NORTH TERRACE DRAINAGE DESIGN FEASIBILITY STUDY

PROJECT MANGEMENT TEAM

PROJECT MANAGER	HUGH BURGER
ASSISTANCE PROJECT MANAGER	DAVID ARGENT
WATER ENGINEERING MANAGER	CHRISTIAN SOMMERVILLE
ENVIRONMENTAL ENGINEERING MANAGER	FERGUS HAMILTON
STRUCTURAL/GEOTECHNICAL ENGINEERING MANAGER	JEREMY BEMMERL
URBAN DESIGN MANAGER	ZEDONG ZHANG
TRANSPORT ENGINEERING MANAGER	MATTHEW TURNER

Version Number	Purpose/Change	Name	Date
0.1	Document Structure	Hugh Burger	8/4/15
0.2	Geotech Addition	David Argent (9:05pm)	8/4/15
0.3	Urban Planning Addition	David Argent (9:56pm)	8/4/15
0.4	Water Addition	David Argent (9:20am)	9/4/15
0.5	Transport Addition	Hugh Burger (1:30pm)	9/4/15
0.6	Structural Addition	Hugh Burger (12:10am)	10/4/15
0.7	Environment Addition	Hugh Burger (1:00am)	10/4/15
0.8	Project Costing	Hugh Burger (4:00am)	10/4/15
0.9	Final Editing	Hugh Burger (10:00am)	10/4/15
1.0	Submission for Printing	Hugh Burger (2:30pm)	10/4/15

Table of Contents

i. List of Tables

ii. List Figures

1. Introduction

The City of Norwood Payneham and St. Peters is a metropolitan council, covering an area of 15.1km^2, 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.

Over the years it has become apparent that as a result of heavy rainfall events, 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, as is shown in Figure 1. 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 feasibility study and will require careful consideration and detailed evaluation to select viable solution(s).

2. Existing Condition

2.1. Water

The existing stormwater infrastructure in place on and relative to North Terrace is not adequate enough to collect and distribute the surface water in major storm events and as a result, the road is receiving a backlog of water, which in turn is pooling in the road's low points.

As can be seen in Figure 1, the existing number of side entry pits (SEPs) along North Terrace between Hackney Road and College Road is four. Two on the Southern Side and two on the Northern Side. Two grated pits are also located at the Hackney Road end of North Terrace. Between the most North-Easterly grated pit on North Terrace (within the project area) and College road, there is approximately 280 metres of road with no surface drainage infrastructure.

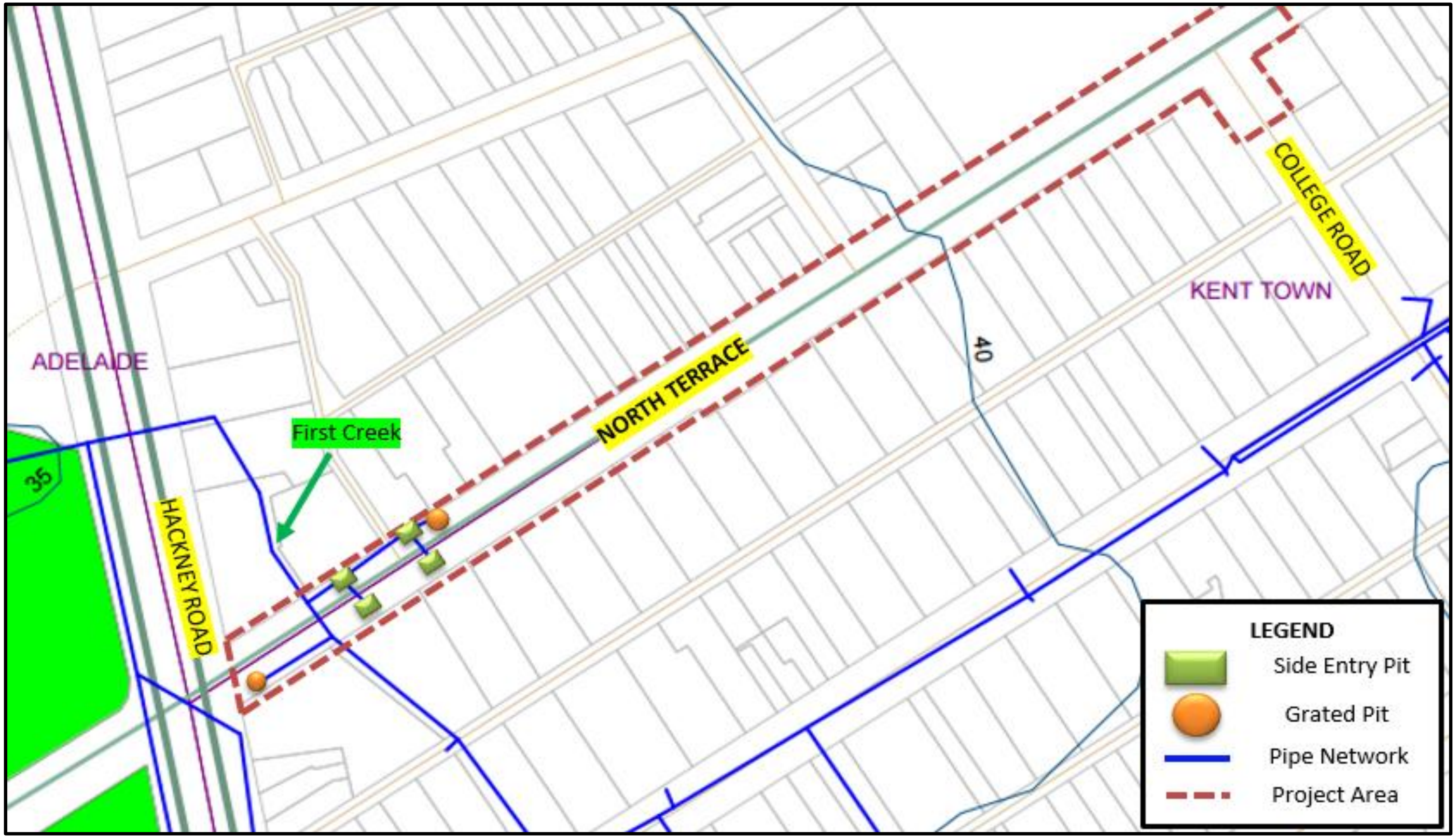


Figure 1 - Diagram of project area and its respective infrastructure (Hydro-Future Consulting, 2015

2.1.1. Design Flow Determination

In any stormwater related design project, the determination of runoff flow and the respective volumes are critical and should be worked out initially.

Design Flow Runoff for Entire Catchment Area

This section of the feasibility study will calculate the stormwater flow into the project area from the contributing catchment. Drainage options will later consider this flow rate in determining the most appropriate and feasible design option. The flow rate will be calculated using the following equation:

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

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

C=Runoff coefficient

I= the rainfall intensity (mm/hr)

A= the catchment area (ha)

The client requires that project be designed 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,
- the North Terrace low point adjacent to First Creek shall be designed to an ARI of 1 in 20 years

Since a large proportion of the design project lies in the trapped low point of the road way adjacent to the First Creek with very small proportion not classified as low points; an ARI of 1 in 20 years will be adopted in the preliminary calculation of the runoff flow. It is considered that all the catchment area will drain into the critical trapped low point area, therefore; an ARI will be used for all the catchment area not just the design area. This ensures that the project satisfies the requirements of the clients and is as conservative as possible.

However, if these design requirements could not be achieved due to significant costs associated with it (e.g. Service relocation), a request to compromise the standard design requirement will be negotiated with the council. A re-estimation of ARI values will then be recalculated to accommodate this compromise.

$$\boldsymbol{Intial\ ARI\ = 1{:}\,20\ Years}$$

Determining the Catchment Area

The first step in calculating the stormwater flow rate is determining the catchment area that contributes to the stormwater flow in the design area. This was estimated using Figure 2.

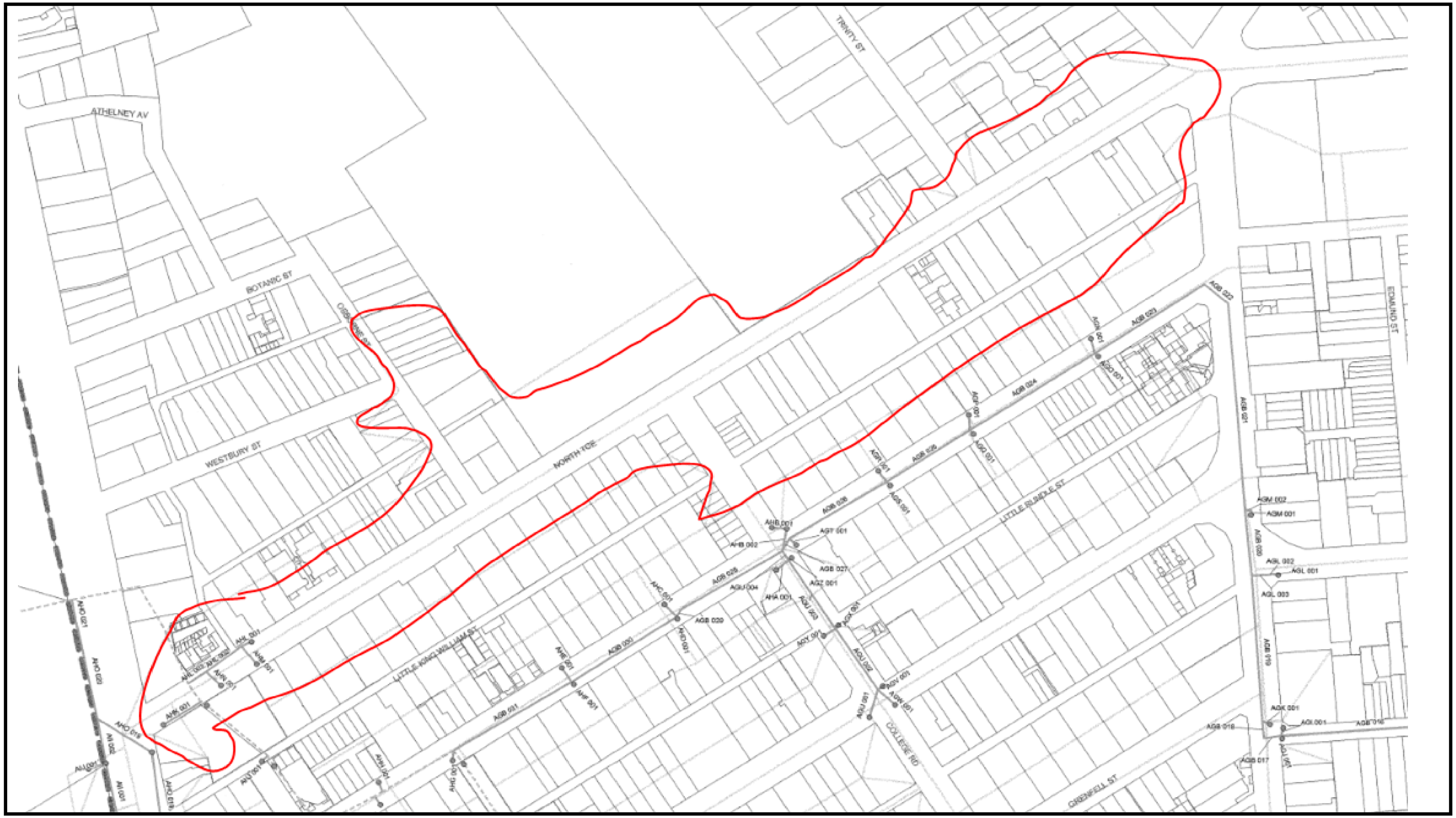


Figure 2 - Catchment Area for the Project (Tonkin Consulting, 2015)

The total catchment area was then divided into three separate sub-catchment areas to estimate the runoff from different sub-catchment that will be entered into the stormwater system via the side entry pits along the North Terrace.

The calculated areas are shown in Table 1 below:

Table 1 - Total Sub-Catchment Areas

Sub-Catchment	Total Residential/ business Area (ha)	Total Road Area (ha)
1	3.2	1.13
2	2.21	0.90
3	1.10	0.37

Determining the Pervious and Impervious area

After the total catchment area was determined, the total impervious (paved) and pervious (Green fill) areas were calculated. To assist in analysing the pervious and impervious areas of the catchment, google map images (Google Maps, 2015), similar to shown in Figure 3 were used. Based on this images we assumed the impervious area to be 90% and the pervious area to be 10% of the whole catchment as the total catchment area consists of a similar distribution of pervious and impervious areas in each allotment which can be observed in Figure 3 below. Therefore for all the three sub-catchments 90% of impervious area was assumed.

