are to be made from concrete. In the DRAINS analysis the pipe type used was concrete, under roads. This can be seen in Figure 251. From Chainage 0, First Creek until Chainage 245.00, Pit 4 the concrete pipe has a diameter of 750mm. The remaining pipes used in the system are 600mm in diameter; this includes pipes joining in a 60 degree angle to pits to the main water pipe. The existing pipe diameter assumed to be 675mm.

4.1.9.8. Pit Locations & Types

Engineering Drawings HF-103 shows where pit locations can be found. This drawing shows both the new and existing pits in the area. The existing pits are located at the bottom and Northern side of the road. Typically, all pits used in the project belong to the City of Adelaide Pits, 3% cross-fall, 1% grade family and were Adelaide Single pit size. (Ahammed 2014)

Due to the slope of North Terrace, majority of new pits in the project area are on grade pits, Pit 1 and Pit 2 (Figure 3) on the other hand are sag pits as they are located at the bottom of the hill where flood mitigation is most critical due to trapped low point. Refer to Appendix A section 1.2 which shows the drains input and parameters used in the analysis.

Design IL for the pits was calculated by taking 0.2m off from the pipe invert level at the positions that they occur.

For example, the design IL for Pit 1 = 34.62 - 0.2 = 34.420m

Design IL for all Pits can be seen in Engineering drawing HF-101 and HF-102 and also in Table 5.

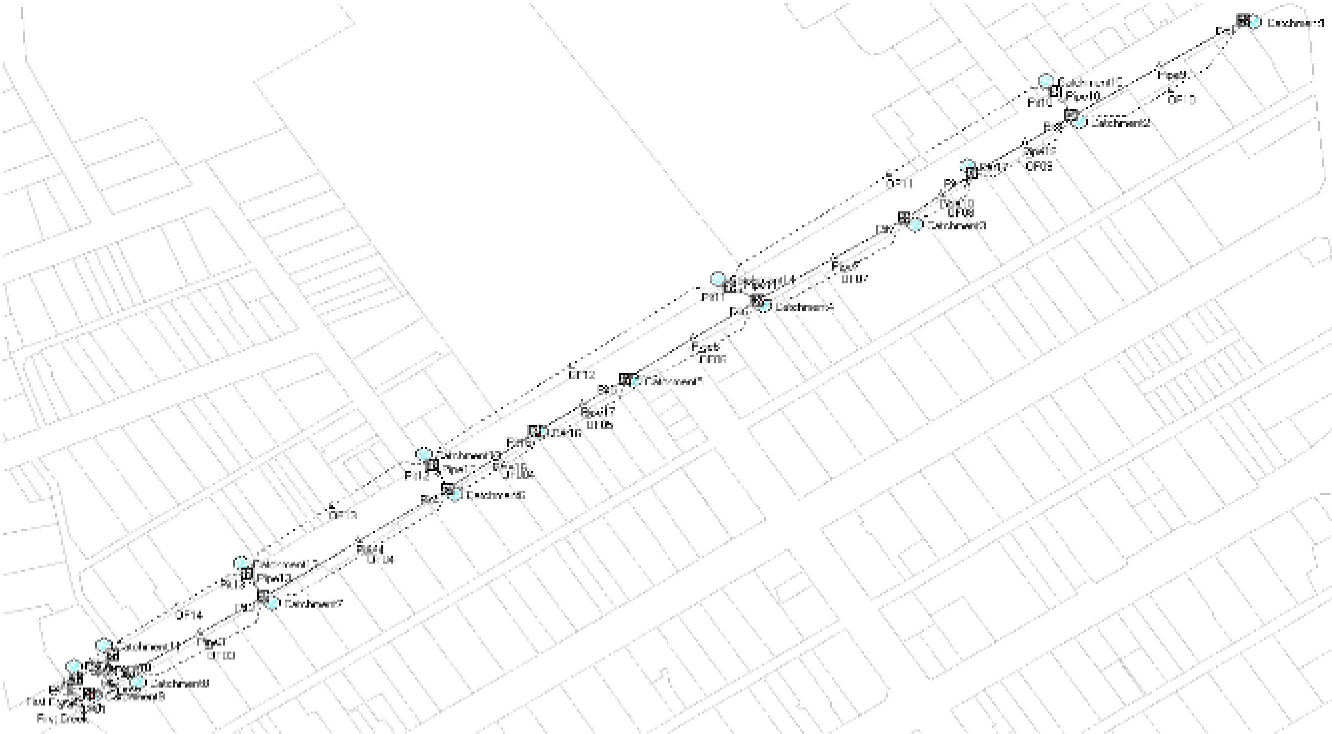
Table 5: Pit design Invert Levels

Pit No.	IL (m)
1	34.420
2	34.950
3,13	35.861
4,12	37.871
16	39.220
5	40.450
6,11	41.056
7	42.200
17	43.344
8,10	44.345
9	45.632

4.1.9.9. Overflow Route Details

Overflow routes between all pits need to be considered and have been included in the drains analysis. In order to produce the most accurate results for our system, overflow routes are critical to mimic the overflow of water between pits as it travels down the length of the road.

Typical inputs and details in regards to overflow routes can be seen in Figure 253. Travel time considered between pits is varied and was dependent on how far the water has to travel. Where



pits are close together a travel time of 0.5 mins was considered whereas large distances such as where the maximum length of 100m occurs travel time of 2 mins was used.

4.1.10. Methodology for DRAINS Analysis

The first step of the DRAINS analysis was to set up the model. As we are using both the rational and ILSAX model, the design properties for the two different models were included. The design procedure is included in Appendix 1.1.2. Blocking factors for the pits were considered in the design process. (On-grade pits – 0.2, Sag Pits -0.5).

Figure 3: Completed DRAINS model outline

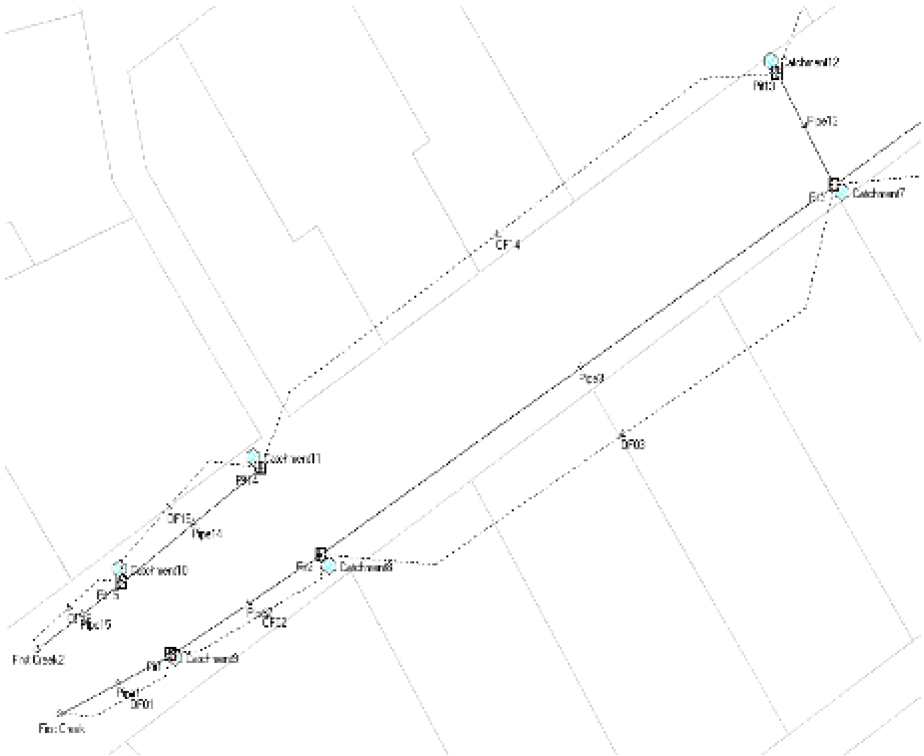


Figure 4 - Close Up of Model at First Creek

In our analysis we considered a total of 26 storm events. As we were designing up to an ARI of 50 years this would be a worst case scenario and was used as the major storm. The minor storm scenario was selected for 20 year ARI. Screenshots on selecting these storm events are shown below in Figure 5nd Figure 6.

4.1.11. DRAINS Analysis Output

The DRAINS analysis outputs for both minor and major storm events have been shown below.

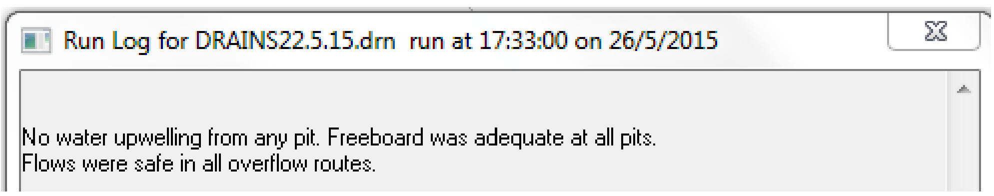


Figure 5- Run Log for Minor Storm Analysis

As shown in Figure 5 no errors occurred when the system was exposed to a 1 in 20 year ARI.

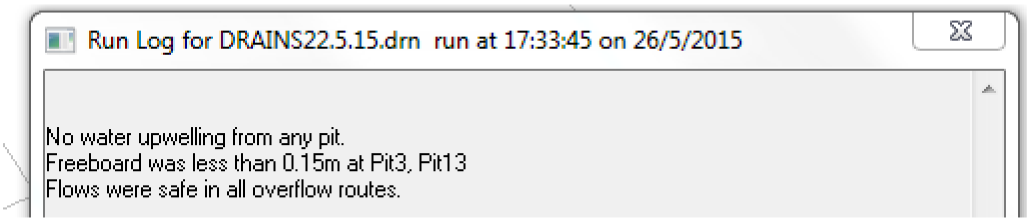


Figure 6- Run Log for Major Storm Analysis

As shown in Figure 6, freeboard less than 0.15m occurred when exposed to a 50 year ARI rainfall event. Freeboard means that when exposed to a worst case 50 year ARI storm, Pit3 & Pit 13 will fill, leaving less than 0.15m for the water to rise before upwelling occurs. A 50 year ARI storm event is extremely unlikely so extra design of the system to remove this problem would be uneconomical and result in an over engineering of the suggested storm water system.

As per client requirements design to a 1 in 5 year ARI is sufficient for arterial roads. However, a 1:20 year ARI should be considered for trapped low points that occur in the roadway. The flooding scenario of the project area occurs as a result of a trapped low point at the bottom of North Terrace, therefore the minor storm consideration of 1:20 year ARI is the most critical. As our modelling system meets the criteria for a 1:20 year ARI it's deemed to be satisfactory.

Exported Long sections views of both minor and major storm analysis can be seen in Engineering Drawings HF-101 and HF-102.

4.1.12. Service Checking

4.1.12.1. Existing Services

The main purpose of the service analysis is to confirm that the new stormwater drainage system will have no conflicts with the existing services and that it will satisfy the standard, specified clearances. The services associated with the project that are distributed along North Terrace are gas pipes, telecommunication cables, water main piping, existing stormwater pipes and the sewerage pipes. Clearances were obtained using SA Water and other official standards to get accurate measurements with regards to the location. A full scale cross section drawing showing the existing services can be seen in Figure 257 and

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Figure 258.

The project area spans through the whole length of the North Terrace covering 720m in length. The chainages and the surface levels were obtained from the provided long section drawings by Tonkin Consulting which is included in Appendix 1 section 1.1.2. (Tonkin consulting, 2011). However, the long sections only provide information from chainage 0 to 140 m and chainage 280 to 382 m along the North Terrace from First Creek up to College Road. Therefore using the provided information, the missing surface levels were back generated. Hydro-Future also recognised the lack of some information regarding the services within the project area, and at this stage it is assumed that the services that are between the arch culvert and college road are the same and runs throughout the whole project area up to Fullarton Road. However, a full investigation will be undertaken before construction and a site survey will be conducted to get the exact depth of the services.

The width of the North Terrace paved road area is 20 meters, which consists of northern and southern sides with two lanes each. Each road side occupies a footpath which is 3 meters in width and the rest 14 meters consists of four lanes which are 3.5 m each as shown in the provided AutoCAD drawing by Tonkin Consulting (Figure 257 and

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Figure 258)

Information regarding existing services locations was obtained from the Dial before You Dig (DBYD) information. The information received was highly detailed in some sections and poorly detailed in others. For example the gas pipes have their depth and boundaries specified but in the telecommunication and other section it does not include it (APA, 2010).

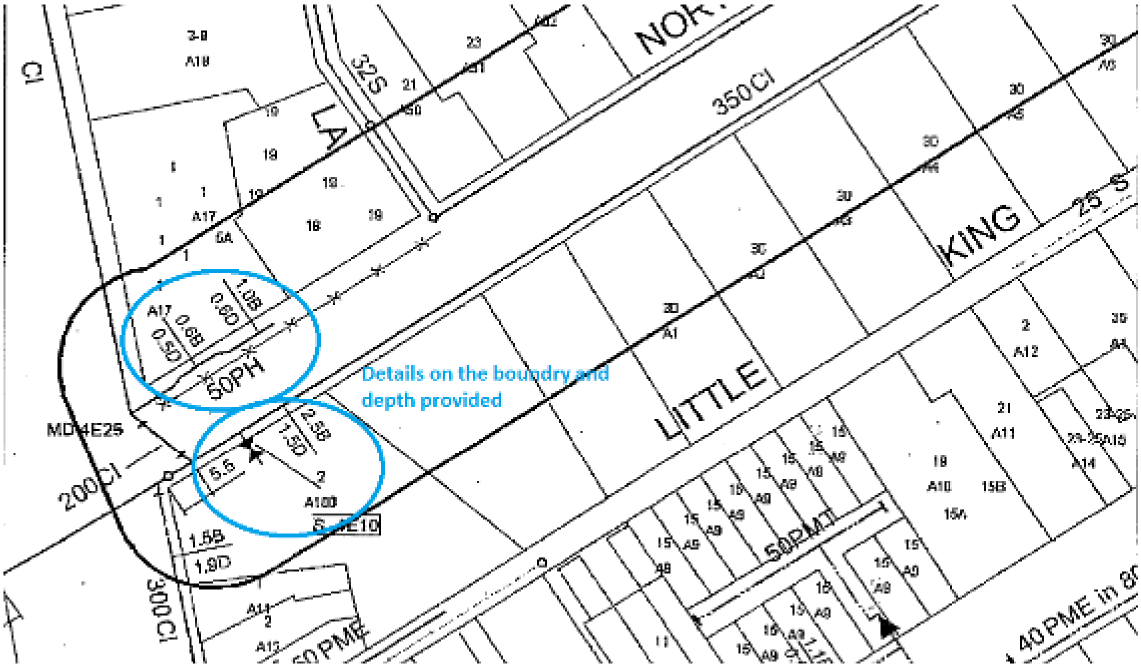

Figure 7: Gas Services along North Terrace

As shown in (Figure 7) the gas pipes have their depth and boundaries specified with all the length and dimensions (APA, 2010). However in the telecommunication information, Telstra has dimensions of the cables specified but with no depth mentioned or distance from the sides of the street (Figure 8). SA Water (water mains and sewerage) have their pipes drawn with their dimensions mentioned, but with no depth or boundary distance indicated (Figure 9 and Figure 10). NextGen Cables have only their cables shown in a sketch but do not show any sorts of dimensions or depths or distance between them and the edge of the road (Figure 11).

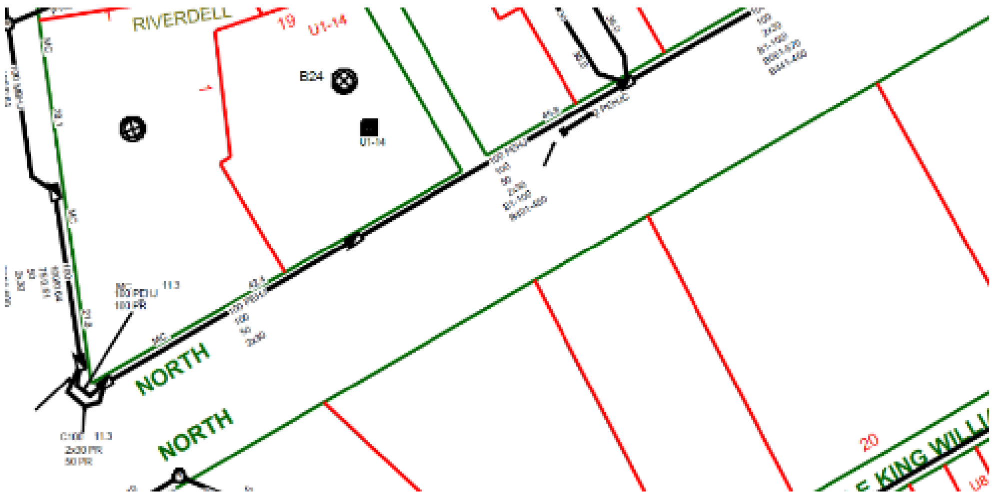

Figure 8: Section of Telstra Cable Plan

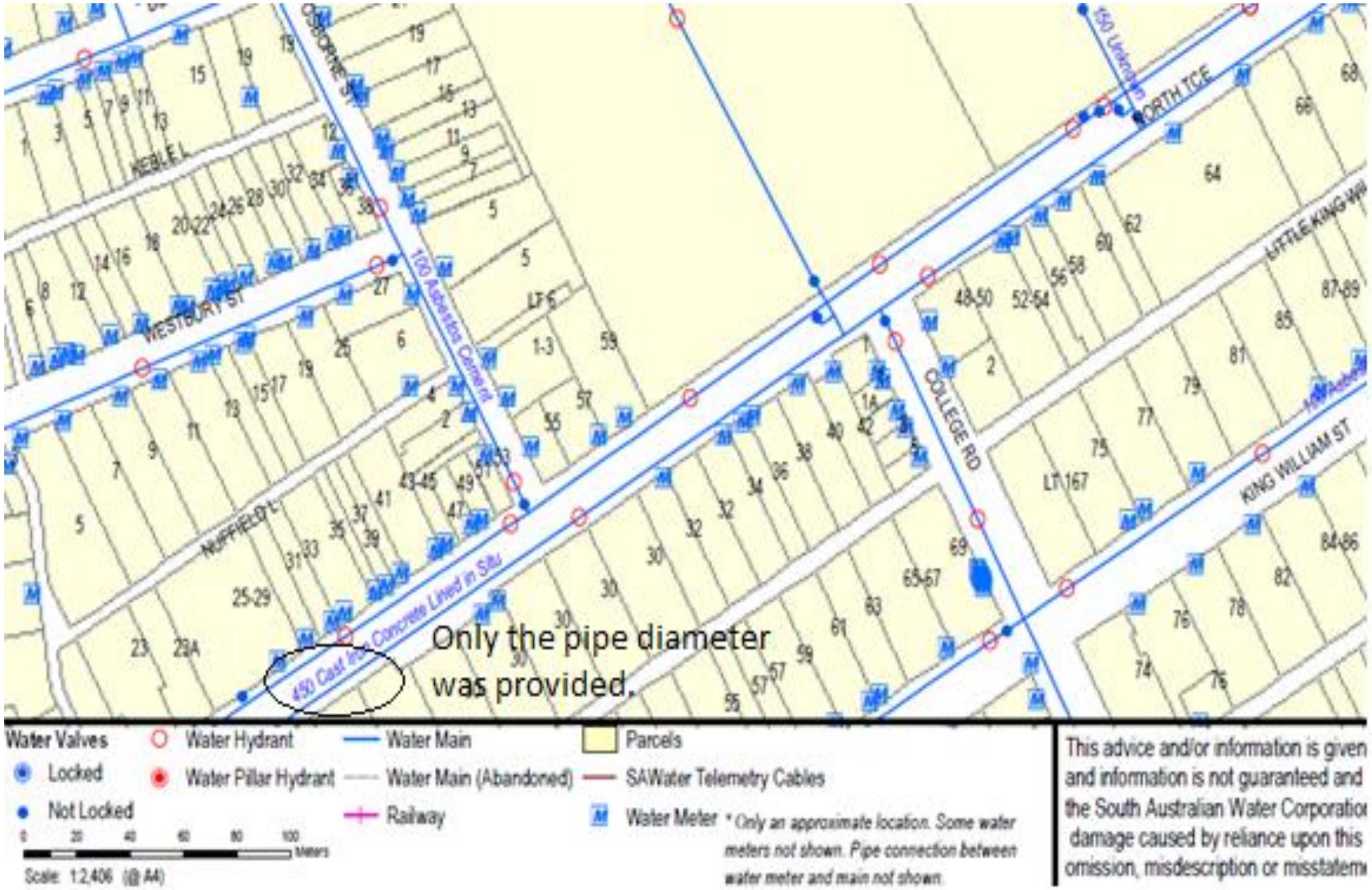

Figure 9: SA Water 'Water Reticulation Plan'

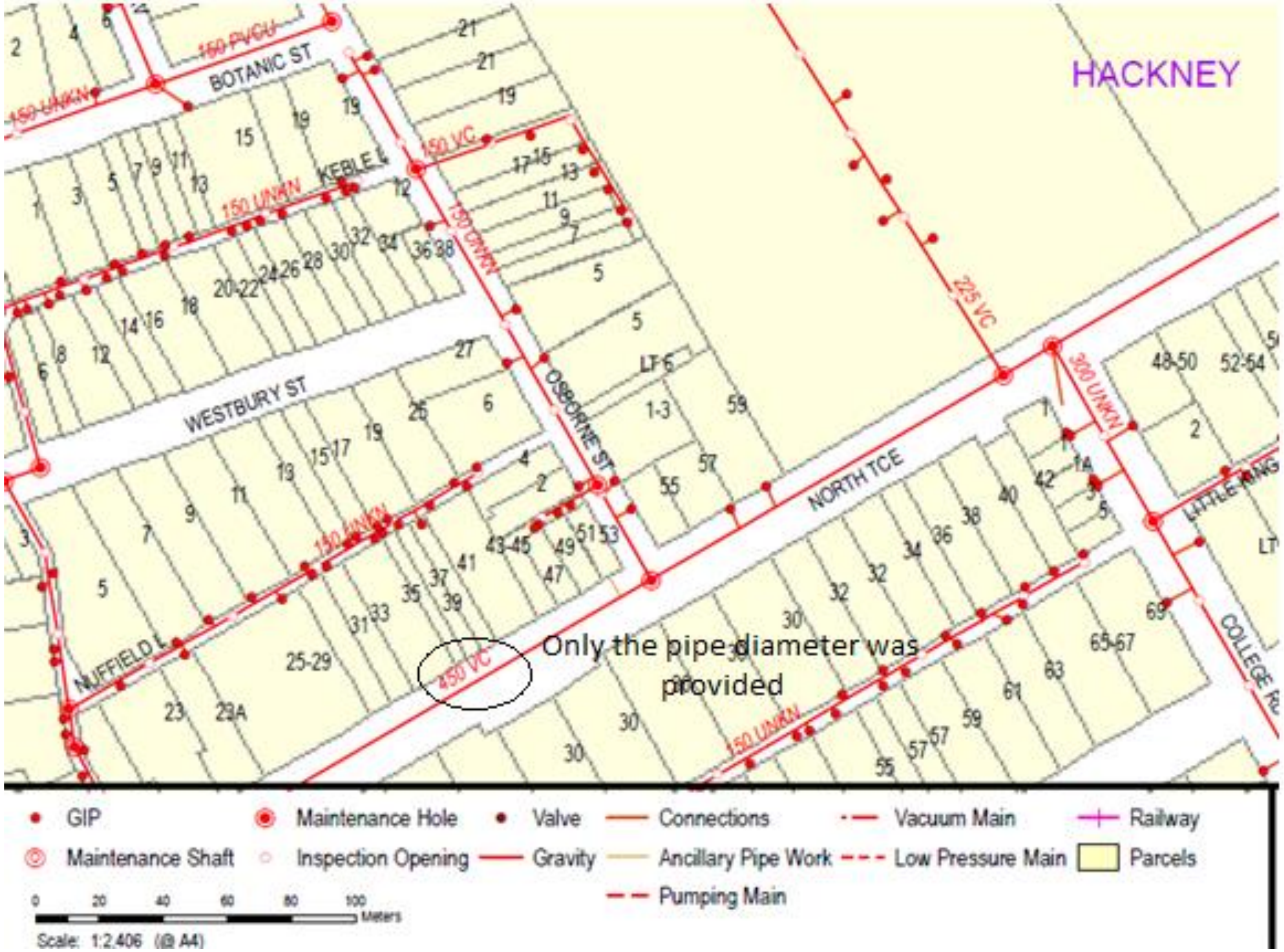

Figure 10: SA Water, 'Wastewater Reticulation Plan'

Figure 11: NextGen Cables

Hence a combination of the DBYD data and the standard service depth information and clearance information from the Urban Planning section 8.1was used for the service checking.

The depth, width, reduced levels, and dimensions of the services of the cross section between the services are shown in the Figure 12 below. Depth clearance of the gas and telecommunication services might vary slightly but is not considered as a significant factor in hindering the project. This is because the variances of clearances are between 300-450mm and 450-600mm for gas and telecommunication services respectively. Those depths will have no conflict with the design as the clearance will still be sufficient to commence construction. The Figure 12 also shows the designed stormwater pipe with no conflict in depth or width with the existing services on the opposite sides.

Clearance Check

In this section the standard clearance check between the new stormwater pipe and the existing services are included. There are four main types of underground utility services as shown in Table 6. The standard clearance information was obtained from the Urban Planning section 8.1.1and the existing clearance between the new stormwater pipe and the existing services were measured using the AutoCAD cross section drawings which are included in Appendix A.3. Below table confirms there will not be any conflict between services.

Table 6: Clearance between utilities and designed stormwater pipe

Types of Utility Underground Services	Pipe Diameter (mm)	Required minimum horizontal clearance (mm)	Existing Horizontal Clearance (mm)	Required minimum vertical clearance (mm)	Existing Vertical Clearance (mm)
Existing Gas Pipe	350	600	1950	150	3090
Telecommunications cable	350	600	2950	150	2790
Existing Sewer	450	1000/600	7100	500	1664
Existing Water Main	450	600	1900	150	864

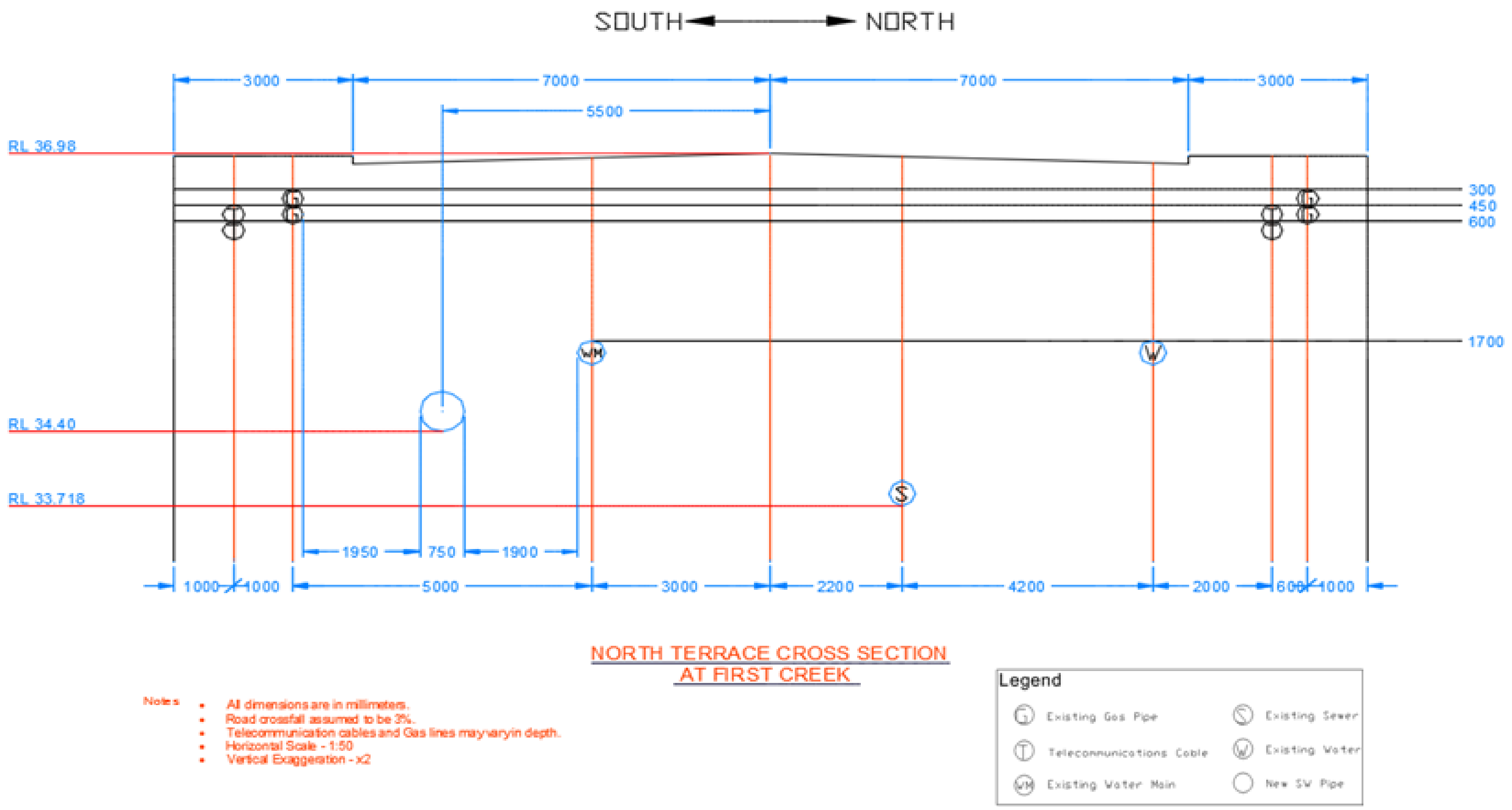

Figure 12: Road Cross Section at First Creek

4.1.13. Technical Specifications

4.1.13.1. Pipes

All pipes used in the system will be Rubber Ring Joint (Belled Socket) Pipes made from precast concrete. All pipes will come with rubber rings included and should be installed in accordance with, the design for the installation of buried concrete pipe standard AS/NZS 3725.

Typical pipe lengths will be 2.44m and both the 600mm and 750mm diameter pipes will be of Standard Strength Load Class 2. (Humes, 2009)

The contractor shall construct the works within the following tolerances:

- Vertical – 50mm
- Horizontal – 150mm

All pipe slopes need to be greater than 0.5%. (Ahammed 2014) and the storm water system was designed meeting this requirement.

4.1.13.2. Pits

All designed pits in the stormwater system upgrade will be modular pits with internal sizes of 900 x 900 x 900mm and made from precast concrete. Pits 900 x 900 x 900mm are the standard sized pit that will be used in our system based on the largest pipe diameter of 750mm. Riser units will be used in depths where the pit may be below the surface level. For the 900 x 900 x 900mm sized pit riser units of either 100, 300 or 600mm in height may be used to reach the required IL. All modular pits will come with a lid with a grate (Humes, 2009).

4.1.13.3. Construction Method of Stormwater System

The following installation of the stormwater pipes and pits will be carried out according to the following standards:

- Design & Installation: AS/NZS3725:2007
- Manufacture: AS4139:2003
- Quality: ISO9001:2008

In our design we are recommending the use of a belled-socket piping joint. The belled-socket joint is preferable as it allows for alignment adjustment. Rubber Ring Joint Concrete pipes with a unique rubber 'V' ring joint will be used. The rubber joint will provide an adequate seal against water leakage from the pipe.

A schematic diagram of a typical belled-socket RRJ pipe and a detail of the rubber joint are given in Figure 13 and Figure 14, respectively. From design and DRAINS analysis, the nominal pipe diameters used will be 600mm and 750 mm.