Figure 3: Percent Pervious vs Impervious in the Catchment Area (Google Maps, 2015)

Therefore, the total impervious area of the whole catchment can be calculated using the following equation with the figures from Table 2:

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

$$\rightarrow Total\ \%\ Pervious\ Area = 100 - Total\ \%\ Impervious\ Area$$

Table 2 - Total Paved and Pervious Areas

Sub- Catchments	Total Paved area %	Total Impervious area %
1	92.6	7.4
2	92.9	7.1
3	92.5	7.5

Rainfall Intensity

The travel time or time of concentration (t_c) is defined as the longest duration for water to flow out from the catchment outlet (Argue, 1986). The travel time for each catchment is defined as the critical storm duration and ultimately plays a major role in calculating the ARI. The time for concentration for paved area should be taken to be at least 10 mins (Ahammed, 2014). It is observed that all the houses in the catchment are very close to the road. Therefore, total time of concentration is taken as 20 mins for all the three sub-catchments.

$$\boldsymbol{Estimated\ t_c = 20\ mins}$$

The rain fall intensity of the design project was estimated using data from the Bureau of Meteorology data, 2015. The resultant IFD (Intensity- Frequency-Duration) relationships obtained for Kent Town is available in the Figure 4 & Figure 5 below.

Intensity-Frequency-Duration Table

Location: 34.925S 138.625E NEAR.. Kent Town Issued: 27/3/2015

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	45.9	61.4	83.6	99.7	121	154	182
6Mins	42.7	57.1	77.7	92.6	113	143	169
10Mins	34.5	46.0	62.2	73.9	89.7	113	133
20Mins	24.6	32.6	43.7	51.5	62.2	78.0	91.4
30Mins	19.6	26.0	34.6	40.7	49.0	61.2	71.6
1Hr	13.0	17.2	22.6	26.4	31.7	39.3	45.8
2Hrs	8.53	11.2	14.6	16.9	20.2	24.8	28.8
3Hrs	6.67	8.73	11.3	13.0	15.5	19.0	21.9
6Hrs	4.37	5.69	7.27	8.33	9.82	12.0	13.7
12Hrs	2.80	3.63	4.60	5.25	6.17	7.48	8.56
24Hrs	1.69	2.20	2.80	3.20	3.77	4.58	5.25
48Hrs	.957	1.25	1.61	1.85	2.19	2.68	3.09
72Hrs	.671	.882	1.14	1.31	1.56	1.91	2.21

(Raw data: 17.77, 3.79, 0.91, 35.01, 6.8, 1.72, skew=0.56, F2=4.47, F50=14.98) © Australian Government, Bureau of Meteorology

Figure 4: Intensity-Frequency-Duration Table for Kent Town (Bureau of Meteorology, 2015)

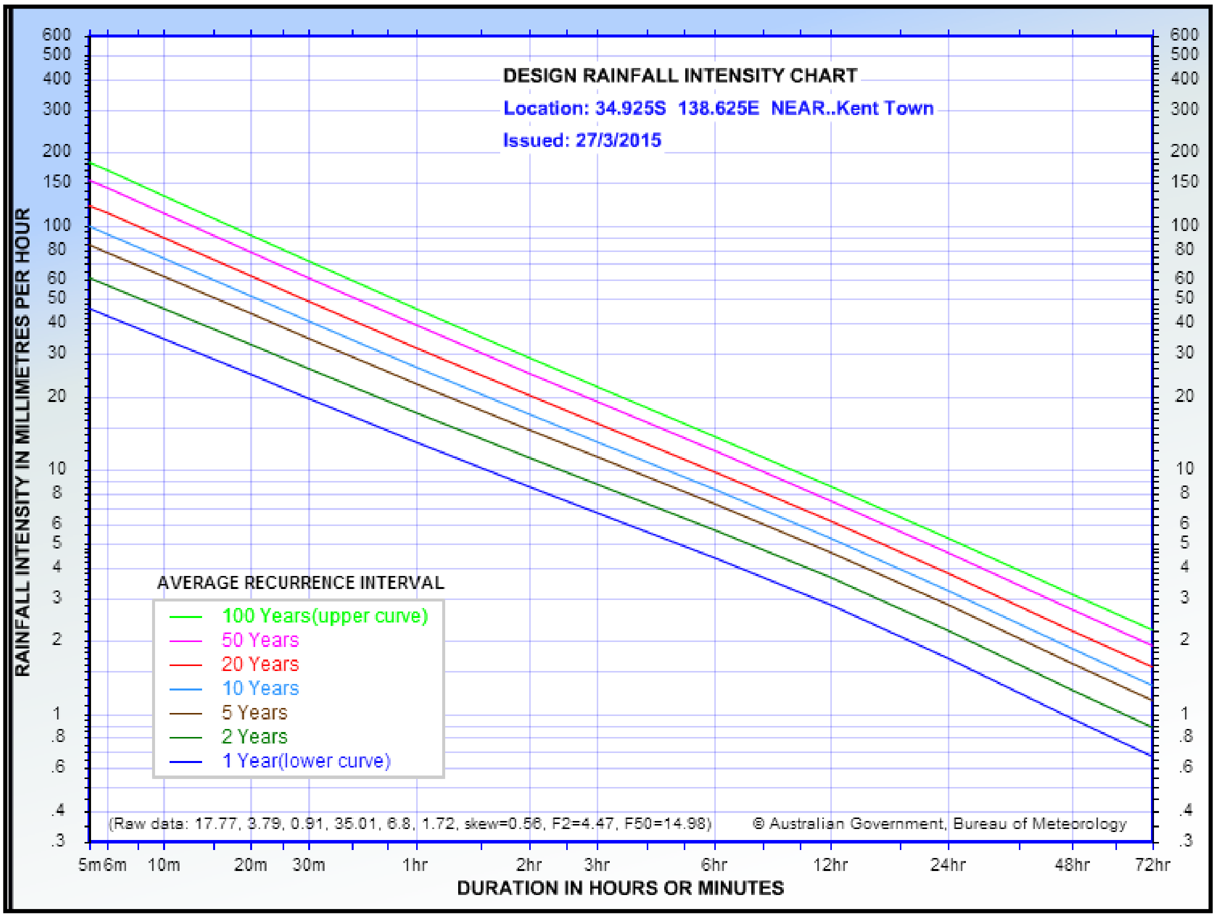


Figure 5: Intensity-Frequency-Duration Chart for Kent Town (Bureau of Meteorology, 2015)

Using the table in Figure 5 and the estimated value of ARI for 1 in 20 years and Time of concentration equal 20 mins; Rain Fall Intensity (I) can be predicted as 62.2mm/h.

$$\boldsymbol{I = 62.2mm/h}$$

Runoff coefficient

The run-off coefficient (C_{10}) 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 C_{10} = 0.9 while for residential land use C_{10} = 0.1 is assumed for Southern Australian region as shown below in Figure 6 the aforementioned runoff coefficient has to be multiplied by a frequency conversion factor F_y, as the design ARI is higher than 10 years. (Argue, 1986).

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

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

Surface Classification	Northern Aust. zone	Southern Aust. zone
First grade connected paved areas: - roadways) - roofs)	C_{10} = 0.90	C_{10} = 0.90
Second grade connected paved areas, e.g. - sealed carparks,) driveways, paved) outdoor areas,) etc.)	C_{10} = 0.75	C_{10} = 0.75
Unconnected paved areas) and) Pervious areas:) - mixed with paved areas) as in residential land) use) - major urban open space) areas, parks, etc.)	C_{10} = 0.70	C_{10} = 0.10

Figure 6 - Runoff coefficient values (Argue, 1986, Table 5.3 on pg 31]

FREQUENCY CONVERSION FACTOR F_y

ARI(years)	1	2	5	10	20	40	60	80	100
Conversion factor, F_y	0.8	0.85	0.95	1.00	1.05	1.13	1.17	1.19	1.20

Figure 7 - Frequency Conversion factors (Argue, 1986, Table 5.5 on pg 32]

Using designated values from Figure 6 and Figure 7, the runoff coefficients for the pervious and impervious areas within the three 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 calculated runoff coefficient values for both pervious and impervious areas, the weighted runoff coefficient for each sub-catchment was calculated using below mentioned equation.

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

Therefore the weighted runoff coefficient for each sub-catchment was calculated (Appendix A1. Design Flow Determination) and can be seen in Table 3 as follows:

Table 3 - Weighted Runoff Coefficients

Sub-Catchment	Weighted Runoff Coefficient (C10)
1	0.88
2	0.89
3	0.88

Runoff/ Design Flow (Q)

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

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

Therefore the total design Runoff flow rate is as follows in Table 4:

Table 4 - Flow rates of each sub-catchments

Sub-Catchment	Flow Rate (m^3/s)
1 - Q_1	0.662
2 - Q_2	0.475
3 - Q_3	0.223

Design $\boldsymbol{Q = Q_1 + Q_2 + Q_3 = 1.36 m^3/s}$

Relevant calculations are included in Appendix A1. Design Flow Determination

2.1.2. First Creek Site Inspection

An onsite visit was made to inspect the current state of First Creek and to assess the location of any potential areas of instability. It was found in many locations that retaining walls had already been installed and several photos were taken to show the cross sections that exist within the creek. Figure 8 shows the location of the photos taken on site.

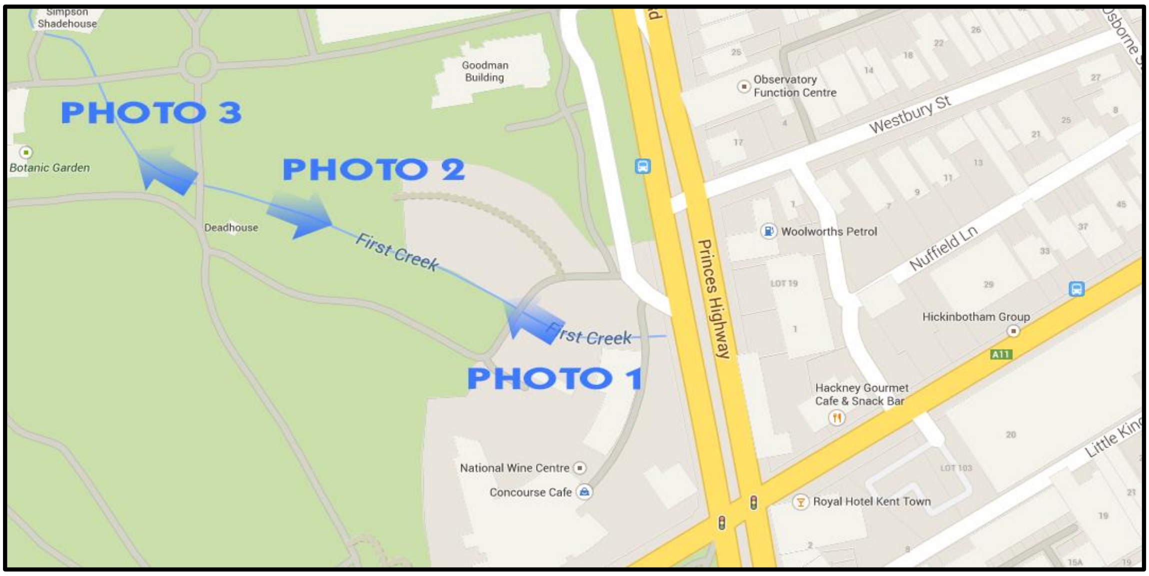


Figure 8: Location of Photos Taken of First Creek (Hydro-Future 2015)

Figure 9 has been taken just outside of the Adelaide Wine Centre and shows the existing embankment to the left as well as an existing stone and concrete retaining wall to the right.

Figure 9 – First Creek, note the Concrete and Stone Retaining Wall (Hydro-Future, 2015)

Figure 10, below, shows the approximate dimensions of this section of the creek, closest to the Wine Centre.