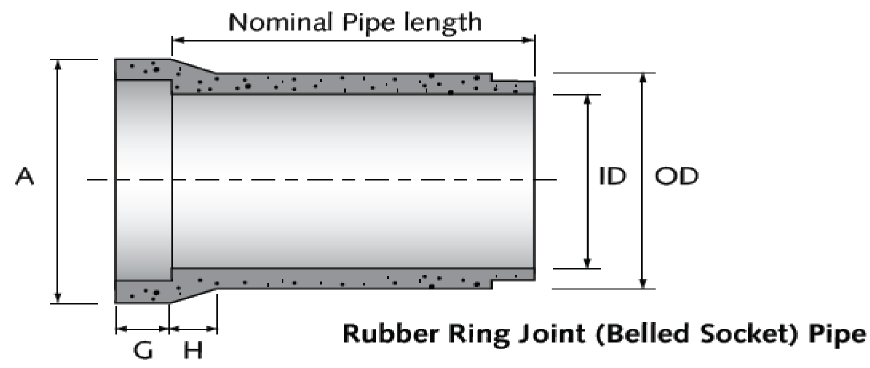

Figure 13: Rubber Ring Joint (Belled Socket) Pipe (Humes, 2009.)

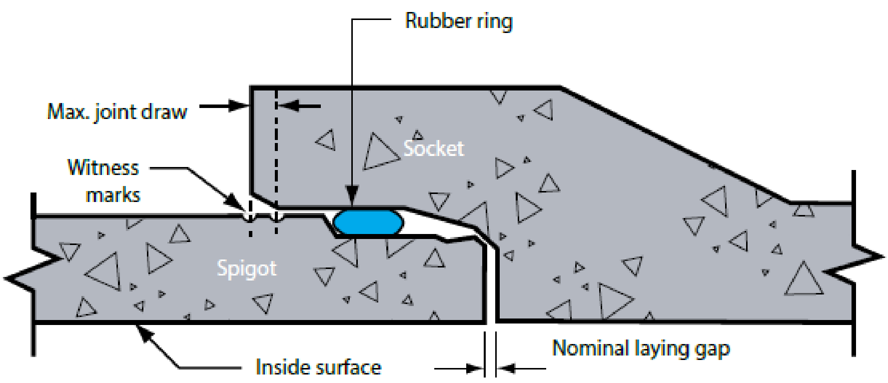

Figure 14 Detail of the rubber joint in the belled socket pipes (Humes, 2009)

4.1.13.4. Installation of the stormwater pits

The new North Terrace Drainage design consists of both side entry pits and junction pits. Standard modular pits can be used for both of these purposes. As the stormwater system is designed in accordance of Adelaide Council Standards, single side entry pits and 900 x 900 x 900mm modular pits are selected for the project. The junction box connection is as shown in Engineering Drawing HF-109.

Delivery

Prior to delivery, a pre-installation site meeting will be held to finalize shipping plans including the sequence of deliveries and the order of unloading and installation of the pits. It is important that the delivery of the pits is planned correctly to save time and minimize the site and road congestions. The aim of the meeting is to determine the capacity of the lifting crane and the site location at which the pits will be lifted.

Lifting

The weight of the selected 900 x 900 x 900 mm concrete pit is about 930 kg. (Humes, 2012) Since pits are heavy, lifting of concrete pits should be handled in a safe and correct manner. First, the approved lifting clutches will be connected to the cast-in lifting anchors (Figure 15). All the pits will be supplied with cast-in lifting anchors to enable safe handling. The cast-in lifting anchors are designed in a way to prevent stress and possible concrete cracking during lifting. Second, a spreader beam will used to lift the pits as shown in Figure 16 to avoid any chipping of the pit.

Figure 15 preparing a concrete pit prior to lifting

Figure 16 lifting a concrete pit (Humes 2012)

Pit Installation

Pit installation will be carried out as follow:

1. Before delivering the pit into its position, the size and location of the pipe should be marked on the thin knockout section of the pit as shown in Figure 17; the knocking should be started from the center of the knockout circle, using a proper tool such as a sledge hammer.
2. A small ball-peen hammer or a similar tool should be used to make the circular hole up to required outer diameter of the pipe. (750mm and 600mm)
3. The stormwater pit will be lifted using swift lift clutches and lowered into its position. The pit will be guided by crane operators into the downstream pipe.
4. The pipe will then need to be lifted slightly to enable the joint to be made before the pit base touches the ground;

5. The pit level should be checked using a spirit level and also the downstream and upstream invert levels should be checked.
6. The pit should be set directly on prepared bedding material and should make sure of enough clearance underneath the pipe to allow for sealant application.

Figure 17: Modular pits with knockout sections (Humes 2012)

Pit Installation (multiple segments)

The installation of the multiple segment pits will be carried out as follow:

1. The segments will be sealed using a site approved non-shrink grout;
2. The sealing product will be applied to cover the entire contact area; therefore it should be made sure of no remaining gaps. This should be verified and approved by the site engineer;
3. The second segment will be placed on top of the already installed segment. A quality bond will be achieved according to the weight of the pit segment;
4. Wait for the required curing time without disturbing the segment. The curing time depends on the type of the sealing product that will be used.

Joining pipes to pits

The concrete pipes will be joined to the storm water pits as follow:

1. The pipe should be placed into the penetration within the pit wall. While placing the pipe a cutting will be done, so that pipe will flush with the internal wall of the pit.
2. The pit wall will be sealed using an approved non-shrinkage epoxy grout. The specification of the adequate epoxy grout mix should be approved by the project

manager, and the site engineer should verify this on site. The seal should be applied from both the outside and the inside to guarantee an adequate sealing for water proof and a quality flush finish with the pit wall as shown in Figure 18 and Figure 19.

Figure 18 Outside connection of the pipe with the pit using site-approved cement mix(Humes 2012)

Figure 19 inside connection of the pipe with the pit using site-approved cement mix

Side Entry Pits

Side entry pits top cover will be installed directly on top of modular pits which will be connected to the main drainage pipe via a junction box. Cement mortar can be applied up to 10mm if required for leveling purposes. The opening of the inlet system has to be flush with the inner surface of the pits as in Figure 20**.**

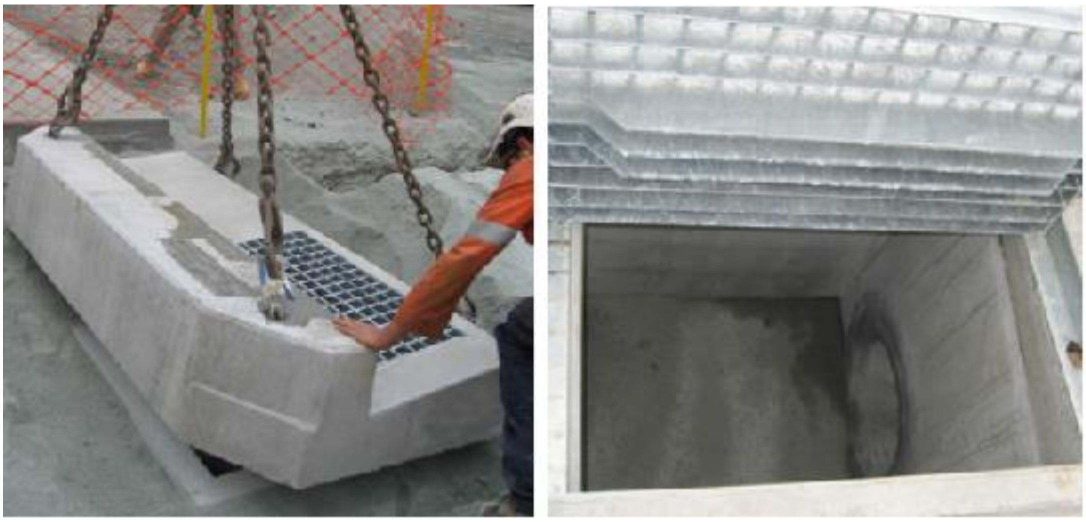

Figure 20: Side Entry Pit Installation (Humes 2012)

4.1.13.5. Installation of the stormwater pipes

In regarding the pipe installation and pipe connections the following specifications have to be considered, according to the manufactures installation instructions. In this project Humes pipe and pits are considered in purpose of technical specifications. Therefore Humes installations guidelines are considered.

Handling on-site prior to installation

When the Rubber Ring joint (RRJ), reinforced concrete (RC) pipes are delivered to the site, it is necessary to minimise the exposure of the rubber rings to the direct sunlight, especially on warm days (Humes, 2009). Therefore, the rubber rings will be kept in plastic bags and stored in the pipe barrel. Furthermore, the pipes should be stacked on a level surface to avoid the risk of falling which can cause both injuries to workers and property damages (Humes 2009**)**

Lifting the pipes

Either a 30-ton excavator or lifting crane can be utilised to lift the pipe. The lifting machine should be certified for the pipe load. The pipe section will be lifted using certified chains that can be connected to a hook that's attached to the centre of the pipe. The photograph in Figure 21 shows a concrete pipe being lowered into a trench using a crawler excavator.

Figure 21 lowering a concrete pipe into a trench (Humes 2009)

Fitting the rubber joints

The rolling rubber rings do not need lubrication (Humes, 2009.). However, it is necessary to ensure that both the rubber rings and the pipe sockets are clean and dry prior to the insertion of the rubber rings. It should be ensured that the ring has no twists to guarantee uniform rolling when joining. The rubber rings should be fitted to the pipe's spigot on the ground before lowering the pipe into the trench.

Pipe Foundation

The foundation ground under the pipe should be prepared to provide stability and uniformity along the pipeline. Therefore, any loose or soft soils should be removed and replaced with compacted granular material to ensure that there is no any differential settlement along the pipeline's centerline. Excessive differential settlement under the pipeline can potentially cause misalignment and damage to the pipeline (Humes, 2009). Pipe foundation details are included in more depth in geotechnical section 6.3.4.

Figure 22: preparing the bedding for the pipe and pipe's socket (Humes, 2009)

Laying and joining the pipes

Pipe laying should be progress in upstream direction with spigots pointing downstream having been inserted into the socket. Therefore when laying the RRJ pipes, it is recommended to fit the spigot into the socket to ensure that joints are restrained from opening due to pipe movement during the settlement. These RC pipes should be laid "top up" where the lifting holes or cast in lifting anchors should be on top side on laid pipes. (CPAA 2012),

A certain force should be applied to joint RRJ pipes. The required force increases as the pipe diameter increases. Pipes larger than DN450 up to DN1200 can be joined using leverage tools that are shown in Figure 23.

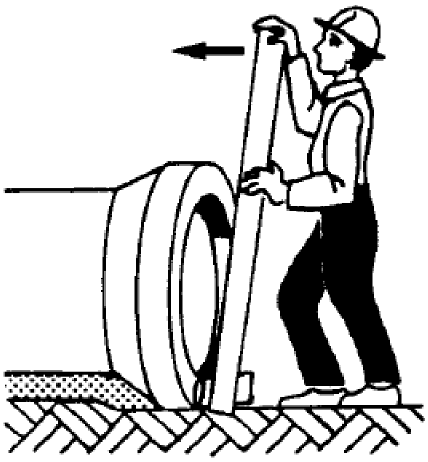

Figure 23 Leverage tools used to join pipes of size up to DN1200 (Humes 2009.)

4.3.1.1. Construction Plan

As the storm water system 780m in length including the pit to pit connection across the road, a time period of 70 days were roughly assumed for the construction of the stormwater system. Five stormwater pipe sections of 2.44m will be assumed to be installed within a one day. For each day after the pipe installation backfill the trench, compaction of the top layers and laying of bitumen on top will be undertake. As North Terrace is a busy road, the construction process can be slow down than the expected time period. Most of the across the road pit to pit connections have to be done overnight as the full road closure won't be applicable for North Terrace during day time. The pipe and pit handling at the construction site have to be followed according to the work safety and health regulations under water industry regulations Act 2012.

4.3.2. Task Specific Safety Assessment

Task Specific Safety Assessment Form	
Department/Section: Water	26/05/2015
Brief Description : Stormwater system installation.	
• Pipe/Pit Installation • Use of heavy machinery • Drilling • Pipe/Pit unloading • Pipe/Pit Delivery	
Summary of major risks or hazards	
• Collapse of trenches • Pipe and other construction falling down due to unsafe handling • Accidentally hitting services pipe through excavation which can cause explosion or causing flooding. • Exposed to buried services and or electrical cables. (Explosive services tools(power, tele) • Liquid and chemical exposure	
Mitigation strategies	
• Employee and visitors must be given suitable training and instructions. • Conduct safety plan to identify hazards and assessing risks. • Pipes must be in alignment with each other and in the same direction. • In the lower bundle of pipes, pallets must be centred. • Permit to work forms. • House Keeping (Clean work area, standard disposal procedures) • Equipment used must be inspected before the commencement of construction. • Use barriers to separate pedestrians and set mobile plant to reduce the risk of collision. • Benching, battering, or shoring the sides of the excavation to reduce the risk of ground collapse.	
Safety equipment required & number	
• Safety Helmets , Steel cap boots, Gloves and masks , Reflective polyester safety vest • First Aid Kit • Signs to show site under construction	

4.4. Water Harvesting

4.4.1. Rainwater Tanks Design

4.4.1.1. Rainwater tank location

From the feasibility study stage, the water harvesting system would be designed to cover up to 25% of total stormwater runoff which translates into a total capacity of approximately 130 kL. Through studying the roof sizes and the immediate surroundings of properties within the catchment area, certain properties have been selected based on various criteria outlined below. Figure 24 and Figure 25 illustrate the locations of all selected properties and their respective rainwater tanks.

Criteria for selected properties:

- Close to vegetation – shorter delivery route for irrigation
- Available space for tank installation
- Relatively significant roof size
- Fulfil the requirement of capturing 25% roof runoff.
- Optional: Additional tanks can be implemented for properties 1 and 2 in form of underground concrete tanks to further reduce roof runoff. Due to lack of reasonable emptying means, these are not selected as main design site

Criteria for selection of tank locations:

- Near the existing down pipes
- Considering the other properties and make sure tank does not block their natural light, ventilation or outlook or detract from the streetscape
- Requirement of pump, (environmentally safe and make sure it is located in a suitable area)
- Consider access to the tank for maintenance purposes;
- Building regulations may also limit where you can locate a rainwater tank on your property in relation to the front, side and rear boundaries;
- Be aware of the stress placed on fences, from a rainwater tank above or near any retaining walls on property and neighbour's property
- In order to reduce construction cost, tank should be located close to the building, near existing downpipes.
- The location of the tanks also considers available spaces on the concerned properties

➢ **Underground** (Figure 262): It can be saved spaces but cost might be higher than above ground option, may be require pump to extract the water. Some area may require regulation of the council.

➢ **Above ground** (Figure 263) :most popular option, efficient spaces

- In line with South Australian (SA) Council guide line (Department of Planning and Local Government 2010)

Type of the tank:

- Plastic (Polyethylene)- most common type, easy to move, many shapes and colours
- Metal tank or Steel stainless steel- above ground, many shapes and selected colours
- Fibreglass tank- above ground, many shapes and sizes, can be located underground as well
- Concrete tank- underground, cylindrical or cube.

Figure 24: Selected properties (in red) and rainwater tank locations (in yellow) (Google Maps 2015)

Figure 25: Selected properties (in blue) and rainwater tank locations (in yellow) (Google Maps 2015)

4.4.1.2. Rainwater tank capacity

Rainwater tank size was determined using Raintank Analyser (UniSA 2015). Sample of input parameters used for Property 26 would be shown in

Table 7 and the output shown in Table 8.

Manual inputs include:

- Roof area and irrigation area: measured using Google Map Area Calculator Tool (Daft Logic 2015).
- Household demands are estimated based on the occupants of each property (Coombes 2003).
- Initial losses: based on roof types as stated by South Australian Government's guideline on Rainwater tank (Department of Planning and Local Government 2010).
- First flush loss: is calculated as 10 L per 100 m^2 roof area (Department of Planning and Local Government 2010) calculated under section 4.4.2.1

Table 7: Required parameters for Raintank Analyser for property 26

Manual inputs	
Total roof area (previous section, m^2)	207
In house daily demand (L)	470
Irrigation area (m^2)	70
Initial loss (terracotta, mm)	1
First flush loss (L)	20
Plant	Turf
Software Assumptions	
Irrigation	**Sprinklers**
Plant available water(PAW, mm)	16.5
% allowable depletion	50%
Application efficiency	75%
Irrigation depth (mm)	11
Soil Type	Sandy
Root zone depth (mm)	150
Available holding capacity (mm)	110
City	Adelaide

Table 8: Raintank Analyser Output data for property 2 (UniSA 2015)

Rain-tank Analyser Output Data	
Rainfall input data years	25.2 year
Average annual rainfall	548 mm/year
Average annual in-house demand	171.55 kL/year
Average annual irrigation demand	44 kL/year
Tank Selection	
Suggested tank size is approx.	3667 L
Average annual	71 kL/year
Average number of days with zero supply	193 days/year
% of total demand supplied by tank	33 %

The same procedure would be carried out for all properties to obtain rainwater tanks required for each property. The resulting tank sizes are analysed and enlisted into 4 main categories. While all 4 categories will be designed in the Geotechnical and structural section 6.5, their approximate dimensions are predicted to be similar to information given in Table 9.

Table 9: Required tank capacity for each selected properties

Property No *	Roof Size (m^2)	Required Capacity (L)	Suggested Tank Size	Category	Location
2	806	9278	9500	4	South
6	640	7278	8000	3	South
7	1060	11944	8000+4000	1+3	Southwest
8	504	8667	9500	4	North
9	286	5000	5500	2	North
10	768	8833	9500	4	West
11	300	5333	5500	2	Southwest
12	635	7278	8000	3	Southwest
13	400	7000	8000	3	Northwest
14	450	7667	8000	3	East
15	261	4667	5500	2	North
16	239	4333	5500	2	North
17	275	5000	5500	2	East
19	228	4000	4000	1	South
20	225	4000	4000	1	South
21	226	4000	4000	1	South
22	296	5333	5500	2	South
23	471	8000	8000	3	South
24	286	5000	5500	2	Southeast
25	277	5000	5500	2	Northwest
26	207	3667	4000	1	Northeast
Total		= 131,278 L (25% of total roof runoff)			

*Note: The property numbers are presented according Figure 24 and Figure 25.

4.4.2. Rainwater Tank Connection System

4.4.2.1. First flush Device

First flush devices are designed to wash off potentially harmful contaminations accumulated after a long dry period. First flush will be diverted using a downpipe diverter. The size of the pipe is determined based on individual roof size. For optimal water quality, it is recommended that the first flush volume is estimated at 10 L per 100 m^2 roof area (Department of Planning and Local Government 2010). The components of the diverter are explained in Figure 26 and Table 10.

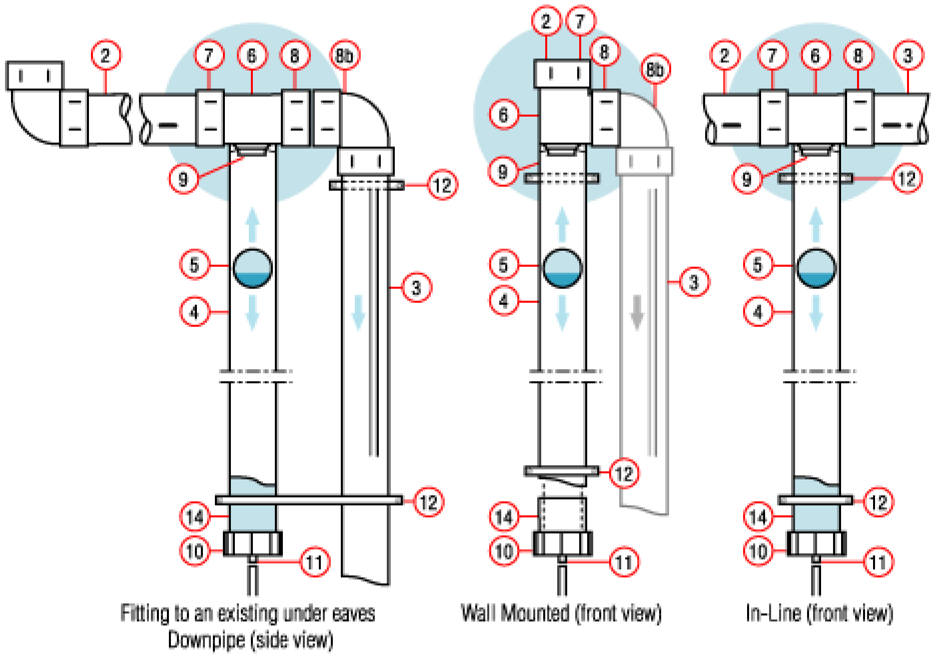

Figure 26: Typical downpipe first flush device components (Rain Harvest Systems 2010)

Table 10: Reference chart for downpipe first flush device components (Rain Harvest Systems 2010)

Reference chart					
1	Diverter chamber	**7**	Chamber Inlet	**13**	Plastic Filter Screen
2	In-feed from the roof	**8**	Chamber Outlet	**14**	Socket
3	To the tank	**8b**	Elbow		
4	Diverter chamber	**9**	Ball seat		
5	Sealing ball	**10**	Screw cap		
6	The Junction	**11**	Flow control valve		

The diverter pipes' diameter will be 100 mm (WDDP01) in corresponding the inlet diameter calculated in section 4.4.2.2 (Rain Harvest Systems 2010). Length of Diverter Chamber is approximately 1 m per 8.8 L of water (Rain Harvest System 2010).

Table 11: Downpipe diverter length for each property

Property	Roof Size (m^2)	First Flush Volume (L)	Diverter length (m)
2	806	81	10
6	640	64	8
7	1060	106	12
8	504	50	6
9	286	29	4
10	768	77	9
11	300	30	4
12	635	64	8
13	400	40	5
14	450	45	5
15	261	26	3
16	239	24	3
17	275	28	4
19	228	23	3
20	225	23	3
21	226	23	3
22	296	30	4
23	471	47	6
24	286	29	4
25	277	28	4
26	207	21	3

4.4.2.2. Inlet and Overflow Design

As rainwater drains away from the roof, it will enter a complex distribution system. To balance inflow and overflow, the diameter of inlet and overflow pipes will be the same. A typical connection of inlet and over flow design is shown in Figure 27.

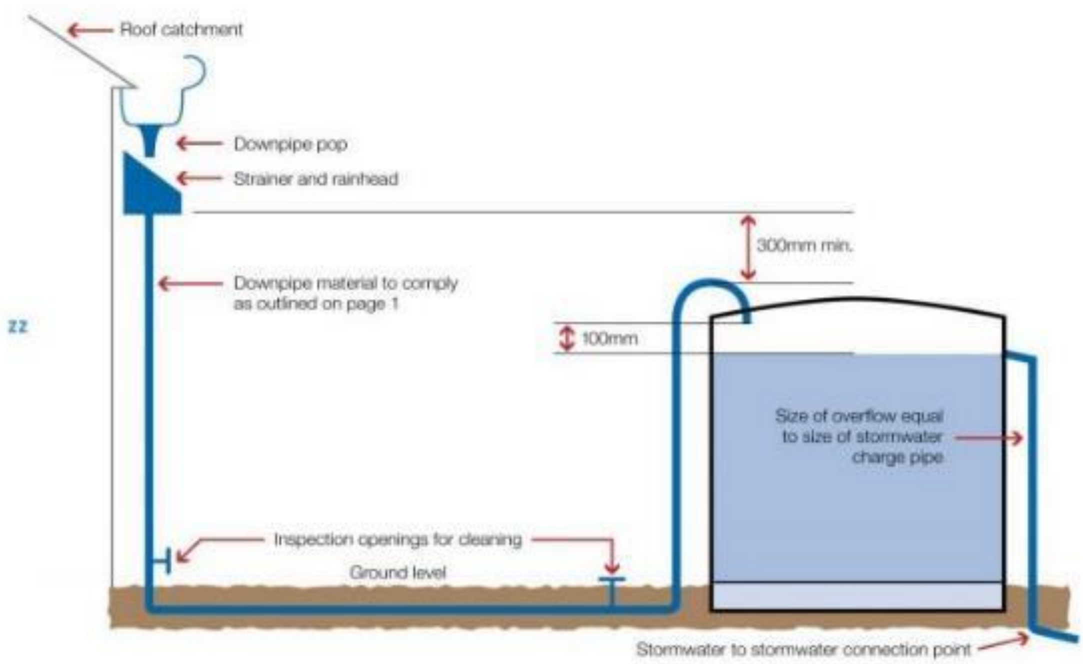

Figure 27: Typical connection system of inlet and overflow pipes. (VBA 2014)

The required diameter of inlet pipes would be designed to accommodate the stormwater flow rate generated by the roof area. Using property 13 (roof area = 635 m^2) as an example, the stormwater flow rate was calculated based on the roof area similar to Appendix 1 . The flow rate value was obtained to be Q = 0.028 m^3/s using the Flow calculation Excel spread sheet.

Manning equation states that:

$$Q = \frac{1}{n} A R^{\frac{2}{3}} S^{0.5} = 0.028\, m^3/s$$

Where Q: flow rate, R: Wetted perimeter, S: Channel slope, A: Channel area

$$\frac{1}{0.011} \pi \left(\frac{d}{2}\right)^2 \left(\frac{d}{4}\right)^{\frac{2}{3}} \left(\frac{3}{7}\right)^{0.5} = 0.016\, m^3/s$$

$$d = 0.096m = 96\, mm$$

Using available market size, diameter of inlet and overflow pipes for property 13 would be 100 mm.

The most critical parameter identified was the slope of roof where property 13 was the critical case. Therefore, it was concluded that inlet and overflow pipe of 100 mm diameter would be used for all properties involved in rainwater harvesting scheme. Material used will be PVC pipe.

In order to minimize clogging of the inlet, the roof rainwater will be discharged through an open air nozzle into the filter head of rainwater tank. To prevent overflow, outlet pipe will be connected at 100 mm from the ceiling of the tank (VBA 2014). The overflow rainwater will connect to the existing storm water drainage of within the property.

4.4.2.3. Technical Specifications

Inlet pipes and fittings

Inlet pipes diameter was designed to be 100 mm for all properties in accordance to AS/NZ3500.3 (Arid 2008).

Eaves gutter is a horizontal trough that collects rainwater along the perimeter of the roof as shown in Figure 28. Its slope must be at least 1: 500 (0.2%) or higher (Arid 2008).

Figure 28: Examples of eaves gutter (hobbithouseinc 2015)

Box gutters (Figure 29) for parallel roofs and internal guttering are expected to be at least 1:200 (0.5%). All gutter slopes must be designed to direct water flow towards the rainwater tank location.

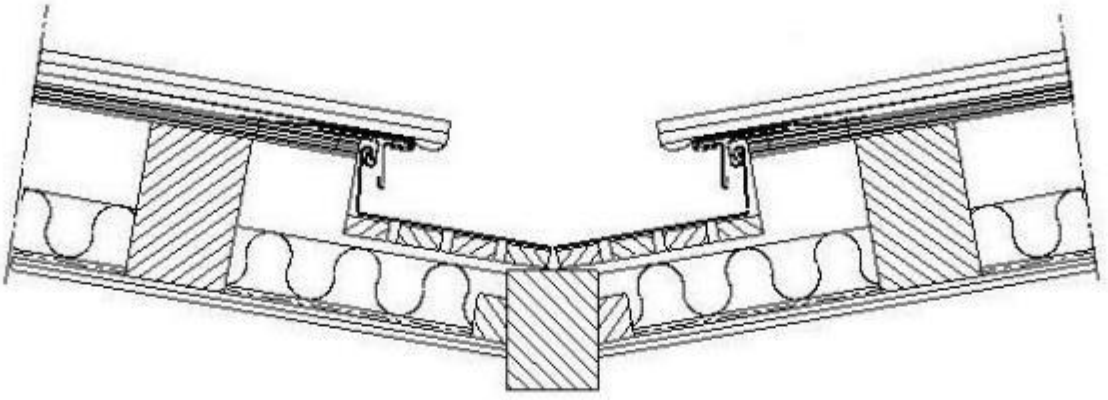

Figure 29: Example of box gutter (Ned Zink, 2015)

All gutters have to be continuous and only lapped joints are allowed. Lapped joints must be in the direction of flow to the nearest downpipe. There is usually a specific joint bracket for manufactured gutters which can be found in the manufacturer's installation requirements. The brackets are to be placed without hindering water flow, maintenance and to avoid debris build-up, blockages etc.

Rainwater head would include a screen (Figure 30) to prevent contamination due to leaf, vermin and insects. Rainwater heads come in a manufactured size that matches the pipe size and are provided as a set. Regular cleaning of the screen should be carried out to avoid flow hindrance.

Figure 30: Typical leaf screen (Patrick, D & Steve, S 2013)

Overflow pipes and fittings

The direction of tank overflow must ensure no stormwater ponds form under the floors or foundation of the building. This is to minimise the risk of timber rotting and the corrosion of metal fittings and reinforcement in the concrete as it could lead to potential cracking of house and retaining walls.

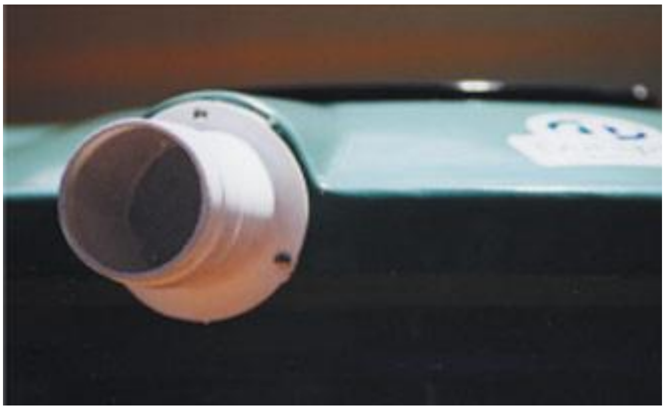

Figure 31: Typical overflow pipe outlet (Water Tank Accessories 2013)

Overflow outlet (Figure 31) is designed to be on the highest water surface of the tank. To balance in and out flow, its diameter has to be the same as inlet pipe, in this case 100 mm. The overflow pipe must be connected to the nearest outlet pipe of the corresponding property through a reflux valve.

Reflux valves are a backflow prevention device that is able to avoid reverse flow of water from potentially polluted source to minimise contamination of stormwater reticulation (SA Water 2006). A 100mm reflux valve is shown in Figure 32.

Figure 32: Typical 100mm reflux valve with access cap (Pipe and Fittings 2015)

4.4.3. Pump Design:

4.4.3.1. Pump selection

As stated in the feasibility stage, the captured rainwater in water harvest scheme has various applications including irrigation and toilet flushing. In order to do that, a certain amount of flow rate is required as provided by Department of Planning and Local Government (2010) in Table 12. As seen in Table 93, the tanks required are relatively tall thus it would be tough to implement gravity tank design. In dry season, water head would be insufficient to provide necessary flow rate. In such case, a pump would be used.

Table 12: Indicative Flow Rate and Pressure Requirements for a range of Demands

Application	Flow Rate Recommended (LPM)	Water Pressure Recommended (kPa)
Lawn sprinkler/Garden hose	15	140
Garden irrigation	60	400
Internal use	15	50
Washing machine	15	100
Toilet flushing	10	50

The pump would be designed for a critical case scenario in this report based on required pipes and required heads. Table 13 shows the required parameters for calculating total head loss. The static lift was identified to be 4m in case property owners would like to use captured water for washing or toilet flushing purpose on the second floor.