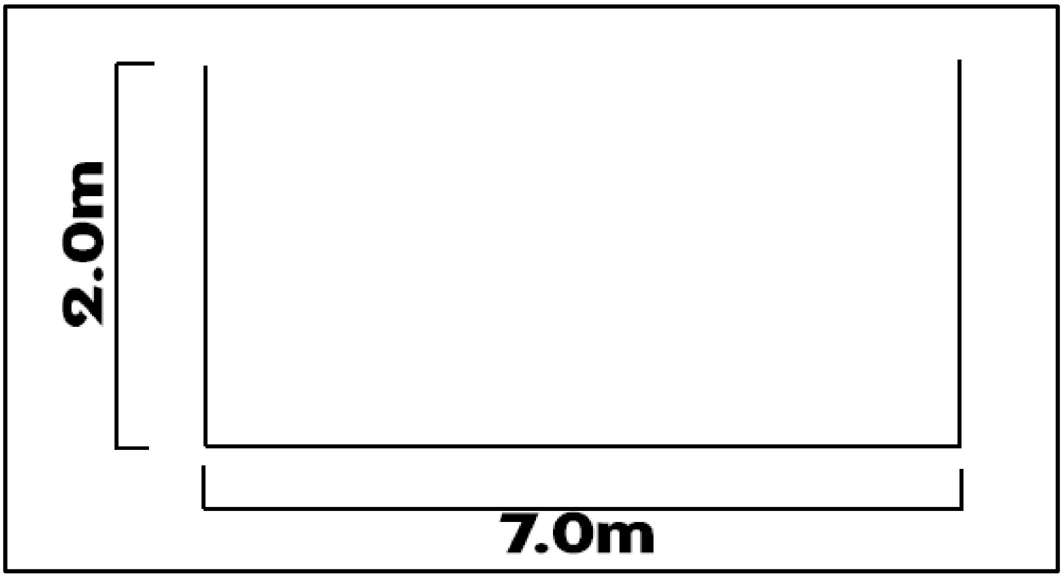


Figure 10 - Approximate dimensions of First Creek near the Wine Centre

Figure 11 is facing towards the Wine Centre highlighting the differences in cross section of the creek bed.

Further downstream the creek width increases as seen and a retaining wall is present on the right side, with the creek bed layered with large quarry rocks. These rocks work to reduce flow, trap a percentage of suspended solids, stabilise the creek and in-turn reduce erosion of the river banks. A small amount of vegetation can be seen in this picture, which improves water quality and will complement the rocks to reduce flow and catch suspended solids. A noticeable amount of rubbish was seen in this section of the creek at the bed level on the day of this inspection.

Figure 11 - Photo of First Creek showing wider channel and rock-lined bed

The approximate dimensions of this section of First Creek are depicted in Figure 12 below and show the sloped sides of the banks.

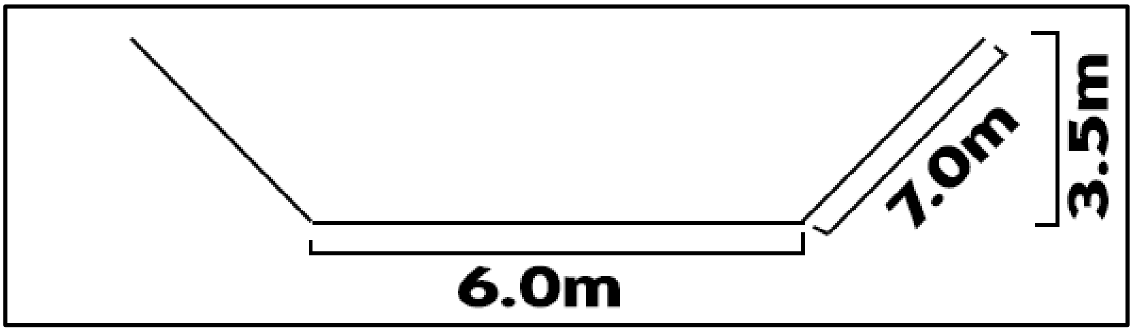


Figure 12 - Approximate wide channel dimensions of First Creek

The last photo (Figure 13) taken of First Creek is at the North-Western bend, about 600 metres from Hackney Road with a smaller channel height, suggesting this section of the creek may flood during large storm events.

The bank on the left hand side has visible erosion, most likely due to moderate volumes of water passing through this section of the creek. A retaining wall can be seen on the right hand side of the creek and appears to be supporting the vegetation well. Figure 14 shows the approximate dimensions of the section of First Creek in this location.

Figure 13: Photo of First Creek Showing Erosion

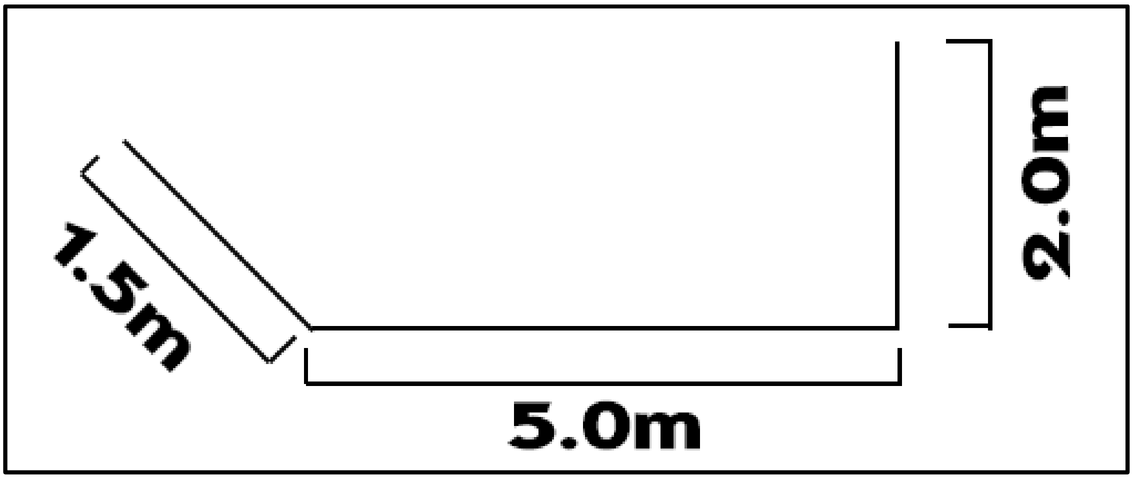


Figure 14 - Approximate minor channel dimensions of First Creek

2.2. Environment

Figure 15 below is an extract from the SA GOV Atlas site that identifies the land use in the project area. The majority of the area appears to be retail commercial businesses with a school located on the north eastern side of North Terrace. The botanic gardens and the road gardens are also located close to the project area and could potentially be affected by the construction process.

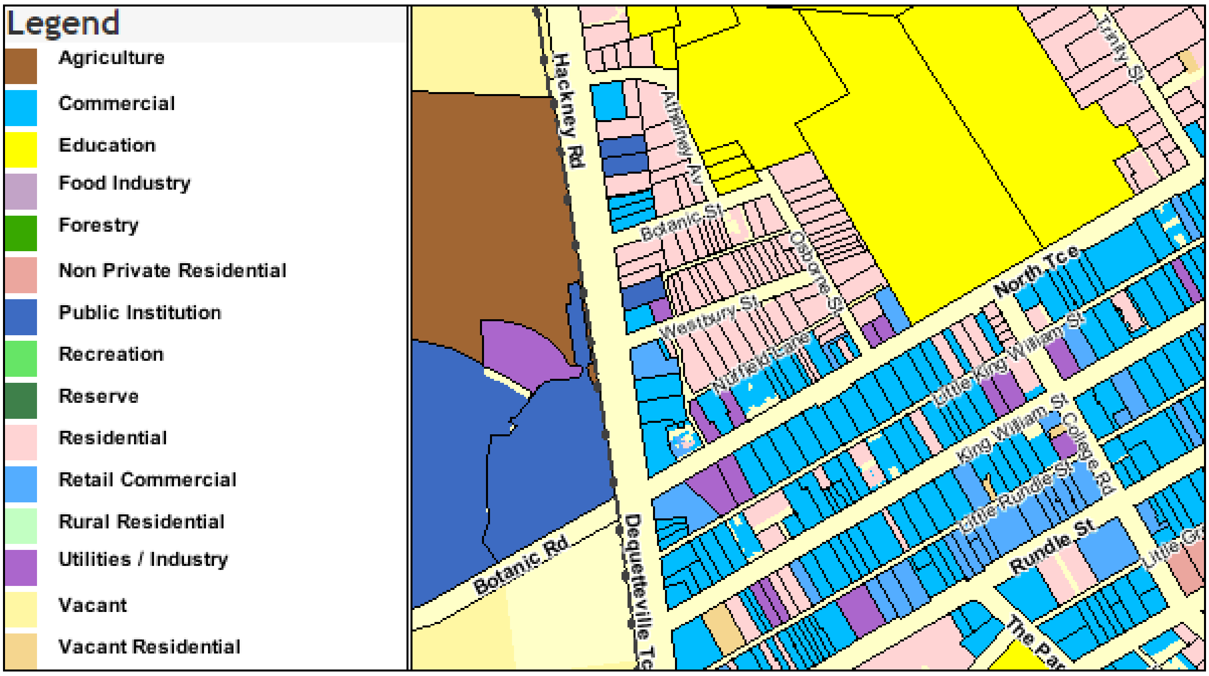


Figure 15- Project Location Land Use Diagram (SA GOV, 2015)

The infrastructure currently in this location comprises of footpaths, retail commercial buildings, driveways and services including street lighting and drainage. There is a sandstone arch culvert located over First Creek that is approximately 150 years old. It has been noted that a number of sandstone bricks from the culvert are missing and may need replacing to ensure that the culvert can be used for the proposed design. There is a diverse selection of building stones used throughout the subject section which is consistent with the building materials of the surrounding streets in the City of Norwood, Payneham and St. Peters which creates a visually appealing street with historic appeal.

2.2.1. Stormwater

The current stormwater system collects water from both sides of North Terrace as well as the surrounding catchment area. The system then transports the water to First Creek; it is understood that at present there is a gross pollutant trap that filters water as First Creek enters the River Torrens, currently there is no system in place to improve water quality from the project area to First Creek. Therefore, the current water quality is based purely on the litter, rubbish and other pollutants that may be deposited on the road.

2.2.2. Vegetation

The existing vegetation in the project area is minimal, there is no solid median in the centre of the road and there are multiple driveways which prevent a significant amount of vegetation along the footpath. A number of juvenile trees are placed at regular intervals along the southern side of North Terrace whereas the north side has fewer trees planted, these trees are of reasonable size and health.

2.2.3. Traffic/Road

The road is an undivided dual carriageway that provides adequate walkways on either side. This section of North Terrace has an Annual Average Daily Traffic (AADT) of approximately 34,200 vehicles and is usually busy from 5am until 3am as it is one of Adelaide's key arterial roads providing access both in and out of the city centre to the North-Eastern Suburbs. This is a significantly busy road during peak hour periods, therefore a traffic management plan will be created by the Transport Engineering team to ensure that traffic flow has minimal interruptions.

2.2.4. Native Fauna

The project location is a relatively small area that is almost entirely paved and does not cater for native wildlife. First Creek runs through this section underground, emerging at the Adelaide Botanic Gardens. The only vegetation along this section of road is trees, these are primarily aesthetic as the large volumes of traffic would discourage fauna. However, these trees will still need to be inspected for any bird's nests so that construction for the project can be done as far away as possible to ensure the breeding cycles are not disturbed.

2.2.5. Waste and resources

Figure 15 shows that North Terrace is the main access point for a number of businesses in Kent Town for the general public, including residents and everyday commuters. A site investigation determined that there is currently two rubbish bins along the footpaths in this location with one bin located next to a bus stop. Regular services including side entry pit cleaning, street sweeping and 'Autumn Leaf' pick up services are currently operating in the area.

2.3. Transportation

This project requires a great deal of traffic management preparation as the project involves major construction works on an arterial road, on the outskirts of Adelaide's CBD, North Terrace, Kent Town; usually busy from 5am in the morning until 3pm.

One of the most important considerations for this project, during the construction phase, is appropriate traffic management planning, which will look at changes to traffic flows and potential delays for commuters. The transportation team will assess all design options, to minimise any slow traffic flows causing delays, and to safely manage traffic and pedestrian movements during the construction period. Although delays in traffic may be unavoidable, the transportation team aims to provide effective traffic management strategies that not only minimise potential delay but maintain a high level of safety for all road users and surrounding communities alike.

Safety will be of the upmost importance during the project for workers, motorists, pedestrians and public transport users alike. It is key that safe working environment are maintained to ensure traffic management during the project is run successfully, even if speeds need to be reduced. Although this would seem to contradict the aim of minimising delays to traffic flows, it will in fact help with traffic control, as at higher speeds accidents are more prone to happen, which can lead to greater delays. Cost is always a key driver in projects and will be a key factor in the recommendations for transport management. Cost will, however, carry little weighting in comparison to safety

Further information regarding the existing condition of the transportation aspects of the study area is discussed in Section 7.3.