Table 13: Designed parameters for calculating total head loss

Surface roughness coefficient, Poly (mm)	0.007
Length (m)*	20
Static lift (m)*	4
Minor losses coefficient	3
Friction losses of suction pipe	0

Table 14: Total head loss in various pipe diameters

d (m)	Q (L/s)	Q (m3/s)	V (m/s)	Reynold number	λ	H_T (m)
0.02	3.3	0.0033	10.5	209039.3	0.018	122.21
0.03	3.3	0.0033	4.7	139359.6	0.018	20.69
0.04	3.3	0.0033	2.6	104519.7	0.018	8.30
0.05	3.3	0.0033	1.7	83615.73	0.019	5.53
0.06	3.3	0.0033	1.2	69679.78	0.020	4.66
0.07	3.3	0.0033	0.9	59725.52	0.020	4.33
0.08	3.3	0.0033	0.7	52259.83	0.021	4.18
0.09	3.3	0.0033	0.5	46453.18	0.021	4.11

Table 14 shows the various values of head loss in many sizes of pipe. Pump would be selected to fulfil the head loss and diameter.

In order to provide the most critical flow rate and water pressure needed for lawn and sprinkler in 12 and head loss in 14, an equivalent pump of Ozito 600W Transfer Water Pump is required as seen in Figure 33.

Figure 33: 600W Transfer Water Pump (Bunnings 2015)

4.4.3.2. Pump cover

It is natural for the pump to generate noise which is the main cause of tension and aggravation between neighbours. Therefore, it is important to minimise pump noise to the standard of 45 to 51 dB (EPA 2008) using acoustic pump cover for box. Another alternative would be using a submersible pump, example in Figure 34.

Figure 34: Ozito 350W submersible water pump (Bunnings 2015)

4.4.3.3. Technical specification

There are few aspects to consider in pump selection (Arid 2008):

- Provide necessary power or head for household needs as an example shown in Table 15: Sample pump specifications (Bunning's 2015)
- Performance of selected pump should abide by AS/NZ/NZS 2417.2001 minimum grade 2 requirements, electrical safety standards AS/NZS 60335.2.41 and electromagnetic compatibility AS/NZS CISPR14.1.
- Connected pipe types and diameters are chosen based on required pressure and flow rate
- Ensure there is no leakage
- Pump is to be as close as possible to the tank
- Reduce suction lift by placing pump as close as possible to tank
- Limit number of fittings in pipe work to minimise friction losses by valves, elbows and filters
- Consider installing floating pump intake to allow clean water into pump inlet

Table 15: Sample pump specifications (Bunning's 2015)

Model Name	600w Transfer Water Pump	Model Number	TRP-650
Material	Metal and Plastic	Colour	Silver, Grey, Black, Red
Product Dimension	W: 375 H: 240 L:195	Package Dimensions (mm)	W: 375 H: 240 L:195
Weight	10 kg	Maximum Flow (LPM)	60
Float Switch	No	Max Pressure (kPa)	4000
Horse Power (hp)	0.87	Maximum working temp (°C)	35
Inlet Connection	25mm Female	Min working temp (°C)	1
Outlet Connection	25mm Female	Max Suction Lift (m)	8

This pump will also be sufficient in case property owner would like to use the captured rainwater for toilet and washing. The pump can be connected to water main through a dual check isolation valve (Figure 35) to avoid rainwater entering water supply mains.

Figure 35: Typical 25 mm dual check isolation valve (AVG 2013)

To reduce noise, the pump would be placed inside an acoustic pump cover and correspondingly, a 25 mm hose can be connected for gardening as seen in Figure 36 andFigure 37.

Figure 36: Acoustic filter enclosure (Acoustic Filter Enclosure 2012)

Figure 37: Typical 25 mm x 20m hose (Bunning's2015)

4.4.4. Maintenance Plan

While the rainwater tank is a simple system, it does come with certain risks such as water quality that may occur when the system is not maintained properly. Therefore, it is necessary to carry out maintenance for the tank, catchment, and distribution system. Maintenance of the system will be carried out periodically as elaborated in the Urban Development section 8.8.2.

4.4.5. Cost

The cost of the entire system rainwater tank system including GST had been estimated by the Department of Planning and Local Government (2010) as followed and it only consist of three categories.

Table 16: Approximate cost of rainwater tank system (Department of Planning and Local Government, 2010)

Item	Approximate Cost ($AUD) for Each Tank Size		
Size	5 kL	10 kL	15kL
Round Galvanised tank	550	850	1110
Pump	270	270	270
Pump Plumber and fittings	500	500	500
Float system	200	200	200
Concrete base	200	200	200
GST	160	180	200
Total	1800	2020	2210

A detailed bill of materials and total cost of 1 complete system was developed based on current market price of required components. It was estimated that the price would be $45,000 as shown in Bill of Quantities.

4.4.6. Task Specific Safety Assessment

Task Specific Safety Assessment Form	
Department/Section: Water	26/05/2015
Brief Description of works to be undertaken : Rainwater tank Installation	
• Soil Excavation • Concrete work • Transport and position rainwater tank • Plumbing work	
Summary of major risks or hazards	
• Inaccessible tank location. • Crushed under tank due to mishandling tank during transport or collapse of concrete base. • Damage tanks • Pipes and other structures falling down due to unsafe handling. • Injury due to inappropriate handling of required equipment. • Injury due to inappropriate posture when handling heavy items. • Exposed to existing underground services and or electrical cables. (Explosive services tools (power, tele)). • Damage current property outlet during installation of overflow pipe.	
Mitigation strategies	
• All personnel involved must be given suitable training and instructions. • Conduct safety plan to identify hazards and assessing risks. • Strictly abide by WHS standards. • Plan accessibility before transporting tanks to sites. • Equipment used must be inspected before the commencement of construction. • Contact DBYD for information of RL of existing services. • Ensure all supporting structures are secured	
Safety equipment required & number	
• Safety Helmets ,Safety Shoes	

- **Gloves and masks**
- **First Aid Kits**
- **Reflective Polyester safety vest**

4.5. Bioretention System Design

A Water Sensitive Urban Design (WSUD) system will be used in this project as a tool to minimise wastewater generation and to treat the stormwater of the road and pathways to a suitable level before it's released into receiving waters. Considering the features of this project and drainage system plan, for the flexibility in location and application at a range of scales and shapes, a bioretention basin was selected in the project to treat runoff prior to entry into drainage. Since the bioretention system in the project cannot treat the whole runoff, a gross pollutant trap (GPT) will be utilised at the outlet of drainage pipe as a best management practice (BMP) for the WSUD system. Bioretention basins are used as a bonus to achieve better runoff treatment outcomes.

4.5.1. Design Description

Bioretention basins are proposed to facilitate effective treatment of stormwater runoff while maintaining a 5 yr. ARI level of flood protection for North Terrace Kent Town. According to the Feasibility report and research conducted by the Urban Planning Team, two locations for the bioretention basins are defined in Figure 38. Bioretention basin 1 is set in front of Royal Hotel carpark which is 69m away from Hackney Rd. Bioretention basin 2 is set in front of St. Peter's Junior School, which is opposite to cross-section of College Rd and North Terrace.

Figure 38: Bioretention basin locations

In this project, both bioretention basins have been designed. However, the location of Bioretention basin 2 is within school property and there is brick fencing that separates a pathway from the proposed land. Considering the high cost of removing this fence and the potential impacts on construction, Bioretention basin 2 is optional in the WSUD system. It will be implemented only with the schools permission and the councils consent. It is expected to be a good project to be represented in the school grounds.

4.5.2. Design Procedure

Different methodologies including hydrology, hydraulics, geotechnical, and biological were applied in the biorentention basin design. The Model for Urban Stormwater Improvement Conceptualisation (MUSIC) software was used in conception, design and size verifying of the basins.

The design includes following aspects:

- Selection of size and location
- Identification design criteria and catchment characteristics
- Estimation design flow rates
- Selection of perforated pipe
- Design Inlet and outflow

- Check Velocities over vegetation
- Check surrounding soil
- Filter media and plant specification

4.5.3. Bioretention System Designs

Two bioretention basins have been designed in the project. As Figure 39 shows, each Bioretention basin serves 7160 m^2 and 4240 m^2 contribution areas respectively. Both bioretention basins adopt a geotextile liner to prevent exfiltration. Coarse sediment forebay is used upstream to prevent clogging at the inlet, and a grated pit is used at downstream outlet to connect with the main drainage pipe. Bioretention basin 1 is located on the north side of the road, 69m away from Hackney Rd. The rectangular basin has a width of 2m and total length of 37.5m including 35m for Bioretention basin and 2.5m for coarse sediment forebay. Bioretention basin 2 is located on the south side of the road, 400m away from Hackney Rd. The rectangular basin has a width of 3m and total length of 36m including 35m for Bioretention basin and 1m for coarse sediment forebay.

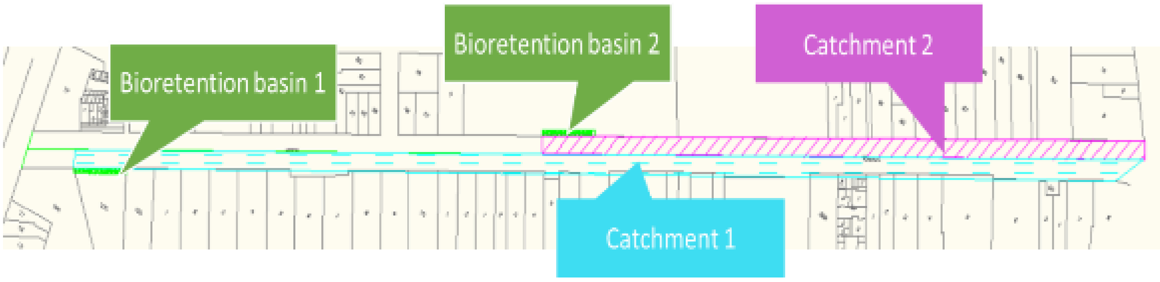

Figure 39: Treated catchment areas

A proposed layout and cross-section of the proposed bioretention system is shown below. The filtration system of the Bioretention basins includes three layers of soil media which are a sandy clay filtration media (BR1:500mm, BR2: 300mm) to support the vegetation, a coarse transition layer (100mm) and a fine gravel drainage layer (200mm).

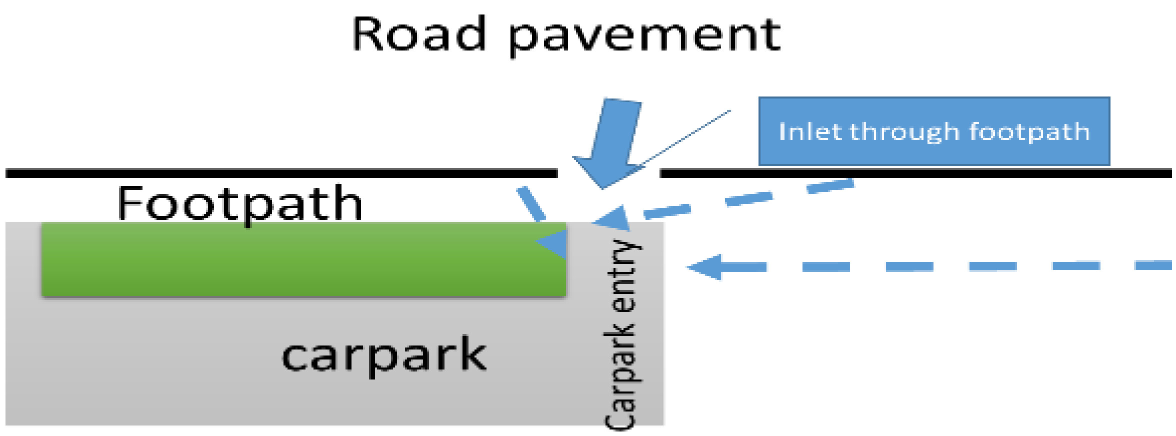

Figure 40: Layout of proposed Bioretention basin

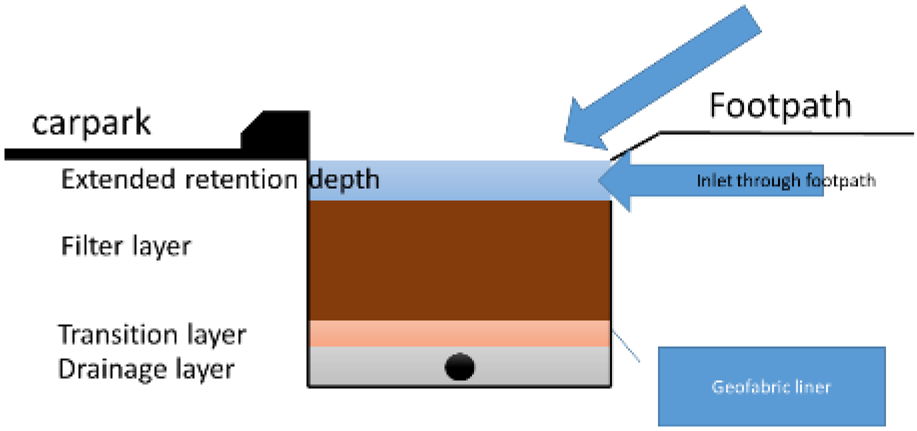

Figure 41: Cross-section of Bioretention basin

Since the locations of the bioretention basins are in currently vegetated areas, there will be a runoff rate reduction. Hence, in the design, the removal of TSS, TN, and TP is a primary concern.

4.5.4. Gross Pollutant Traps Analysis

To achieve a better runoff quality, a GPT is designed for the project. According to the environmental team, The GPT requires: minimum Trap Size Area is $6.85m^2$, Average Annual Sediment Export is 6 tonnes. The typical value of sediment density is $1.7g/cm^3$ (Andersen et al. 2010). The required sediment volume can be estimated: $V = \frac{m}{\rho} = 3.529m^3$.

Through relevant literature review and infield investigation, we selected some similar GPT from suppliers, which meet both design specifications and drainage system requirements. Among all, we chose one (Ecosol GPT 4750) whose dimensions and capacity are closest to our GPT, and based on this, we consider it is appropriate to reference its capture efficiency parameter to use in our GPT.

Ecosol Product Code	Inlet/Outlet Pipe Diameter	Approx. External Dimensions (L x W x D from invert)	Holding Capacities		
			Solid Pollutants	Free Oil and Grease	Water
		(mm)	(m^3)	(litres)	(litres)
GPT 4200	Up to 375mm	2200 x 900 x 750	0.23	268	667
GPT 4300	150 to 600mm	2700 x 1350 x 750	0.32	469	1,181
GPT 4450	225 to 900mm	3600 x 1650 x 1050	1.03	1,347	3,348
GPT 4600	300 to 1200mm	4500 x 1950 x 1350	2.43	2,994	7,211
GPT 4750	450 to 1350mm	5600 x 2300 x 1650	4.83	5,711	13,608
GPT 4900	600 to 1650mm	6500 x 2600 x 1975	8.30	9,576	22,768
GPT 41050	750 to 1800mm	7450 x 2950 x 2300	13.11	14,850	35,262
GPT 41200	900 to 2100mm	8630 x 3300 x 2625	19.52	22,793	51,698
GPT 41350	1050 to 2400mm	9700 x 3700 x 2950	27.70	30,578	72,495
GPT 41500	1200 to 2400mm	10680 x 4000 x 3250	37.94	41,491	98,317
GPT 41800	1350 to 2400mm	12730 x 4700 x 3900	65.33	70,452	166,836

Figure 42: Parameters of GPT (Ecosol 2015)

According to the Capture Efficiency shown in figure below, the removal of TSS, TP and TN can reach 80%. 45% and 45%. This is sufficient to reach the design water treatment target.

Pollutants	Capture Efficiency (Up to)
Gross Pollutants (>2000µm)	99%
Total Suspended Solids (TSS) (20 - 2000µm)	80%
Total Phosphorous (TP)	45%
Total Nitrogen (TN)	45%
Total Petroleum/Hydrocarbon	99%

Figure 43: Capture efficiency of GPT4750 (Ecosol 2015)

4.5.5. Bioretention Basin 1

4.5.5.1. General Design Description

Bioretention basin 1 serves $7160\ m^2$, it is possible to treat approximately 50% runoff of roads and pathway area. MUSIC was used for design and outcome checking. It shows that at least 85%, 40% and 55% of TSS, TP and TN can be removed by the bioretention basin design. Combined with Wetland and GPT treatments, desirable stormwater improvement can be made. If Bioretention basin 2 (with a $4240\ m^2$ contributing area) is implemented, with two basins more than 75% runoff from pavements can be treated in the project. The system can be more beneficial to stormwater treatment. The table below shows general design descriptions.

4.5.5.2. Design objectives

The design objective of Bioretention basin is to ensure the safety of drainage water that is released to receiving water body. It is desirable to reach the water treatment target with the 80%, 40% and 45% reduction of TSS, TP and TN respectively under minor runoff event, 5 yr. ARI, for pavements and roads (Department of Water: Government of Western Australia 2007, Melbourne Water 2007, Local Government Association of South Australia 2009).

4.5.5.3. Concept design

The Bioretention basins are designed according to the following criteria:

- Bioretention basin 1 area of $70\ m^2$ (maximum land area available);
- Maximum width of the Bioretention basin is to be $2\ m$;
- Extended detention depth is$200\ mm$. $(100\ m-\ 300m\ m)$;
- Filter media shall be a sandy clay loam;
- Coarse sediment forebay is needed as no space is allowed for grass buffer.

4.5.5.4. Site characteristics

The site characteristics of the Bioretention basin are:

- Urban, paved roads and footpaths;
- Typical overland flow slope of 1%;
- Red brown earth and alluvia soil (sandy loam to light clay loam) are dominated in design project;
- Catchment area: Roads, 0.5012 ha; footpath, 0.2148 ha;
- Fraction impervious is: road and footpath, 0.9.

4.5.5.5. Confirm size for treatment

The MUSIC model is used to check if the concept Bioretention area is able to reach treatment target. Figure 44 shows the layout of modelling.

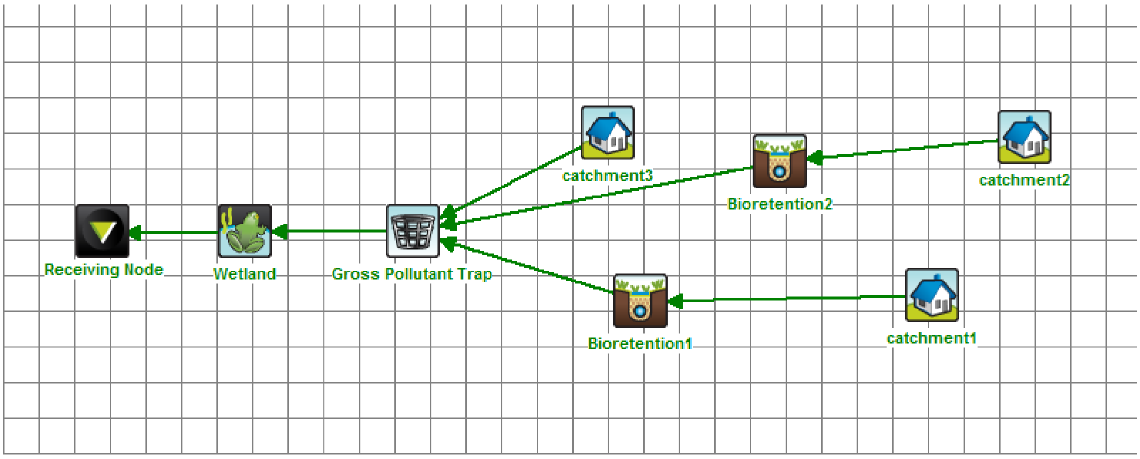

Figure 44: MUSIC modelling layout

The contributing area for bioretention area is 0.716 ha, area available 35m x 2m. By adjusting filter material depth and extended detention depth the results shows a 70 m^2 rectengular bioretention basin with 0.5m sandy clay filter material and 0.2m pounding depth. The reduction can reach 89.3%, 42.8% and 58.4% for TSS, TP and TN.

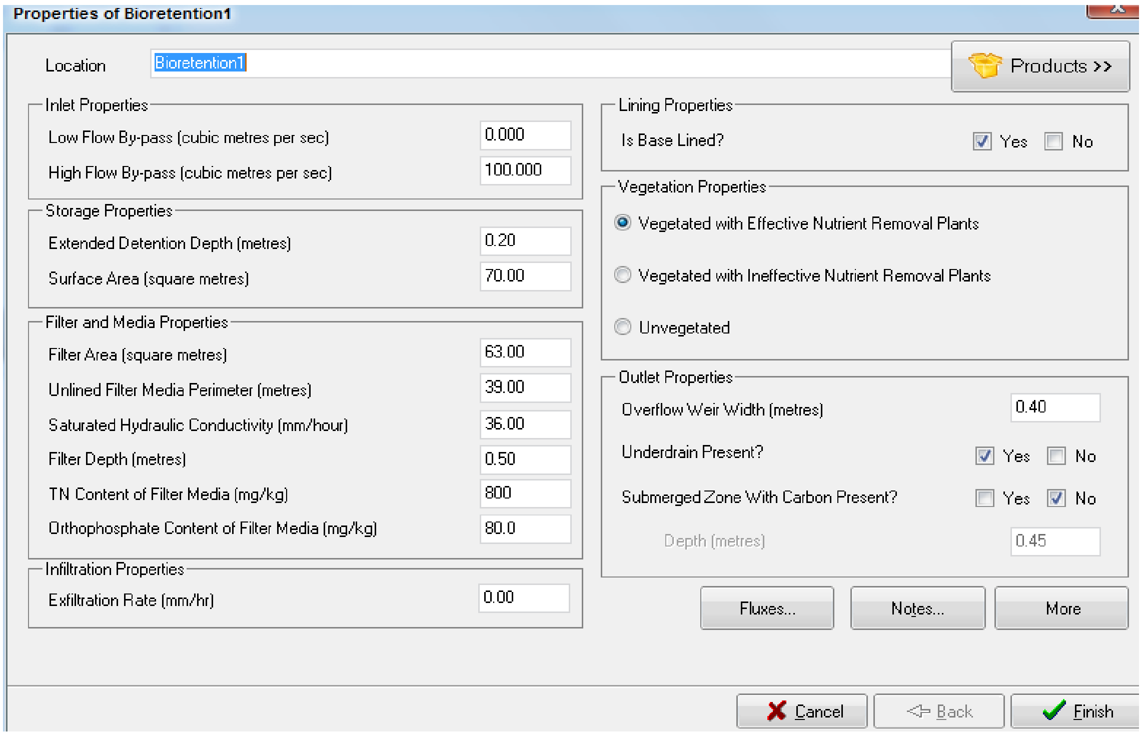

Figure 45: MUSIC modelling input for Bioretention basin 1

Mean Annual Loads - Bioretention1

	Inflow	Outflow	% Reduction
Flow (ML/yr)	1.94	1.79	7.3
Peak Flow (m3/s)	0.115	83.8E-3	26.9
Total Suspended Solids (kg/yr)	306	32.7	89.3
Total Phosphorus (kg/yr)	0.686	0.393	42.8
Total Nitrogen (kg/yr)	5.09	2.12	58.4
Gross Pollutants (kg/yr)	96.9	0.00	100.0

Figure 46: Mean annual loads output for Bioretention basin 1

The modelling indicates that only the reduction of TP in Bioretention basin 1 (42.8%) is slightly lower than target percentage (45%). Other pollutant reductions far exceed the design requirements. Considering the function of GPT and Wetland, and the limitation in land-use, size of Bioretention basin are confirmed:

Table 17: Dimensions of Bioretention basin

Bioretention basin	*Length* (*m*)	*Width* (*m*)
1	35	2
2	35	3

4.5.5.6. Estimating design flows

Major and minor design flows

Rational method is used in the project because the contributing areas are smaller than 1 ha.

The project is designed according to the following criteria; hence 100 yr. and 5 yr. ARI are used as major and minor peak flow rate.

- Local/ Arterial roadway – 1 in 5 year Average Recurrence Interval standard,
- Trapped low point in roadway – 1 in 100 year Average Recurrence Interval standard,
- The North Terrace low point adjacent to First Creek shall be designed to an ARI of 1 in 100 years.

Time of concentration (tc) and Rain Fall intensity

For paved road, standard tc=5min is used. The rainfall intensity (I) for 1 in 5 years is 83.6 mm/h and for 1 in 100 years is 182 mm/h.

Design runoff coefficient

Using similar method as in flow calculation section4.1.7 runoff coefficients for the pervious and impervious areas was calculated.

Calculation for 1 in 5 years and 1 in 100 years is shown below.

$$C_5 = F_5 C_{10} = 0.95 \times 0.9 = 0.855$$

$$C_{100} = F_{100} C_{100} = 1.2 \times 0.9 = 1.08$$

Peak design flows

Based on previously mentioned data, the flow rates for the two sub-catchments were calculated using the equation mentioned below.

$$Q = \frac{CIA}{360}$$

$$A = 0.716$$

$$Q_5 = 0.142 m^3/s$$

$$Q_{100} = 0.391 m^3/s$$

4.5.5.7. Maximum infiltration rate

The maximum infiltration rate represents the design flow for underdrainage system. The capacity of the underdrains needs to be greater than the maximum infiltration rate to ensure the filter media drains freely and does not become a 'choke' in the system. (WSUD Engineering Procedure)

A maximum infiltration rate (Q_{max}) can be estimated by applying Darcy's equation:

$$Q_{max} = KLW_{base} \frac{h_{max} + d}{d}$$

Where k = the conductivity of the soil filter (m/s)

W=the average width of the ponded cross section above the sand filter (m)

L= the length of the Bioretention zone (m),

h_{max}= depth of podding above the soil filter (m), d = depth of filter media (m)

Hence, Bioretention basin 1 $Q_{max} = 0.00084 m^3/s$

4.5.5.8. Inlet details

Coarse sediment forebay

Clogging with sediment and oil can occur during construction or after long-term use. The construction process, pre-treatment techniques, and maintenance requirements should be designed to minimise clogging.

Considering there is no space for installation of vegetation swale or buffer, coarse sediment forebay is designed and located near the inlet of Bioretention basin as a runoff pre-treatment. This sediment may smother vegetation and reduce infiltration to the filter media. The forebay should be designed to remove particles that are 1mm or greater in diameter from the minor ARI storm event and provide appropriate storage for coarse sediment to ensure desalting is required no more than once per year.

$$V_s = A_c R L_0 F_c$$

Where $A_c = Area\ of\ catchment$

$R = capture\ efficiency$ (Assume 80%)

$F_c = \ cdesired\ lean\ frequency$ (Assume 1 year)

$L_0 \ = sediment\ loading\ rate\ (1.6\ m^3/ha/yr$ For developed area)

Hence, required volume of forebay sediment storage $V_{s1} = 0.916\ m^3$

Depth of sediment forebay D = 0.5 m (max = filter depth + 0.3m = 0.8m)

$$A_{s1} \frac{V_{s1}}{\mathrm{D}} \approx\ 2\ m^2$$

Check:

$$R = 1 - [1 + \frac{1}{n} \frac{V_s}{\frac{Q}{A}}]^{-n}$$

Where $n\ =\ 1.35$ (using λ = 0.26)

$V_s = setting\ velocity\ 0.1$

When $A_{s1} = 5\ m^2, R_1 = 80.4\% > R = 80\%$

Hence, design $A_{s1} = 5\ m^2$

For Bioretention basin 1, size of coarse sediment forebay: L× $W = 2.5m \times 2m,\ d = 0.5\ m$

Flow width at entry

Minor runoff event 5 yr. ARI is checked in the entry design. Assume footpath and gutter longitudinal gradient$S = 1\%$. Queensland urban drainage manual (QUDM 2013) is used for checking the flow capacity under minor event design flowQ_5. Manning's equation (assume uniform flow) is used to estimate depth at design flow.

$$Q = \frac{1}{n} R^{\frac{1}{2}} S^{\frac{1}{2}}, n = 0.3(urban)\ ,$$

$Q_5 = 0.142m^3/s$, Depth of flow = 0.169m

Width of flow = 3.5m, velocity = 0.482 m/s

Inlet opening at entry

As both Bioretention basins are set between car park and footpath, inlets are set both above the ground and under pavement to allow all runoff from road and footpath to enter freely into basins. Length of inlet opening is calculated using broad-crested weir flow conditions.

$$Q_5 = CLH^{\frac{3}{2}} \rightarrow L = \frac{Q}{CH^3}$$

Where, C=1.7 (weir flow coefficient) , $H = flow\ depth\ from\ previous\ section.$

$Q_5 = 0.142m^3/s$, Depth of flow = 0.169m use H=0.2m

Therefore, L = $1.202m$

A 1.25m-wide opening are adopted Bioretention basin 1, as stormwater flow into the basin through the entry of car park where currently no kerb therefore, water can flow into coarse sediment forebay freely. The width of coarse sediment forebay is 2m which is satisfied for 1.25m opening.

4.5.5.9. Vegetation scour velocity check

Assume Q_5 and Q_{100} will be conveyed through the bioretention basins. Check for scouring of vegetation by checking that velocities are below 0.5 m/s during Q_5 and 1 m/s during Q_{100}.

Area =$width \times detention\ depth$ =$2 \times 0.2 = 0.4\ m^2$

$$V_{5\ average} = \frac{Q_5}{A} = 0.355m/s$$

$$V_{100\ average} = \frac{Q_{100}}{A} = 0.977m/s$$

Hence, $V_{ave\ 5} < 0.5\ m/s$ (OK), $V_{ave\ 100} < 1m/s$ (OK).

4.5.5.10. Sizing of perforated collection pipes

Perforations inflow check

Estimate the inlet capacity of sub-surface drainage system (perforated pipe) to ensure it is not a choke in the system.

Head = filter + ponding depth+ transition+ drainage+ 0.5pipe diameter

The following are the characteristics of selected slotted pipe

Clear opening = $2100\ mm^2$

Slot width = 1.5 m

Slot Length = 7.5 m

No. of rows, 6

Pipe diameter = 100 mm

Orifice flow condition occurs when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events), that is:

$$Q_{Pref} = BC_dA\sqrt{2gh}$$

Where C_d = orifice discharge coefficient (0.6)

B = 0.5 (50% of the holes are blocked)

H = depth of water above the centroid of the orifice (m)

A_0 = orifice area (m^2)

g = acceleration due to gravity (9.81 m/s^2)

H=0.2+0.5+0.1+0.2+0.05=1.05m

$Q_{Pref} = 0.00286\ m^3 > Q_{infiltration} = 0.00084m^3/s$,

Therefore, both tank $Q_{Pref} > Q_{infitration}\ (OK)$

One pipe with $Dia = 100\ mm$ is satisfied.

Perforated pipe capacity

The Colebrook-White equation is applied to estimate the flow rate in the perforated pipe.

$$Q_{pipe} = [-2(\sqrt{(2gD)})S_1 log_{10}[\frac{k}{3.7D} + \frac{25v}{P(\sqrt{2gDS_{1]}})}]A \ = \ 0.00267m^3/s \ ,$$

Where, $A = \frac{\pi}{4}D^2 = 0.00784\ mm^2$

$k = 0.007m$ (Assumption)

$v = 10^{-6}$

$pipe\ slop = 0.5\%$

$g = 9.81$

The capacity of this pipe is $Q_{pipe} = 0.00267m^3/s$, which exceeds the maximum infiltration rate of bioretention basin 1. Hence, $1\phi100$ pipe is adopted for perforated collection for the bioretention system.

Drainage layer hydraulic conductivity

A drainage layer is also required to convey treated water from the base of the filter media or saturated zone into the perforated under-drains. In this project, 200mm fine gravel drainage layer is used. A 100mm coarse sand transition layer is used surrounding the perforated under-drains to reduce the risk of washing the filtration later into the perforated pipe.