2.4. Geotechnical

This section investigates the Geotechnical considerations for the project area. We consulted councils, government organisations and geotechnical engineering contractors in an attempt to gain definitive information on the subsurface soil conditions and groundwater levels, however we were unsuccessful. Due to the lack of geotechnical borehole data Hydro-Future has relied on published literature and geotechnical borehole data from surrounding areas to present our findings.

The geotechnical research objectives are, but not limited to:

- Investigating and reporting upon the regional soil profiles to determine the geological conditions located within and around North Terrace.
- Investigation and reporting on surrounding borehole data to understand and classify the subsurface soil conditions.
- Analysis of the sub-surface soil layers including relevant soil properties which will be used to create a geotechnical model that can be used in both the feasibility phase and detailed design phases of the project.
- Investigating and reporting upon geological cross sections to determine where large quantities of soil and rock are located.
- Location of the groundwater table.
- Understanding of the location of old concrete slabs which were used when the old tram system was operational.
- Investigation of trench stability, including a cost analysis.
- Identifying any areas requiring a retaining wall including a visual-tactile assessment of the soil.
- Investigating the various retaining wall configurations and associated materials which may be feasible for the final design, including a cost analysis.

2.4.1. Regional Soil Deposits

Based on the Soil Association map of the Adelaide Region (Figure 16, 1989) the two major soil profiles in the project region are considered to be consistent with Red Brown Earth Type 5 (RB5) and Alluvial Soil (AL). The location of relevant borehole data with respect to the two main soil deposits and the site location are also outlines below in Figure 16.

The RB5 soil profile, which encompasses the project location, is also prevalent through the southern area of the central business district. It can be seen to include both the borehole location and the site location which may represent a close correlation between the regional and borehole soil profiles.

The AL soil profile, which is also found in the project area, is encountered in close proximity to the current location of the River Torrens, encompassing the entire Adelaide botanical gardens. It can be seen that the borehole location clearly falls within the AL region, and have used some borehole data in this location to assist in gaining a better picture of the AL soil profile in the project area.

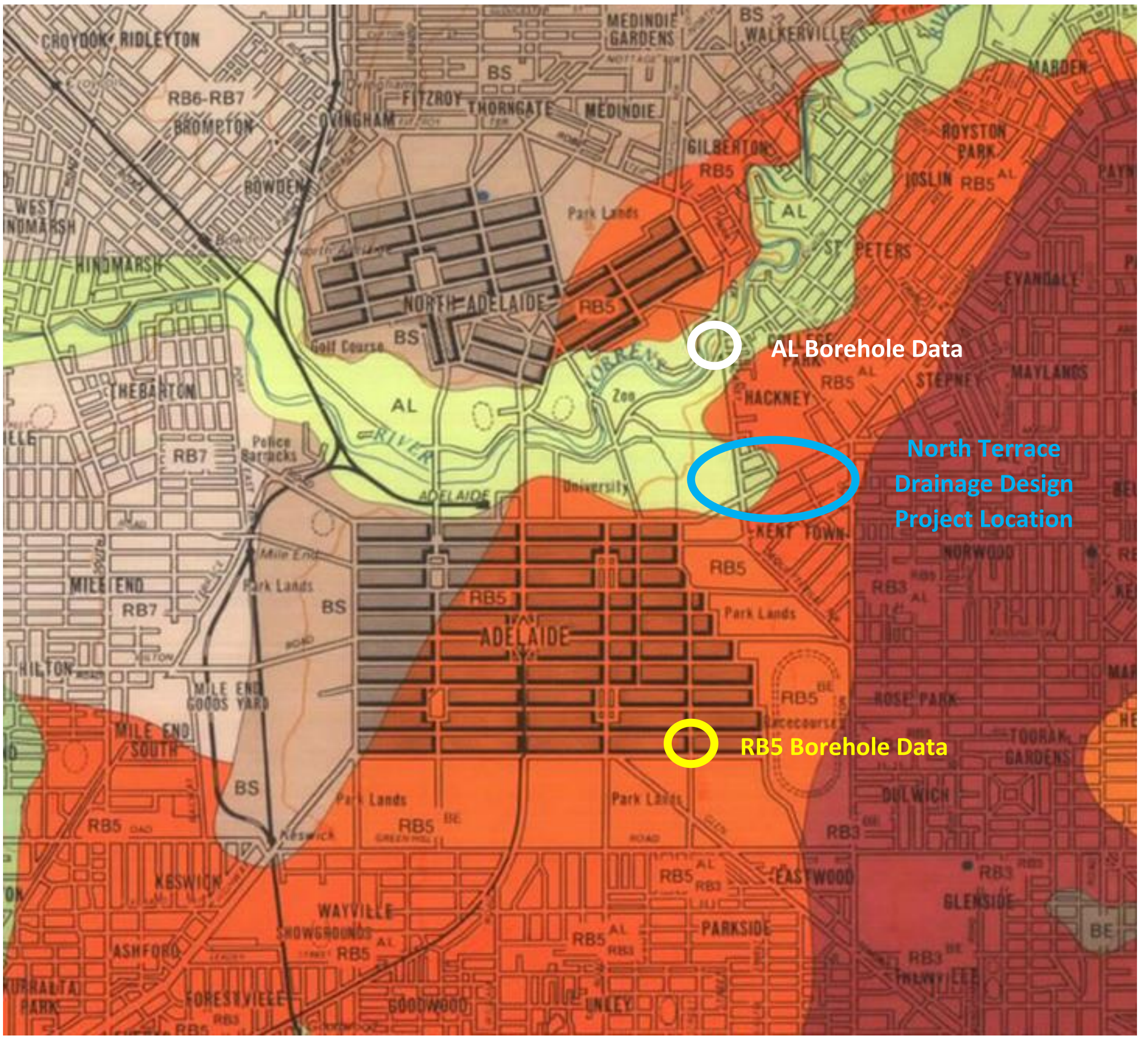


Figure 16: Soil Association Map of the Adelaide Region (GSSA, 1989)

From the Regional soils maps, alluvial soils are likely to be present along the majority of North Terrace between the Royal Hotel Kent Town and North Terrace Tyres which covers approximately 275 meters of the Project Area. This area coves the location of the sandstone arch culvert, the concrete box culverts and a majority of the road in which the stormwater system may be installed.

The RB5 soil group is likely to represent the remaining area, Figure 17 highlights the location of each soil group. The area which RB5 encompasses will also include a large region of road which the stormwater pipe may be constructed as well as the area of land which St Peters College is currently occupying. The area of land which contains St Peters College is of high interest for water sensitive urban design features throughout this feasibility study which may be directly affected due to the composition of RB5.

Although the soil maps for the region indicate that two distinct soil profiles are likely to be present, the exact arrangement of these soils cannot be understood, without an intrusive site investigation, in the form of either boreholes or test pits. The boundaries of the soil deposits shown in Figure *16* does not represent a sudden change in soil composition, with a large region of unconformity. It does give a rough estimate of where we expect a change in soil composition.

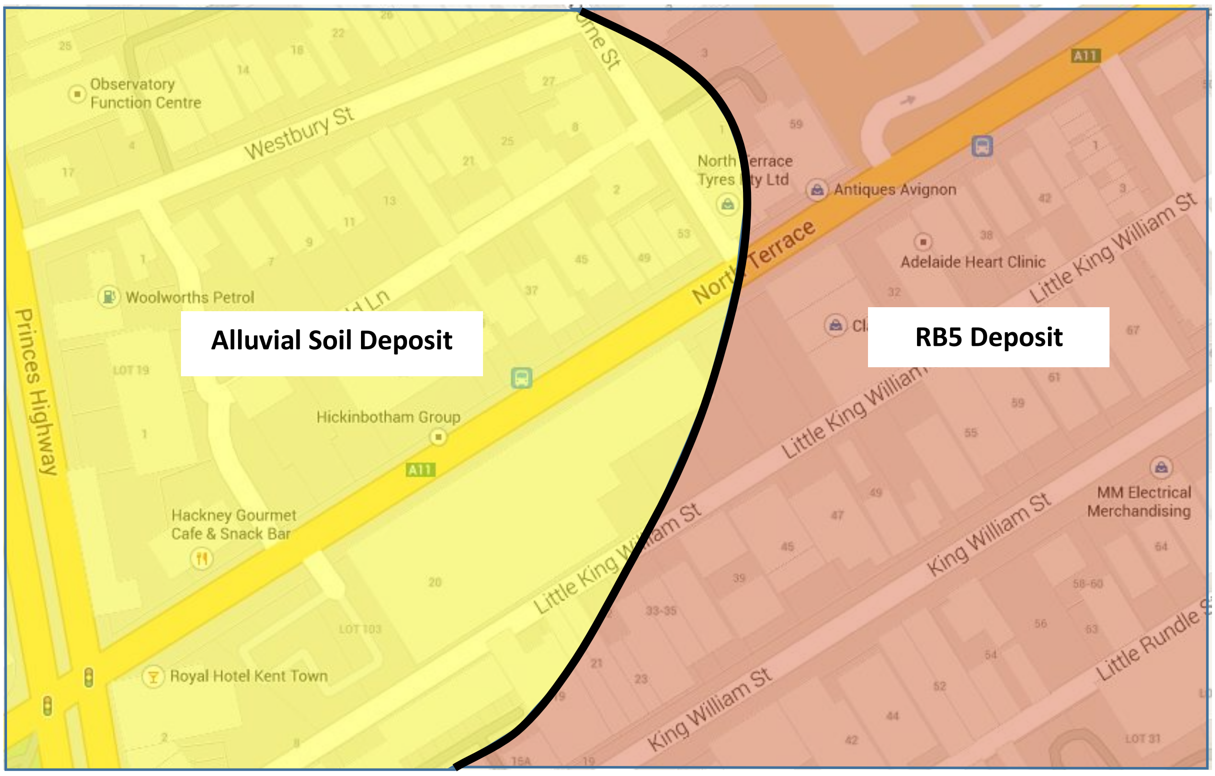


Figure 17: North Terrace Soil Deposits (Image Source: Google Maps)(Hydro-Future 2015)

Table 5 gives information on the minor soil types that are likely to be present within the project area. A detailed explanation of the minor soil compositions are listed in the following sections of the report.

Table 5: Region Soil Profiles (Taylor, 1974)

Soil Region	Dominant Soil Type	Minor Soil Type
Alluvial	AL	SA, RB3a, RB9, EMS
Red Brown Earth	RB5	RB3, RB9, RB5a, AL

2.4.2. Soil Profiles

Using the related literature by Taylor et al. (1974), which corresponds to the Soil Association Map of Adelaide a soil profile for each deposit, RB5 and AL, has been analysed to determine the general structure of the respective soil groups which may be encountered. Further details of the soil profiles have then been extracted from borehole data to produce a detailed description of the subsurface soil layers.

Red Brown Earth

Red brown earths are a predominant soil group endemic to the Adelaide region, and will generally range from sandy clays to high plasticity clays, of a stiff to hard consistency. The RB5 soil profile is often encountered on land with minimal slopes or elevation (30 to 60 meters) and around currently active or past creek lines (Taylor et al. 1974). This shows high agreement with the location of the project site which is surrounded by geological land marks including the Torrens River and first creek. The elevation of this area is regarded as reasonably flat as the project location is next to the Adelaide central business district which aligns with the elevations in which RB5 may be found.

Soil Composition

The two identified soil groups which are associated with the North Terrace Stormwater Project have been further characterised into dominant and minor compositions. The dominant soil group refers to the soil compositions which are readily occurring throughout the red brown earth regions of the soil map and are likely to consist of sands and clays. The minor category refers to soil compositions which will occur within the designated regions of RB5 however will not occur often or only in small amounts. The soil compositions of the dominant and minor categories can be seen below (Taylor et al. 1974):

Dominant:

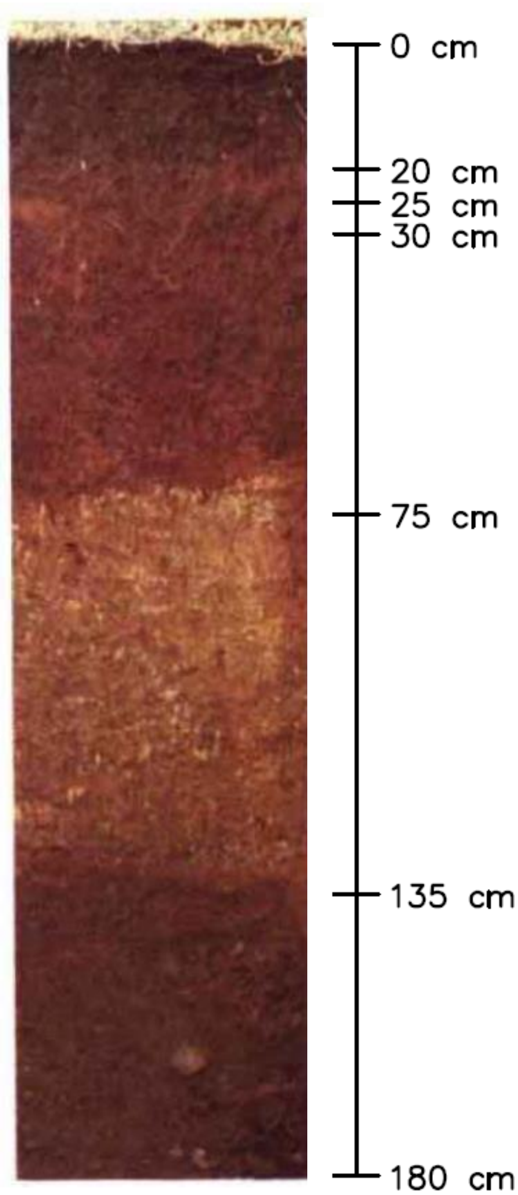
- RB5: Red-brown sandy clay soils with granular structure over clay with variable lime.

Minor:

- RB3: Heavy red-brown clay soils with prismatic or blocky structure over clay with variable lime.
- RB9: Mottled silty clay over brown silty clay with granular structure, slight lime, becoming sandy with depth.
- RB5a: Brown clay or sandy soils with granular structure over sandy clay with some lime.
- AL: Layered stream alluvium, Silts, sands & Gravel.

Soil Profile

Taylor (1974) provides a photograph, to show in detail the RB5 soil profile, which we have used to create a geotechnical model. This is shown in in Figure 18.



Depth 0 -20: Sand (SC) with some clay fines, few pebbles, dark grey.

Depth 20-25: Sand soil (SC) with some clay fines, light brown.

Depth 25-30: Clay soil (CL), very sandy and silty, low plasticity, light red-brown

Depth 30-75: Clay soil (CL-CH), high plasticity, red-brown, granular structure, dark grey interfaces.

Depth 75-135: Clay soil (CL), moderate to high plasticity, light red-brown; granular structure; moderate to high lime mainly as concretions. Some pebbles.

Depth 135 – 180: Clay soil (CL), silty to sandy, moderate plasticity, light red – brown, slight lime concretions. Few pebbles.

Depth (180 +): Continuing silty and sandy.

Figure 18: RB5 Soil Profile (Taylor et al 1974)

It can be seen in Figure 18, that the RB5 subsurface soil layers which represent the orange region of the Adelaide soil maps has a large amount of interchanging soil compositions until 0.18 meters depth is reached. The surface soils consist of sand with a small amount of clay particles or a clay with a high amount of sand content until a full clay layer is reached at 0.3 meters depth. The soil profile shows clay dominant layers with high plasticity between depths of 0.3 meters and 1.35 meters with a silty - sandy clay layer with moderate plasticity from 1.8 meters and below.

Hydro-future was able to gain access to a borehole log, which was undertaken by URS at a site 27 Vincent Place, Adelaide (ACC, 2005), as this is located within the RB5 soil profile, we have interpolated data from this borehole log to gain a better understanding of the regional geology of the project area. As simplified version of this borehole log is presented in Figure 19.

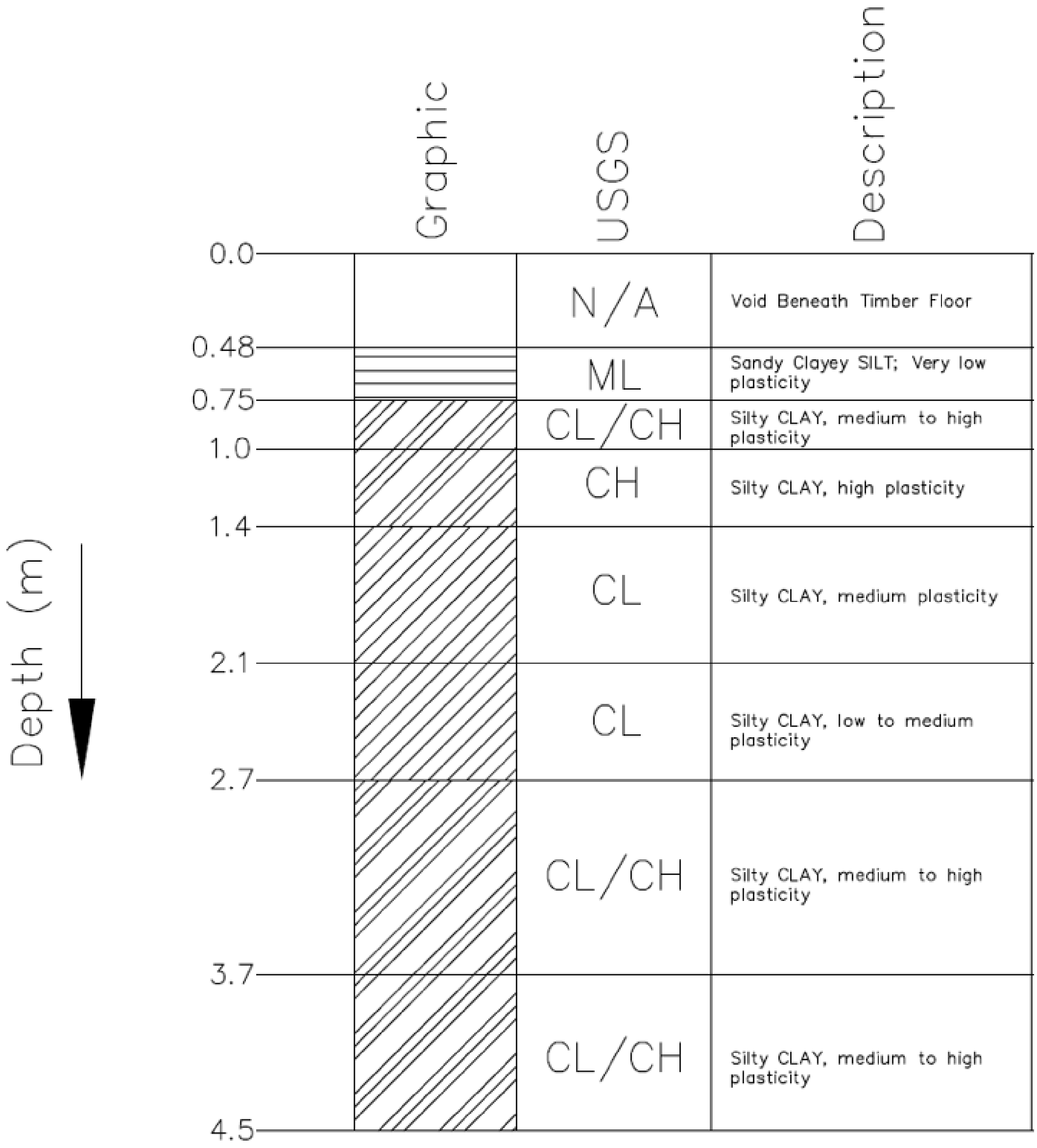


Figure 19: Red Brown Earth Type 5 Borehole Profile (Hydro-Future, 2015)

This borehole has been used to give information for the RB5 soil profile to a depth of 4.5m. The soil profile shows that the subsurface soils are comprised of low plasticity sand, silt and clay up to a depth of 0.75 meters where the dominant soil types become silt and clay, the plasticity range due to this composition change also increases from low to high up until the end of the soil profile at 4.5 meters deep.

Comparison

The borehole data shows information about RB5 in significantly greater detail then the soil profile from Taylor et al (1974). It can be observed that the soil profile in Figure 18 is only given until a depth of 1.8 meters which is considered inadequate due to the construction and design criteria of this project. However Figure 19 includes soil layers up to a depth of 4.5 meters which is acceptable for this type of project.

With respect to the soil layers provided, Figure 18 shows close agreement with Figure 19, which include a mixture of sand, silt and clay compositions at the subsurface (0 -0.75 meters) with clay – silt compositions becoming dominant from a depth of 0.75 meters and deeper. The correlation between the soil profiles reinforces the accuracy of the borehole data which will be used for calculations regarding RB5 soil underneath North terrace.

Characteristics

Following the analysis of the soil profiles above, several of the soil layers have characteristics which can be used to increase design and construction efficiency for the North Terrace Drainage Design project. These characteristics include:

- The minimum cover for a stormwater pipe is 0.6 meters plus the pipe diameter which is likely to fall between the dense clay regions of 0.75 – 4.5 meters and 0.3 + meters of each soil profile respectively. These clay layers will produce more efficient and higher quality compaction rather than the shallow sandy layers which both occur in the early depths of each soil profile.
- The loose density sand layers which are located at the surface of each soil profile will allow for ease of excavation as this soil layer is not densely packed unlike clay.
- Both soil profiles show minimal rock and gravel layers making excavation easier and more efficient.
- There is no visible groundwater level in either profile which allows for the control of moisture content throughout construction and excavation. E.g. before compaction the correct amount of water can be added to the clay layers to achieve the optimum compaction density.

Limitations

Following the analysis of the soil profiles above, several disadvantages relating to the geotechnical data can limit the design and construction of this project, these include:

- The shallow sand layers are not adequate for compaction and may need to be removed to allow for a better quality fill for the reinstatement of the road pavement.
- The dominant clay layers may have a high shrink – swell characteristic which may cause movement of the stormwater pipe.

2.4.3. Alluvial Soil

Alluvial soil is classified as 'AL', this soil deposit includes a large area of the North Terrace project location.

Alluvial soils will range widely with respect to their associated soil layers which can include pebble beds, high sand quantity layers and low plasticity, silt clays. Alluvial soils will be distributed and deposited in locations which are associated with existing natural water ways or past natural water ways which may include the existing flow path or past flow paths of the River Torrens and First Creek (Taylor et al 1974).

Due to the highly variable nature of alluvial origin soils, we have made an assumption of the geological profile based on the relevant soil composition and borehole data which has been located within the region of the North Terrace Drainage Design Project to increase the accuracy and understanding of the local alluvial soil profiles.

Soil Composition

Dominant:

- AL: Layered stream alluvium which consists of clay, silts, sands & Gravel.

Minor:

- SW: Mixed stony Colluvial wash material usually brown and red-brown colours.
- RB3a: As in RB3 but with stone fragments throughout the profile.
- RB9: Nottled silty clay over brown silty clay with granular structure, slight lime, becoming Sandy with depth.
- EMS: Layered sediments of mixed marine and rivin origin sands, silts, clays and organic deposits.

(Taylor et al 1974)

Soil Profile

Although the literature review 'Soils and Geology of the Adelaide Area' has provided Hydro-Future with relevant details including the deposit of soils around North Terrace and the type of soils deposited, a geological profile for alluvial soil has not been associated with this journal article due to the vast variety of soil compositions and layers which alluvial soils encompass. However 'Soils and Geology of the Adelaide Area' has distinguished specific soil layers which may be located around natural geological structures which include the River Torrens and First creek, these soil layers will have a strong relevance to understanding the alluvial soil deposits in the North terrace Project Area which include;

- Sands, clayey sands and fine sandy clays are found in the Torrens River.
- Streams which are located south of the Torrens River deposit silty clays, fine clayey sands and sand layers.
- A 50 cm thick layer of alluvium is associated with the truncated regions of red-brown earth.

(Taylor et al 1974).

The limitations associated with this soil profile include the high amount of unknown soil compositions which can vary depending on the specified location and the lack of information regarding a soil profile for the specified region.

Although some understanding of alluvial soil layers is provided above, it is not an adequate level of information for the feasibility stage. Hydro-Future recognises that these limitations may cause delays with respect to the design and construction of the project and has sourced relevant borehole data within the North Adelaide region where alluvial deposits are found. A bore log which was undertaken by Aurecon on an alluvial deposit, which was within a close proximity of the project site has been used to create a soil profile, which can be seen below in Figure 20.

The soil profile which was provided using borehole data, Figure 20, shows interchanging soil layers between sand and clay. The soil layers also contain sources of gravel and have a lower plasticity then that of the RB5 soil profiles. The soil layers in the borehole data are extremely variant and completely change composition with each layer, this refers to the dominant soil type in each layer which is either sand or clay. The plasticity throughout this soil profile is relatively lower than RB5 which is due to the increased sand composition in alluvial soils.