Impervious liner requirement

Impervious liners can prevent export of water from Bioretention basin into sensitive surroundings. It is used when the hydraulic conductivity ratio of filter material and surrounding soil is smaller than 10 times. Considering the geometry profile of project area, native soil are red brown earth and alluvia soil (soil types are sandy loam to light clay loam) with a hydraulic conductivity of 5×10^{-5}m/s. The selected filter material is sandy clay loam whose hydraulic conductivity is 1×10^{-5}m/s. Therefore, the conductivity of the filter materials is 5 times the conductivity of the surrounding soil of the project. Hence impervious liners are required for both Bioretention basins.

4.5.5.11. *High-flow route and bypass design*

Overflow (5 yr. ARI) is either conveyed by the road reserve or by connecting to an underground drainage system. To determine a grated overflow, the broad crested weir equation is used to determine the length of weir required and the orifice equation to estimate the area of opening required.

Free overfall conditions:

$$Q_5 = CLH^{\frac{3}{2}} \rightarrow L = \frac{Q_5}{CH^3}$$

Where, C= weir flow coefficient (1.7), $H =$ head above weir crest $(0.1m)$

$$L = 2.879m$$

$$\frac{1}{4}L = 0.719\ m$$

Drowned outlet conditions:

$$Q_5 = BC_dA\sqrt{2gh} \rightarrow A_0 = \frac{Q_5}{BC_d\sqrt{2gh}}$$

Where C_d = orifice discharge coefficient (0.6)

B = 0.5 (50% of the holes are blocked)

H = depth of water above the centroid of the orifice (0.18m)

A_0 = orifice area (m^2)

g = acceleration due to gravity (9.81 m/s^2)

$$A_{01} = 0.138m^2$$

$$\sqrt{A_{0_1}} = 0.743\ m$$

Drowned outlet conditions dominate, therefore a 900mm × 900mm grated pits are adopted.

Table 18: General Design Description

BR1	
Contributing area	$7160m^2$
Width	2m
TLength	35m
Filter area	$63m^2$
Extended detention depth	0.2m
Filter media layer	0.5m sandy clay loam
Transition layer	0.1m coarse sand max particle size 1.4mm
Drainage layer	0.2m fine gravel
Perforated pipe	Polyethylene pipe with 100mm diameter
Vegetation	Tall Sedge (*Carrex Appressa*) 5 plants per meter square; 350 plants in total
Coarse sediment forebay	L×W×D= 2m ×2.5m ×0.5m clean once a year
By pass pit	900 ×900×1100 grated pit

4.5.5.12. Outlet Connection

To connect the bioretention treated water to drainage system, a grated pit is installed downstream. Both treated water and bypass flow will be collected in the pit and through a pit outlet pipe flow into the side entry pit of stormwater system. All the pit outlet pipes are 375mm concrete pipes as that is the minimum pipe diameter for pipes under roads with 600mm clearance (Argue 2013). The outlet pipe link GP of bioretention system to the closest side pit of main stormwater system. This will allow the flow and the runoff of the water in the bioretention system to be acceptable and overflow will not endanger the entire stormwater drainage system.

The plan view (Figure 47) indicates how bioretention outlet pipe connect to the side pit 2. The outlet pipe runs a distance of 1.369 meters towards the east of the bioretention system. The pipe is then connected to the side PIT by a connection pipe that is a distance of 4 meters from

900x900 GP, then treated water enters stormwater pipe through side pit 2. This bioretention outlet pipe connection to the main drainage system pit is provided in detail in Engineering drawing HF-10. Detail calculation and checking can be seen in Appendix 1.2.3.

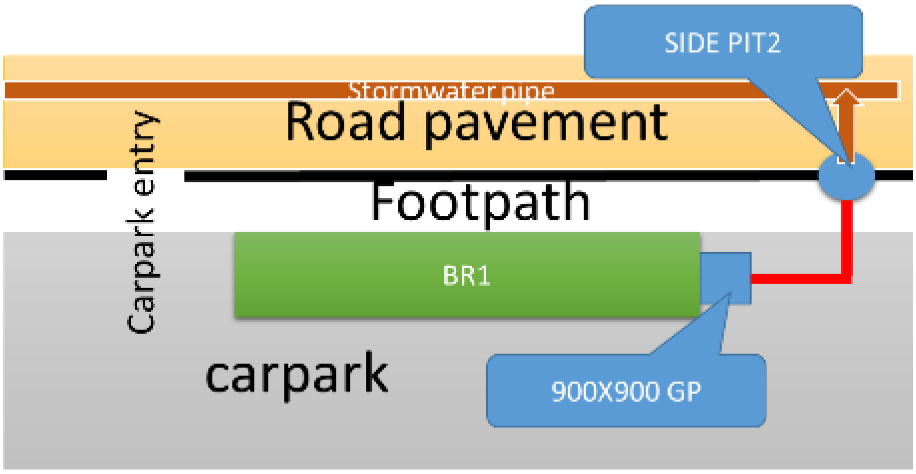

Figure 47: Bioretention 1 Plan View

According to WSA, velocity of pipe flow should be between $1m^2/s$ to $4m^2/s$ (WSA Code). A 0.1m vertical depth of part1 and 0.3m of part 2 are designed. The check of slope and velocity are shown in Table 19 below.

Table 19: Bioretention basin 1 Outlet pipe summary

BR1	Vertical length (m)	Horizontal length (m)	S (m/m)	V (m/s)	Start IL (m)	Finish IL (m)
Part1	0.1	1.369	0.073	3.718	35.80	35.70
Part2	0.3	4	0.075	3.767	35.70	35.40

4.5.5.13. Service Conflict Check

Cross Section view of the Bioretention basin 2 is shown in Figure 260. The connecting pipe is at 35.55 RL and descending at a slope of 7.2° towards Pit 2(Figure 53) this will allow the water to flow from the Bioretention system into the Stormwater drainage system. All the pipes are within the acceptable clearance hence no conflict with services.

4.5.5.14. Calculations analysis

Bioretention outlet pipe slope will heavily depend on the length between the outlet and the connection. Both bioretention basins were calculated in two different parts. The first part shows the slope calculation between the bioretention outlet pipe and the end of the same pipe, whereas part two is from the end of the first pipe towards the second pipe and connecting into the pits. As the horizontal distance between bioretention graded pits (GP) to drainage system GP is less, it contains a very high slope gradient, percentage, and Angle (Table 93). However, Bioretention basin 2 has a larger horizontal distance of 12 meters in length which recorded much lower slope characteristics than Bioretention basin 1. The slope steepness will not have a huge impact as it will only increase the flow rate which will not be a limiting factor. Increasing the slope also prevents any conflicts with services and ensures that there is a decent gap between the pipe and other neighbouring services.

4.5.5.15. Technical specification

Soil media specifications

Three layers of soil media are to be used, a sandy clay filtration media to support the vegetation, a coarse transition layer, and a fine gravel drainage layer.

Filter Media specification (500mm)

The filter media is sandy clay loam which needs to fulfil the requirements outlined in the AS1289 4.1.1 and meet the geotechnical requirements.

- Hydraulic conductivity: 36-180 mm/hr
- Particle size: clay 5 – 15 %, silt <30 %, sand 50 – 70 %
- Between 5% and 10% organic content, measured in accordance with AS1289 4.1

Transition layer specification (100mm)

Coarse sand material such as Unimin 16/30 FG sand grading or equivalent is required. A typical particle size distribution is provided below:

% passing	1.4 mm	100 %
	1.0 mm	80 %
	0.7 mm	44 %
	0.5 mm	8.4 %

Figure 48: Particle size distribution (Melbourne Water 2005)

Drainage layer specification (200mm)

The drainage layer is to be 5mm screenings.

Coarse sediment forebay

A sediment forebay is a small pool with coarse particles trapped, which prevents the clogging of bio-retention basins. The geometries of coarse sediment forebay is L×W×D= 2m ×2.5m ×0.5m.

Pipe specification

The inflow and outflow of the bioretention system can be seen as figure below. Both inlet and perforated pipe are 100mm polyethylene pipe.

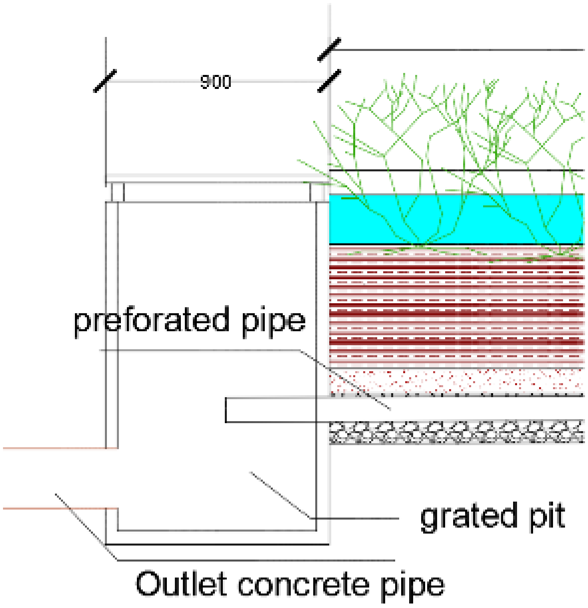

Figure 49: Outlet connection (Engineering drawing HF-106A)

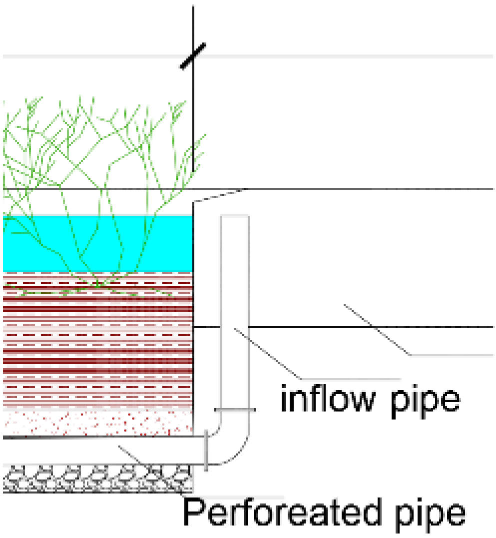

Figure 50: Inlet pipe connection (Engineering drawing HF-106A)

Inlet pipe

One 100mm diameter polyethylene pipe is used for inflow. It will be connected with a perforated pipe.

Perforated pipe

One 100mm diameter polyethylene pipe is used for perforated collection under the bioretention basin. The characteristics of slotted pipe including 1.5mm in width, 7.5 mm in length and 6 slot per row. Lay the pipe under the length of the Bioretention basin.

Outlet pipe

One 375mm diameter concrete pipe is used for outflow. It connects with outlet flow from grated pit with main drainage system.

Grated Pit

900 x 900 x 900mm grated pits are used for the bypass and high-flow route design. It will also be used as the connection of the bioretention basin and drainage system. The minimum invert level is 1.1m.

Vegetation

Tall sedge (Carex Appressa) is selected for both bioretention basins. Carex is a very hardy and useful plant for both wet and dry aspects in the garden. It is ideal for 'Rain Gardens' and suitable for loam to clay loam soil type. The density of planting is 5 plants per square meter (Melbourne water 2005).

Impermeable liner

Geotextiles are woven, matted or knited products constructed from synthetic fibres which are not susceptible to biodegradation. A non-woven geotextile fabric with a flow rate of > 4500 L/min/ sq. meter (e.g., Geotex 351 or equivalent) is selected in the project. Apply to the bottom and the sides of the basin.

Construction methods

The following is a typical construction sequence to properly install a bioretention basin. The construction sequence is more simplified. These steps may be modified by further construction planning:

Step 1: Existing utilities and boundaries of the proposed site are checked prior to any excavation.

Step 2. Temporary Erosion pretention and sediment control (E&S) controls are needed during construction of the bioretention area to divert stormwater away from the bioretention area until it is completed. Special protection measures such as erosion control fabrics may be needed to protect vulnerable side slopes from erosion during the construction process.

Step 3. Coarse sediment forebay should be excavated first and then sealed to trap sediments.

Step 4. Excavators or backhoes should work from the sides to excavate the bioretention area to its appropriate design depth and dimensions. Excavating equipment should have scoops with adequate reach so they do not have to sit inside the footprint of the bioretention area.

Step 5. Place geotextile fabric on the bottom and sides of the bioretention area.

Step 6. Install the perforated collection pipe, pack fine gravel (0.2 m drainage layer) with the pipe. Install grated pit, connect perforated collection pipe and drainage pipe with pit (Figure 51)

Step 7. Add approximately 100mm of the sand coarse transition layer between the drainage layer and the filter media layer.

Step 8. Deliver the filter media from an approved vendor, and store it on an adjacent impervious area or plastic sheeting. Apply the media 0.5m lifts until the desired top elevation of the bioretention area is achieved. Wait a few days to check for settlement, and add additional media, as needed, to achieve the design elevation.

Step 9. Prepare planting holes, install the vegetation, and water accordingly. Install any temporary irrigation.

Step 10. Place the surface cover in both cells (mulch, river stone or turf), depending on the design. If coir or jute matting will be used in lieu of mulch, the matting will need to be installed prior to planting (Step 9), and holes or slits will have to be cut in the matting to install the plants.

Step 11. Install the plant materials as shown in the landscaping plan, and water them during weeks of no rain for the first two months.

Step 12. Conduct the final construction inspection.

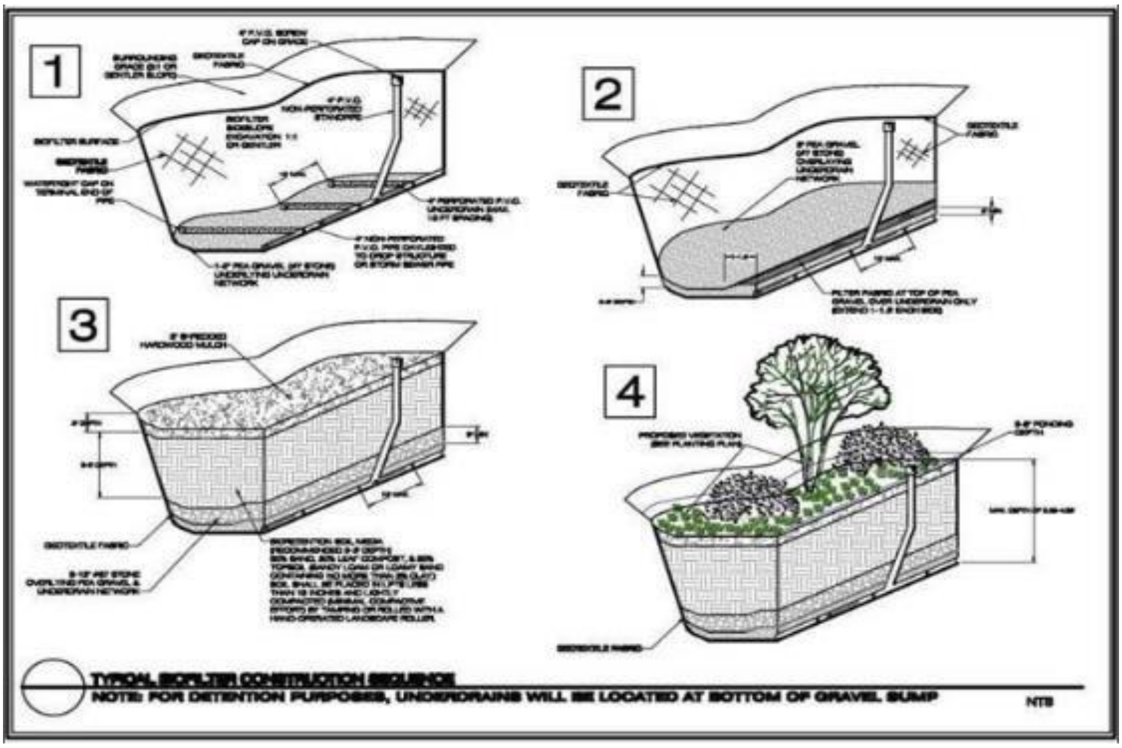

Figure 51: Typical Bio filter Construction Sequence (Melbourne Water 2005)

4.5.5.16. Cost

The costing of the bioretention basins are estimated according to Water Sensitive Urban Design – Greater Adelaide Region Technical Manual (2010).

As the table below, the costing for Bioretention basin 1 (cross-section:2m x1.05m) is $6,742.8.

Table 20: Estimated cost (per meter) for Bioretention basin 1

BR1 work description	Quantity	Unit	Rate	Cost
Excavate (2m x1.05m)	2.1	m^3/m	20	42
Supply and install geofabric liner	4.1	m^2/m	5	20.5
Supply and place under-drainage pipe (100 diameter)	1	m/m	13	13
Supply and place In and out flow pipe (100 diameter)	1	m/m	13	13
Supply and place gravel drainage layer	0.36	m^3/m	45	16.2
Supply and place coarse transition layer	0.18	m^3/m	45	8.1
Supply and place filter media (sand/grave soil)	0.9	m^3/m	55	49.5
Supply established Tall Sedge vegetation ground cover including planting, fertilizer and watering	2	m^2/m	19	38
Supply and install 900x900 x1100 pit	1	no	1104	1104
TOTAL	35	$1/m$		6,742.8

From previous section the cost for safety equipment is $916.

The total cost of construction Bioretention basin 1 is 6742.8+916= $7,658.88

4.5.5.17. *Task Specific Safety Assessment*

<table>
<tr><th colspan="2">Task Specific Safety Assessment Form</th></tr>
<tr><td>Department/Section: : Water</td><td>26/05/2015</td></tr>
<tr><td colspan="2">Brief Description : Bioretention system construction</td></tr>
<tr><td colspan="2">Excavation for coarse sediment forebay and Bioretention basin 1
Coarse sediment forebay installation
Geotextile liner installation
Perforated pipe installation
Soil media fill and compaction
Vegetation planting
Installation of grated pit</td></tr>
<tr><td colspan="2">Summary of major risks or hazards</td></tr>
<tr><td colspan="2">The alteration of existing green garden requires temporary support otherwise earthwork may collapse
Construction site is a confined space. And located adjacent to a road, and at traffic corridor that is in use by hotel car park and pedestrians. It may involve traffic accidents.
A person being trapped by the collapse of the excavation or falling into the excavation
The connection of basin and drainage system may involve a risk of water pollution.</td></tr>
<tr><td colspan="2">Mitigation strategies</td></tr>
<tr><td colspan="2">Conduct safety plan to identify hazards and assessing risks.
Staff training
Benching, battering, or shoring the sides of the excavation to reduce the risk of ground collapse.
Use barriers to separate pedestrians and set mobile plant to reduce the risk of collision.
Set markers and signals to inform the construction</td></tr>
<tr><td colspan="2">Safety equipment required & number</td></tr>
<tr><td colspan="2">Barricade: using 1800 mm BLACK STEEL PICKETS - SP18I steel pickets (20 pickets) joined by BARBED WIRE IOWA - BW250 chain wire (1 pack). 100+120= AUD 220.
Triton Crash Barriers TL3 classification hire No. 20 AUD50 Per day $ 70
Hazard lamp no. 5 price AUD 100</td></tr>
</table>

4.5.6. Bioretention Basin 2 (optional)

4.5.6.1. General design description

Following table shows the design for the Bioretention basins. Detailed design calculations are included in Appendix C section 1.5. Details of design can be seen in Engineering drawing HF-106B.

Table 21: Summary of Bioretention basin2 design

BR2	
Contributing area	$4240m^2$
Width	3m
Length	35m
Filter area	$98m^2$
Extended detention depth	0.2m
Filter media layer	0.3m sandy clay loam
Transition layer	0.1m coarse sand max particle size 1.4mm
Drainage layer	0.2m fine gravel
Perforated pipe	Polyethylene pipe with 100mm diameter
Vegetation	Tall Sedge (Carrex Appressa) 5 plants per meter square,525 plants in total
Coarse sediment forebay	L×W×D= 3m ×1m ×0.5m clean once a year
By pass pit	600 ×600 grated pit

Soil media specification

Three layers of soil media are to be used, a sandy clay filtration media (300mm) to support the vegetation, a coarse transition layer (100mm) and a fine gravel drainage layer (200mm).The filter media specifications, transition layer specifications, drainage layer specifications, pipe specifications, vegetation, impervious liner , layer specifications will be same as the Bioretention basin1.

Coarse sediment forebay

A sediment forebay is a small pool with coarse particles trapped, which prevents the clogging of bio-retention basins. The geometries of coarse sediment forebay is L×W×D= 3m ×1m ×0.5m.

Grated pit

600mmx600mm grated pit is used for bypass and high-flow route design. It also used as the connection for the Bioretention basin and drainage system. The minimum invert level is 1.1m.

4.5.6.2. Outlet Connection

Plan view of the Bioretention 2 is shown in Figure 52. The Pipe outlet is drastically less in steepness than Bioretention 1 with a 1:240 slope gradient. This is due to the longer distance between the outlets which is 12 meters. The pipe is then connected to pit 13 which will flow to pit 7 and reach the main stormwater pipe.

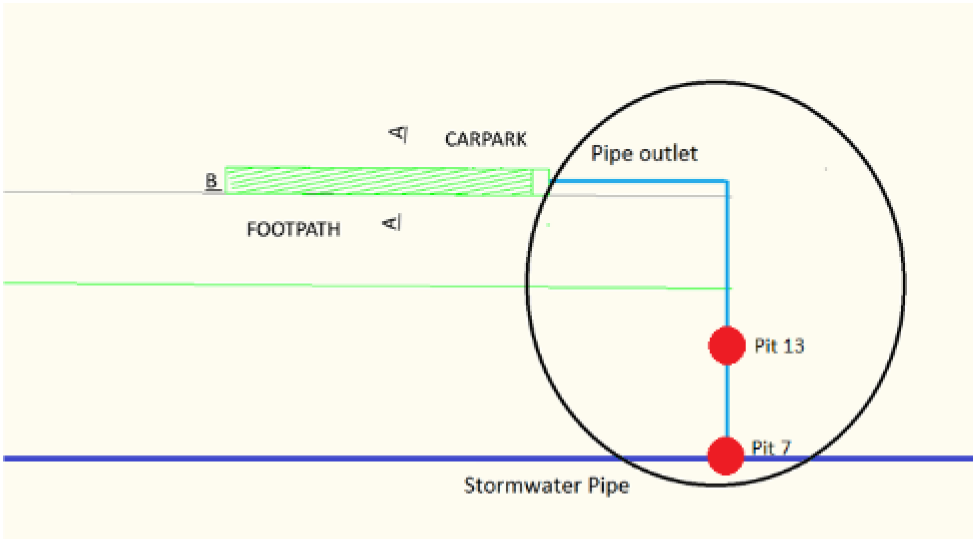

Figure 52: Bioretention 2 Plan View

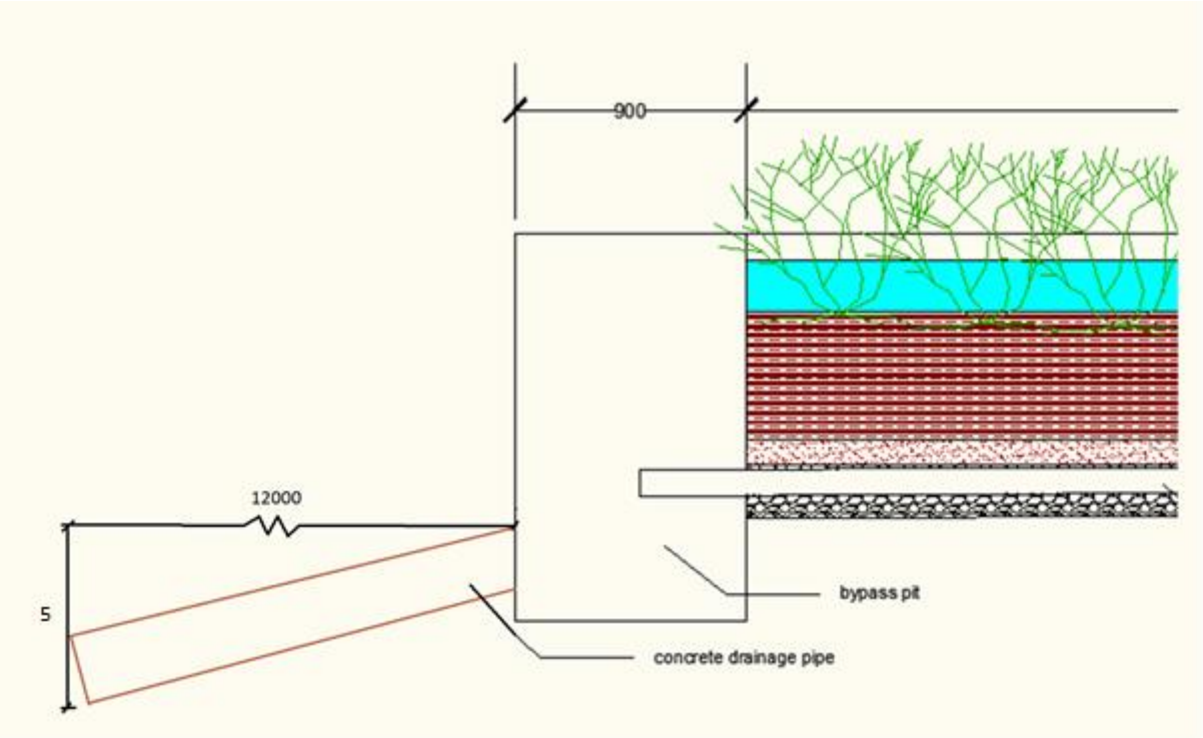

Figure 53: Bioretention 2 Long Section View

Similar to previous figures, Bioretention 2 long section view is shown in Figure 53.The Long section shows the 12 meter distance between the end of Bioretention 2 and the end of the outlet pipe, which is represented as broken line due to the distance being extremely long compared to other dimensions. This shows why Bioretention 2 is significantly less steep than the outlet in Bioretention 1.

The Figure 262 in Appendix 1 section 1.2.3 indicates that there are no conflictions with the services as the pipe passes through to the drainage stormwater system with sufficient clearance and horizontal and vertical clearances. The connection pipe from the middle of the retention basin 2 to the stormwater pipe is 16.5 meters in length and a minimum pipe diameter of 375mm was used.

4.5.6.3. Cost

The costing of the bioretention basins are estimated according to Water Sensitive Urban Design – Greater Adelaide Region Technical Manual (2010).

As the table below, the costing for Bioretention basin 2 (cross-section:3m x0.85m) is $9, 000, excluding the expense on fencing removing and rebuilding.

Table 22: Estimated cost (per meter) for Bioretention basin 2

BR2 work description	Quantity	Unit	Rate	Cost
Excavate (3m x0.85m)	2.55	m^3/m	20	51
Supply and install geofabric liner	4.7	m^2/m	5	23.5
Supply and place under-drainage pipe (100 diameter)	1	m/m	13	13
Supply and place In and out flow pipe (100 diameter)	1	m/m	13	13
Supply and place gravel drainage layer	0.56	m^3/m	45	25.2
Supply and place coarse transition layer	0.28	m^3/m	45	12.6
Supply and place filter media (sand/grave soil)	0.84	m^3/m	55	46.2
Supply established Tall Sedge vegetation ground cover including planting, fertilizer and watering	3	m^2/m	19	57
Supply and install 600x600 pit	1	no	970	970
TOTAL	35	$1/m$		9,000

4.6. Bill of Quantities

Bill of Quantity
Client: Tonkin Consulting
Project: North Terrace Drainage Design
Department: Water

#	Item name	Catalogue reference or special specification (if needed)	Unit	Quantity	Rate	Cost ($)
		Subject: Storm Water system				
1	750mm - reinforced concrete pipe	Concrete pipes - AS 4058 -1992 with rubber ring joints /Class 2	m	245	420	102,900
2	600mm - reinforced concrete pipe	Concrete pipes - AS 4058 -1992 with rubber ring joints /Class 2	m	535	290	155,150
3	900*900 pits – including side entry pits	900mm depth – Every additional 100mm in depth $68	Number	26	960	24,960
4	Precast concrete cover	For 900*900 pits /Side Entry pits - Trafficable	Number	15	230	3,450
5	GP cement	Briton cement (20 Kg bag per cubic meter) Cement: Sand ratio for Mortar is 1:3 (Cement Australia, 2012)	Number	2	75.3	150.60
6	Sand	80Kg of sand per cubic meter	Number	6	63.6	381.6
			Sub-total (1)			286,992.20
		Subject: Stormwater system construction				
1	Lifting Crane	Backhoe with rock breaker (CAT) with swift lifting clutches	Hourly	120	154	18,480
2	Safety Helmets	Assuming 6 people at site on each installation within a day.	Number	6	20	400
4	Safety Vests	Polyester	Number	6	4	80

#	Item name	Catalogue reference or special specification (if needed)	Unit	Quantity	Rate	Cost ($)
7	Labour	5 Individuals, Cost per week ($1,493 each)	Weekly	6	7,465	44,790
8	Spirit level	Stanley Fatmax 2m and 600mm combo	Number	2	100	200
9	Ball Peen hammer	Stanley 900g	Number	3	36	108
			Sub-total (2)			64,058
			Subject: Water Harvesting			
1	PW4000L tank	Rainwater Tank Directs, including inlet	number	5	950	4,750.00
2	BR5500ROUND tank	RENOVI, including inlet	number	8	895	7,160.00
3	8000 L tank	Rainwater Tank Directs, including inlet	Number	6	1506	9,036.00
4	CT9500L SQ tank	Rainwater Tank Directs, including inlet	Number	3	1535	4,605.00
5	Square-line Painted Gutter	DURAKOTE	m	22	16.62	365.64
6	Over-strap Bracket	DURAKOTE	Number	66	19.85	1,310.10
7	Standard Rainwater Head	DURAKOTE	Number	22	147.81	3,251.82
8	Downpipe First Flush Diverter	Water tank Site 100mm	Number	22	65.80	1,447.60
9	PVC Inlet Pipe	Holman 100mm	m	88	6	528.00
10	Tank Outlet Kits	Kerrimuir TAT024	Number	22	18	396.00
11	Reflux valve 100mm	RV 100	Number	22	250	5,500.00
12	600W Transfer Water Pump	Ozito	Number	22	119	2,618.00
13	Dual Check Isolation Valve	BVQ25-20-2NV	Number	22	79.75	1,754.50
14	25mm x 20m Sullage Hose	Holman	Number	22	84.9	1,867.80
			Sub-total (3)			44,590.46

#	Item name	Catalogue reference or special specification (if needed)	Unit	Quantity	Rate	Cost ($)
Subject: Bio-retention basin 1 (Near Royal Hotel Car Park)						
1	100mm pipe	Polyethylene pipe With a row of 6 L×W =7.5mm x 1.5mm slots	m	35	13	455
2	100mm pipe	Polyethylene pipe	m	3	13	39
3	Geotextiles liner	Polyethylene nonwoven geotextile	m^2	143.5	5	717.5
4	Carex appraessa	N/A	Number	70	19	1,330
5	Sandy clay loam	HD:36-180mm/hr, Particle size: clay 5 – 15 %, silt <30 %, sand 50 – 70 % Organic content: 5% and 10%	m^3	31.5	55	1,732.5
6	Coarse sand	Unimin 16/30 FG sand grading or equivalent is required	m^3	6.3	45	283.5
7	Fine gravel	5mm screening	m^3	12.6	45	567
8	GP cement	900x900mm Briton cement	Number	1	960	960
9	GP cover	900x900mm pit cover	Number	1	144	144
9	Black steel pickets	Northwire Pty Ltd Item: SP181	Number	20	5.61	112.2
10	Barbed wire lowa	Northwire Pty Ltd Item: BW-250	Roll	1	103.68	103.68
11	Triton Crash Barriers	TL3 classification	Number/14days	20	30	600
12	Hazard lamp	N/A	Number	5	20	100
13	Excavate	2m x1.05m x35m	m^3	73.5	7	514.5
			Sub-total (4)			7,658.88
			Total			**403,299.54**