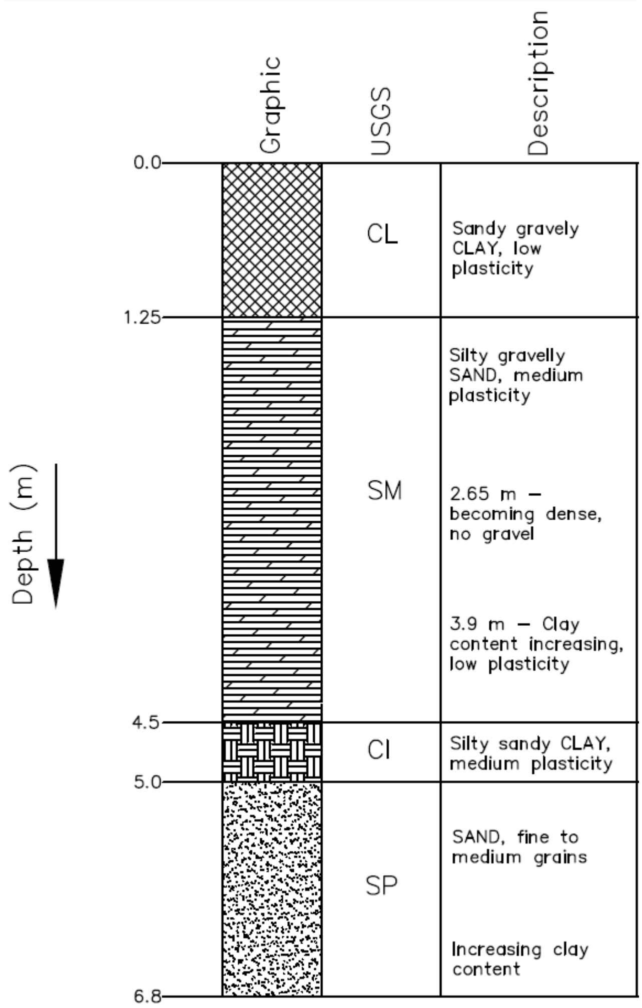


Figure 20: Alluvial Soil Borehole Profile (Hydro-Future, 2015)

Comparison

Taylor (1974) states that soil layers can include sands, clayey – sands, fine sandy – clays, silty – clays and fine clayey – sands will be associated with natural geological features which occur in and around North Terrace. This is reinforced by the borehole data which states that there will be a high amount of varying soil layers and compositions between clays, sands and silts. Alluvial soils were described as being extremely variant in nature, which is also reinforced with the borehole data which interchanges its dominant soil composition from sand to clay with every layer.

Characteristics

Alluvial soils encompass a large variety of soil compositions which can vary depending on the location and geology of the region. Due to these high unknowns associated with the Associated Soil Map of Adelaide Hydro-Future has relied on outsourced borehole data to further analyse this soil deposit, a detailed investigation of the borehole data can be seen in Section 0. The soil characteristics which can be seen in Figure 20 benefit the project in many ways including:

- The low plasticity soils which incorporate high sand content will allow for efficient excavation and reduced resistance when digging.
- The high sand and gravel content in many layers will reduce the shrink – swell characteristics of the soil which will reduce the movement within the stormwater pipe.
- The high sand and gravel content with the low plasticity values will increase the soil permeability and hydraulic conductivity substantially which will benefit the water sensitive urban design features of the project which often rely on high soil infiltration to retain functionality.

Limitations

Following the analysis of the borehole soil profile above, several disadvantages relating to the geotechnical data can limit the design and construction of this project, these include:

- The high sand content within each layer will reduce the strength and stability of excavated areas which include pits and trenches.
- The high sand content within each layer will reduce the quality of soil compaction which is needed to lay the stormwater pipe and reinstate the road pavement.
- The interchanging soil properties will provide some degree of difficulty with respect to the design aspects of the project.

2.4.4. Geotechnical Model

Using the analysis of the regional and borehole soil profiles in Section 2.4.2, Hydro-Future has devised a geotechnical model for the layout of the site as seen below in Figure 21.

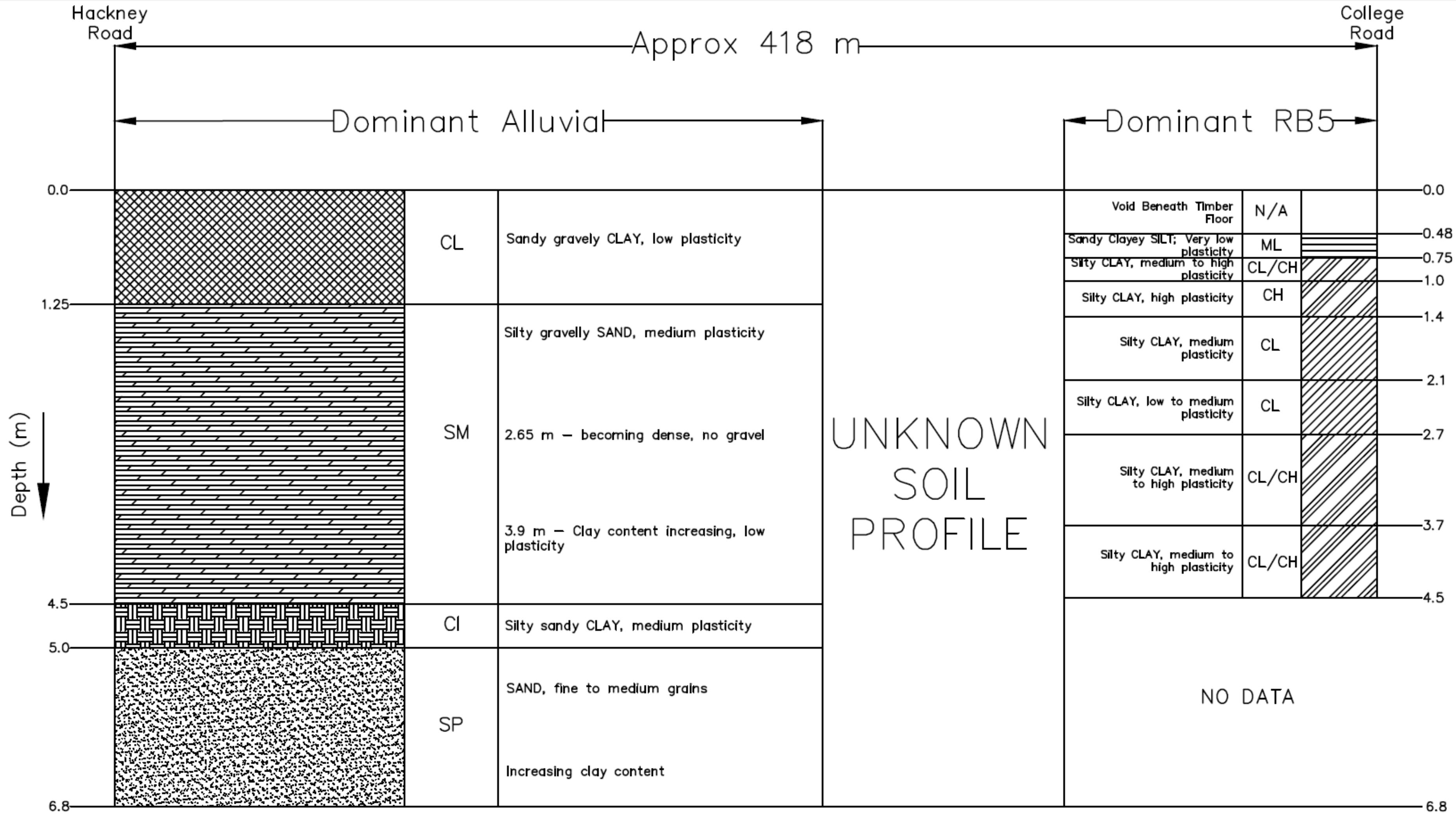


Figure 21: Geotechnical Mode (Hydro-Future, 2015)

2.4.5. Borehole Data

To manage the geotechnical design criteria, Hydro- Future has located borehole data within the regions of the red brown earth and the alluvial soil to gather detailed information regarding the soil properties of the subsurface layers.

Red Brown Earth

Hydro-future was able to gain access to a borehole log, which was undertaken by URS at a site, 27 Vincent Place, which is approximately 2 km away from the project location. As analyzed earlier, the borehole data falls within the region of RB5 and shows close agreement with the Red Brown Earth Type 5 generalized soil profile that is associated with the soil maps.

The borehole data which was acquired to represent the RB5 soil type was originally undertaken in 2010 to analyze the shrink – swell strains on a residential city allotment (ACC 2005).

The borehole has provided the following information:

- Boring resistance created by each soil layer.
- Whether the groundwater table was struck and if so its location.
- 4.5 m deep borehole including each soil layer and its characteristics – USCS classification, plasticity, color, layer thickness, moisture content, relative density and a pocket penetrometer reading.
- The soil was analyzed and identified to be Red Brown Earth Type 5 by URS.

Soil Profile

The borehole data was able to provide Hydro-future with relevant geotechnical information regarding the RB5 soil profile, including the soil layers, USCS classification, depths, moisture content, relative density and pocket penetrometer readings. This data has been presented in Figure 22.

Figure 22: Soil Profile (Hydro-Future, 2015)

Depth (m)	Graphic	USGS	Description	Moisture Content	Relative Density	PP Reading (KPa)
0.0–0.48		N/A	Void Beneath Timber Floor	N/A	N/A	N/A
0.48–0.75		ML	Sandy Clayey SILT; Very low plasticity	<PL	Fb	N/A
0.75–1.0		CL/CH	Silty CLAY, medium to high plasticity	<=PL	Vst	250–350
1.0–1.4		CH	Silty CLAY, high plasticity	~PL	Fb	250–350
1.4–2.1		CL	Silty CLAY, medium plasticity	~PL	Fb	250–350
2.1–2.7		CL	Silty CLAY, low to medium plasticity	~PL	Fb	250–350
2.7–3.7		CL/CH	Silty CLAY, medium to high plasticity	>=PL	St.Vst	150
3.7–4.5		CL/CH	Silty CLAY, medium to high plasticity	>PL	Vst	300–400

Course Grained Soils	
VL	Very Loose
L	Loose
MD	Medium Dense
D	Dense
VD	Very Dense

Table 6: Course Grained Soil Terms (ACC 2005)

Fine Grained Soils	
Symbol	Term
VS	Very Soft
S	Soft
F	Firm
St	Stiff
VSt	Very Stiff
H	Hard
Fb	Friable

Table 7: Fine Grained Soil Terms (ACC 2005)

Geotechnical Model – RB5

Based on the available information, Hydro-Future has created a Geotechnical Model, shown in Table 8, the information has been defined using relevant mathematical methods, geotechnical publications and Australian standards.

The geotechnical model will be used for further calculations into the following:

- The RB5 soil group is predominantly clay, indicating high shrink-swell characteristics which will affect the stormwater piping.
- The soils permeability, which is extremely low for clay, will affect any water sensitive urban design features within St. Peters College & the Clarke Rubber Carpark.
- Trench stability for the region of the RB5 soil deposit.

Due to the similarity in silty CLAY compositions the layers from 0.75 – 4.5 meters depth have been represented as one layer.

Table 8: RB5 Geotechnical Model (Hydro –Future, 2015)

Depth	***Soil***	***Liquid Limit*** $\%_{1,4,7,9}$	***Plastic Limit*** $\%_{4,7,9}$	***Plastic Index***$_{5,9}$	***Hydraulic Conductivity*** K_v (m/day)$_6$	***Unit Weight*** γ (kN/m^3)$_3$	***Undrained Shear Strength*** C_u (kPa)$_{1,3,5,8}$	***Shear Strength*** C' (kPa)$_2$	***Internal Frictional Angle*** ϕ_2	***Permeability*** k (m/s)$_5$
0.0 – 0.75	Sandy clayey SILT	≤35	~10	20	0.2 – 0.5	18.5	75	8.5	28	10^{-6}
0.75 – 4.5	Silty CLAY	>30≤50	~15	30	0.002 – 0.2	21.5	125	3	22	10^{-8}

1) **AS 1726**
2) **AS 4678**
3) **AUSTROADS (1992)**
4) **p35, Lambe, Whitman, 1969**
5) **p48, 337 & 460, Smith 2006**
6) **p18, Oosterbaan, Nijland, 1994**
7) **p14, Craig, 2004**
8) $C_u = \frac{\text{Pocket Penetrometer}}{2}$
9) $\text{Plastic Limit} = \text{Liquid Limit} - \text{Plastic Index}$

Alluvial Soil

The alluvial soil deposit has been analysed in relation to a borehole located on the corner of Bundeys Road and Hackney Road. This borehole location is within the vegetated area of botanical park / Adelaide zoo and is at the edge of the River Torrens which according to 'Soils and Geology of the Adelaide Area' is a location which will contain alluvial soil deposits.

The borehole data was originally collected by Aurecon for a project regarding the construction of a bridge underneath Hackney road in 2010. The borehole data has provided the following information:

- 20 m deep borehole (6.8 m analyzed) including each soil layer and its characteristics – USGS classification, plasticity, color, layer thickness, moisture content, relative density and a SPT blow count.
- Location of the groundwater table.

Using the above borehole data the Hydro-Future can gather relevant geotechnical information regarding the alluvial soil deposit.

Soil Profile

The borehole data has been interpreted, and the relevant information including the soil layers, USCS classification, depths, moisture content, relative density and SPT blow counts have been extracted from the bore log data and included in the following soil profile which can be seen below in Figure 23. As seen below only the first 6 meters of the bore log data have been documented due to the visibility of data and due to the data relevance. This project's excavation is regarded to be reasonably shallow and will not exceed the 6 meters which is shown below in the soil profile.

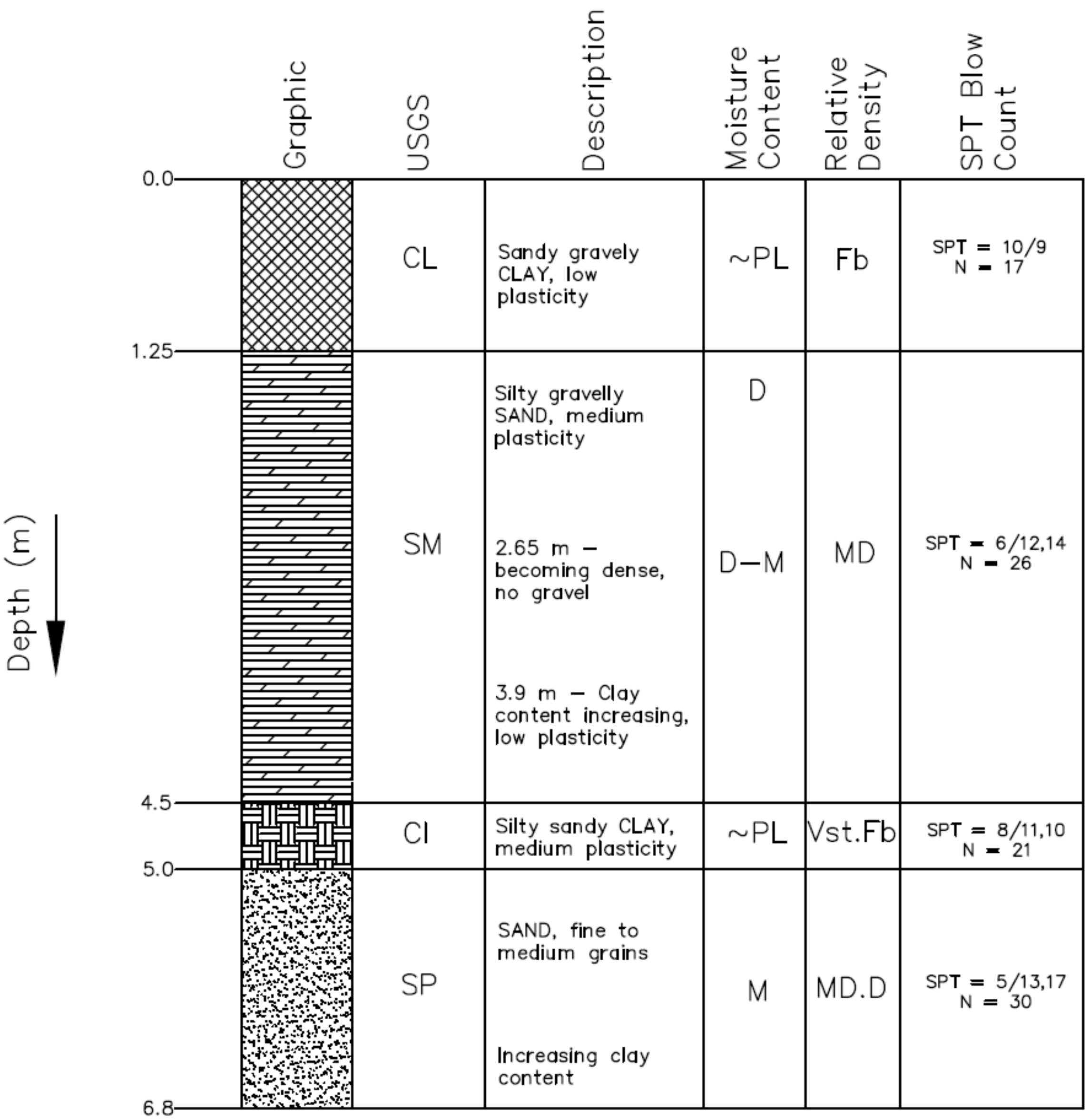


Figure 23: AL Soil Profile (Hydro-Future, 2015)

Note: Refer to Table 6 and*

Table 7 for course grained and fine grained soil terms to interpret the above soil profile data.

Geotechnical Model-AL

Establishing the soil properties for each soil layer within the alluvial soil deposit will give Hydro-Future a clear understanding of how the highly variable sand – clay compositions are behaving along the alluvial soil deposit. The soil properties will allow further calculations into the following:

- The design loads (dead loads) on the sandstone arch culvert and the concrete box culvert as these underground features are located in a heavily composed alluvial soil region.

- Permeability values for water sensitive urban design technologies which may be located underneath the Clarke Rubber or Royal Hotel carpark.

- Trench stability for the region of the alluvial soil deposit.

Table 9 presents the Geotechnical model, and has been created using relevant mathematical methods, geotechnical publications and Australian standards.

Table 9: AL Soil Properties (Hydro-Future, 2015)

Depth	Soil	Liquid Limit (%)[1,4,7,9]	Plastic Limit (%)[4,7,9]	Plastic Index[5,9]	Hydraulic Conductivity K_v (m/day)[6]	Unit Weight γ (kN/m^3)[3]	Undrained Shear Strength C_u (kPa)[1,3,5,8]	Shear Strength C' (kPa)[2]	Internal Frictional Angle ϕ[2]	Permeability k (m/s)[5]
0.0 – 1.25	Sandy gravelly CLAY	30	20	10	0.2 – 0.5	21.5	102	7	17	10^{-6}
1.25 – 4.5	Silty gravelly SAND	0	0	0	1 – 3	20	156	10	26	10^{-4}
4.5 – 5.0	Silty sandy CLAY	40	25	15	1 – 3	21.5	126	3	20	10^{-6}
5.0 – 6.8	SAND	0	0	0	1 – 5	20.5	180	10	33	10^{-3}

1) **AS 1726**
2) **AS 4678**
3) **AUSTROADS (1992)**
4) **p35, Lambe, Whitman, 1969**
5) **p48, 337 & 460, Smith 2006**
6) **p18, Oosterbaan, Nijland, 1994**
7) **p14, Craig, 2004**
8) $C_u = 6N$, ***N = number of blows***
9) $Plastic\ Limit = Liquid\ Limit - Plastic\ Index$

2.4.6. Geological Setting

Several journal articles have been revised by Hydro-Future to collect information regarding the regional soil deposits and the geological profile underneath North Terrace. The 'Engineering Geology of the Adelaide City Area' (Selby, Lindsay 1982) provides valuable information regarding the different soil and rock deposits and their depths beneath North Terrace. In Figure 24, an extract from Selby and Lindsay (1982) shows a geological cross section (G-G) which extends from Mile End across the city until its directly beneath the North Terrace project Site.

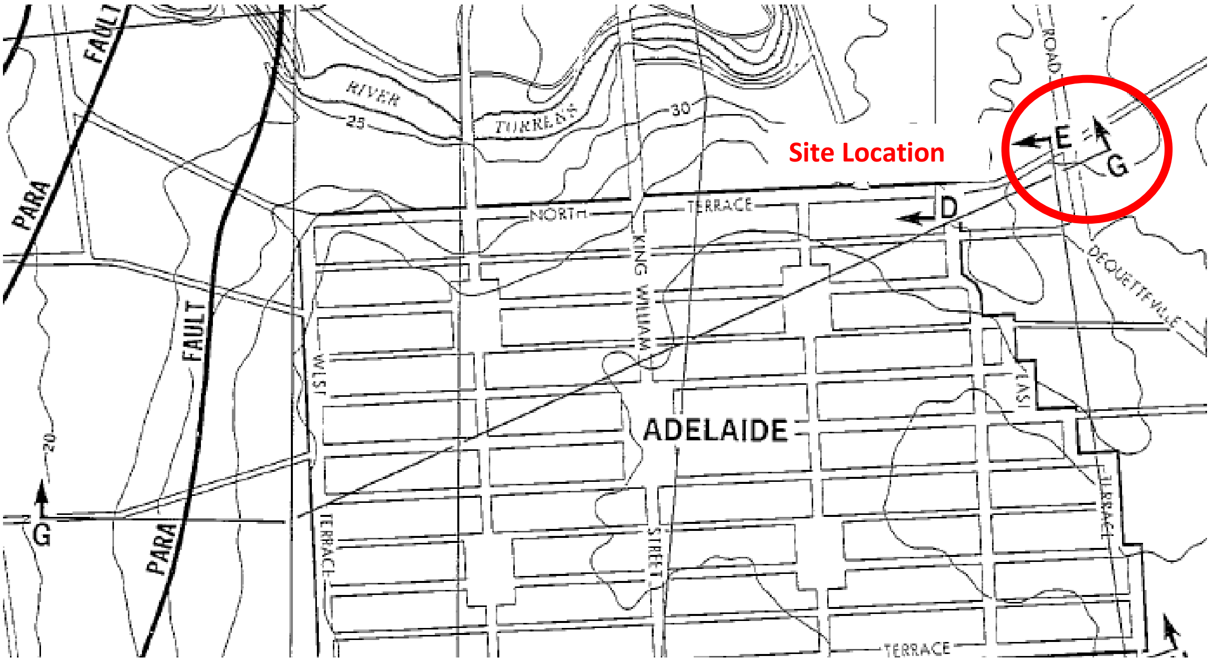


Figure 24: Geological Cross Sections - Plan View (p90, Selby, Lindsay, 1982)

Figure 25 is a cross sectional view of G-G underneath the project site of North Terrace. The cross section shows the groundwater table does not start until approximately 22 meters below the surface. This information allows Hydro-Future to coordinate the excavation of the site with respect to soil within the plastic limit which provides strength to tranches during excavation and allows Hydro-Future to manipulate the moisture content in the subsurface soils to allow for maximum compaction.

Rock levels are a major concern within the construction of this project due to potential delays and implications associated with excavation. Hydro-Future aims to coordinate the construction stage of the project with minimal disruptions, this means identifying if and where rock may be located below the surface of North terrace. Figure 25 identifies that rock is likely to be located approximately 50 meters below the surface of North Terrace, which will not interfere with any excavation. The excavation to lay a stormwater pipe may not exceed a depth of 1.5 meters depend on the required cover and pipe size.

The cross section G – G also details the subsurface soil composition which is predominantly comprised of silt over the first 10 meters underneath the surface. Although the accuracy of the subsurface soil is not of a reasonable standard it enforces the borehole data which is predominantly silt and clay over the deposit of RB5.

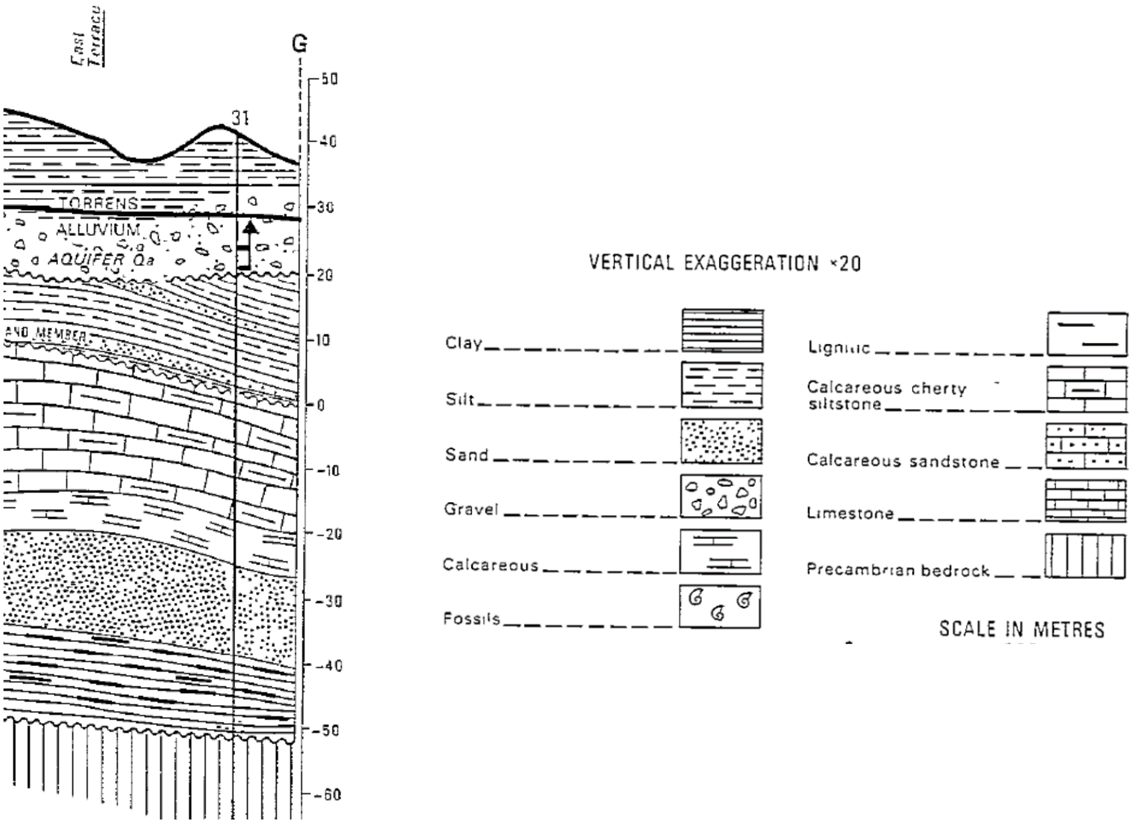


Figure 25: Cross Section G - G (p94, Selby, Lindsay, 1982)

2.4.7. Potential Subsurface Hazards

Hydro-Future recognises the risks of construction in a developed area with high infrastructural history. During the course of construction and excavation extensive precautions and safeguards need to be implemented to ensure that excavation does not clash with any of the past underground infrastructure which may include the North Terrace Tram line which is now likely to be 100 years old. This infrastructure was supported on cement blocks which are highly likely to be encountered during excavation. Figure 26 shows the old tram line in operation over 100 years ago.

Figure 26: Old North Terrace Tram Line (Johnny's Pages, n.d)

2.5. Urban Design

The local council, The City of Norwood, Payneham and St Peters has produced a document entitled 'City Plan 2030' and is based on the concept that any changes made to the community can have large impacts on future generations. This documents highlights four key outcomes that the council feels has the largest influence on the community and these are;

- Social Equity;
- Cultural Vitality;
- Economic Prosperity, and;
- Environmental Sustainability

The council's vision is that in 2030, they have "a city which values its heritage, cultural diversity, sense of place and natural environment. A progressive City that is prosperous, sustainable and socially cohesive, with a strong community spirit. " (City of Norwood Payneham & St Peters 2012)

The design options presented in the following sections have been guided by the findings of the 'City Plan 2030' and have been carefully studied to ensure that the client is able to make informed decisions for the final design.

2.5.1. Service Clearance

Within the project area there are a number of utility services, all of which have the potential to influence the feasibility of design options. This section outlines issues that need to be considered during construction in and around these services, including those located above and below ground.

Utility provider code of practice for Western Australia was used as strong guide to make sure no major damage and injury that caused by the digging around urban roads.

The following acts, regulations, codes of practice and industry guidelines should be consulted when undertaking construction work near utility services;

- Gas Act 1997
    - Gas Regulations 2012
    - AS/NZS 4645
- Electricity Act 1996
    - Electricity (General) Regulations 2012
- Work Health and Safety Act 2012
    - Work Health and Safety Regulations 2012
- Water Industry Act 2012
    - Water Industry Regulations 2012
    - SA Water Infrastructure Standards and Guidelines
- Telecommunications Act 1997

- Telecommunications Codes of Practice 1997
- ACIF C524:2001

- AS 2648.1 1995 Underground Marking Tape
- AS 1345-1995 Identification of the contents of pipes, conduits and ducts
- AS 2566.2 2002 Buried flexible pipelines
- AS 1742.3 Traffic control devices for works on roads
- Code of Practice: Safety Precautions in Trenching Operations
- Code of Practice: For Confined Spaces

To assist in identifying underground services, Table 10 outlines common pipe colours. Based upon the type of service, standard clearances apply when undertaking construction, these are shown in Table 11

Table 10: Common Colourations of Underground Services (UPSC 2010)

Service Types	Standard Colour
Gas	Yellow or Yellow Striped
Electrical	Orange pipe or Orange Striped
Traffic Signals	Orange
Roadside Lighting	Orange
ITS	Orange(power) and White
Telecommunications	White or White and Black Stripe
Water	Blue or Blue Striped
Sewerage	Cream or Grey or Cream/Grey Striped
'Third Pipe'/Effluent Reuse	Purple

Table 11: Standard Clearances from Services (UPSC 2010)

Types of Utility Underground Services	Clearance Zone for Powered Excavation	Typical Depths (mm)
Low pressure gas mains	300 mm	300 –450
Medium pressure gas mains	300 mm	450 – 750
High pressure gas services, mains and pipelines	300 mm	750 – 1200
Telecommunications cables [1]	500 mm	450 – 600 [2]
Water Supply	300 mm [3]	450
Sewer	300 mm [3]	600 - 10000
Notes: Potholing is the preferred method for identifying services [1] Telecommunication service location requires specific conditions for undertaking identification of services (Worksafe Victoria 2004) [2] can be to 1200mm in depth [3] if pipe is 200mm or greater in diameter		

Above ground services include street lighting, telephone boxes and electricity. Stobie poles and their associated power lines have the strictest requirements. To manage risks associated with construction the following needs to occur (as per AS2550);

- Identification of the electricity voltage;
- A documented risk assessment;
- The electricity network operator is informed and imposed condition complied with, and;

A professional spotter (someone with experience, training or both with working around power facilities) supervising the operation at all times Figure 27 shows the clearance zones for operating machinery in proximity to power lines:

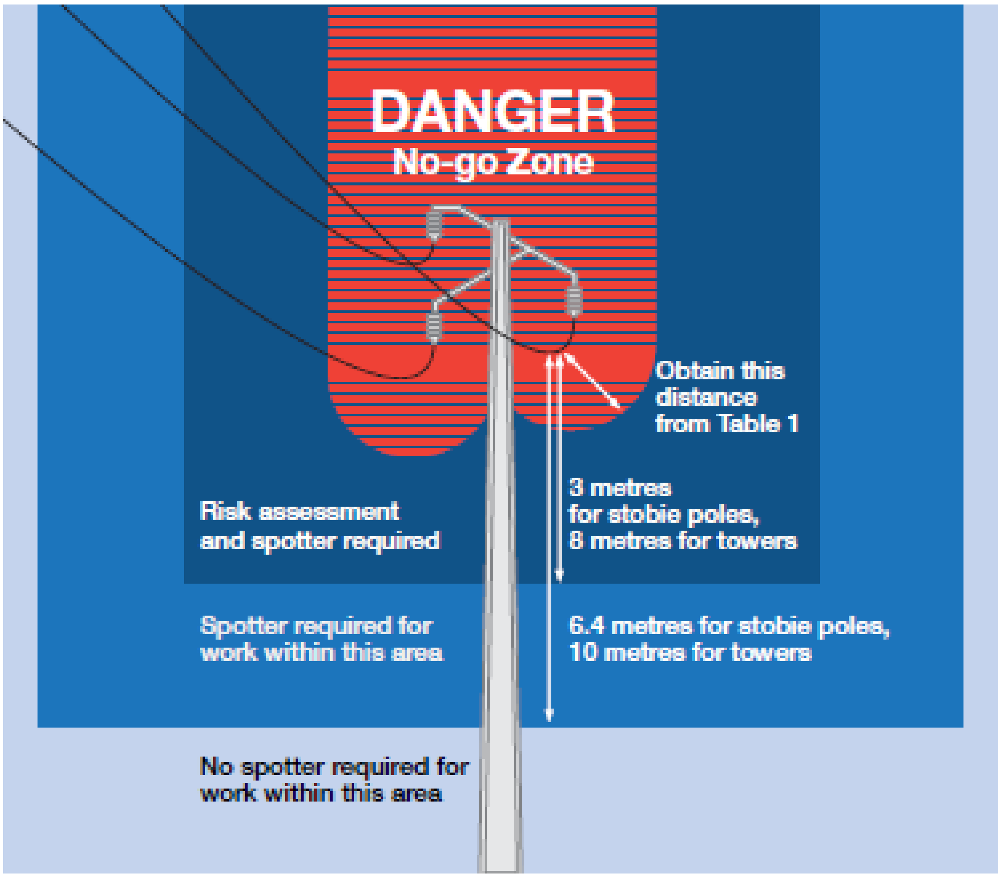


Figure 27 Clearance zones required for operating machinery near power lines (DPTI 2012)

Table 12 shows the clearance distances from power lines, this is to be measured from the position of the closet conductor (DPTI 2012):

Table 12 : Minimum Clearances from Power Lines (DPTI 2012)

	Cranes AS 2550.1 Crane Code		**Machinery** Electricity Regulations 2012	**Safe Approach Limits** Electricity Regulations 2012		**Buildings and Structures, including Scaffolds**	
Voltage	No Spotter	Spotter required	Risk assessment and spotter required	Approach limit – normal persons	Approach limit – with risk assessment	Horizontal direction	Vertical direction
240	6.4m	3.0m	1.0m	3.0m	1.0m	1.5m	3.7m
415	6.4m	3.0m	1.0m	3.0m	1.0m	1.5m	3.7m
7600	6.4m	3.0m	1.5m	3.0m	2.0m	3.1m	5.5m
11000	6.4m	3.0m	1.5m	3.0m	2.0m	3.1m	5.5m
19000	6.4m	3.0m	1.5m	3.0m	3.0m	3.1m	5.5m
33000	6.4m	3.0m	3.0m	3.0m	3.0m	3.1m	5.5m
66000	6.4m	3.0m	3.0m	4.0m	4.0m	5.5m	6.7m
132000 pole	6.4m	3.0m	3.0m	5.0m	5.0m	15m	N/A
132000 tower	10.0m	8.0m	3.0m	5.0m	5.0m	20m	N/A
275000	10.0m	8.0m	4.0m	6.0m	6.0m	25m	N/A

Our team has identified some important public infrastructure that needs to be considered during construction, these are shown Figure 28, Figure 29 and Figure 29.

Figure 28: (left to right) Traffic Light Controller Box, Telephone Junction Box, Fire Hydrant

Figure 29 *Underground Telecommunication Services*

Figure 30: (left to right) Telephone booth and Fire hydrant booster

2.5.2. Heritage issues around the project area

Maintaining the local heritage of the area, is of upmost importance for the project area, as outlined in the council's City Plan 2030.

In South Australia, places and objects of state and local heritage are protected under the *Heritage Places Act 1993* and the *Development Act 1993*. Based on a site visit performed by our Urban Planning team, the project area contains 14 buildings that are on the local and state heritage listing. On the northern side of North Terrace there are 9 buildings, and 5 on the southern side. During construction, emphasis needs to be placed on protection of the heritage buildings.

Protection of heritage buildings during construction is a two stage process, both during and after construction.. During construction, extra fencing should be added around heritage sites, and once the construction is finished, the construction site should be cleaned and restored to its original situation. Based on guidelines produced by the NSW Heritage Office (2002), from a heritage point of view, not only the main building on the site should be protected, but other elements of the site, such as paving, garden, outbuildings lamp standards and so on also should be on the protection list. As a result, fencing should be placed 0.5 metres from heritage site boundaries (NSW Heritage Office 2002). During construction, if any archeological significant material is identified appropriate experts in the area will need to be consulted. Below, photographs of heritage site are presented to allow for easy identification during construction.

Figure 31 Romilly House, heritage building at the intersection of the North Terrace and Hackney Rd (Hydro-Future, 2015)

Figure 32: 37 North Terrace Hackney, SA - Heritage Building (Hydro-Future, 2015)

Figure 33: 33 North Terrace HACKNEY, SA - Heritage Building (Hydro-Future, 2015)

Figure 34: 23 North Terrace Hackney, SA - Heritage Building (Hydro-Future, 2015)

Figure 35: 39 and 41 North Terrace Hackney, SA - Heritage Building (Hydro-Future, 2015)

The following processes should be applied before any construction begins:

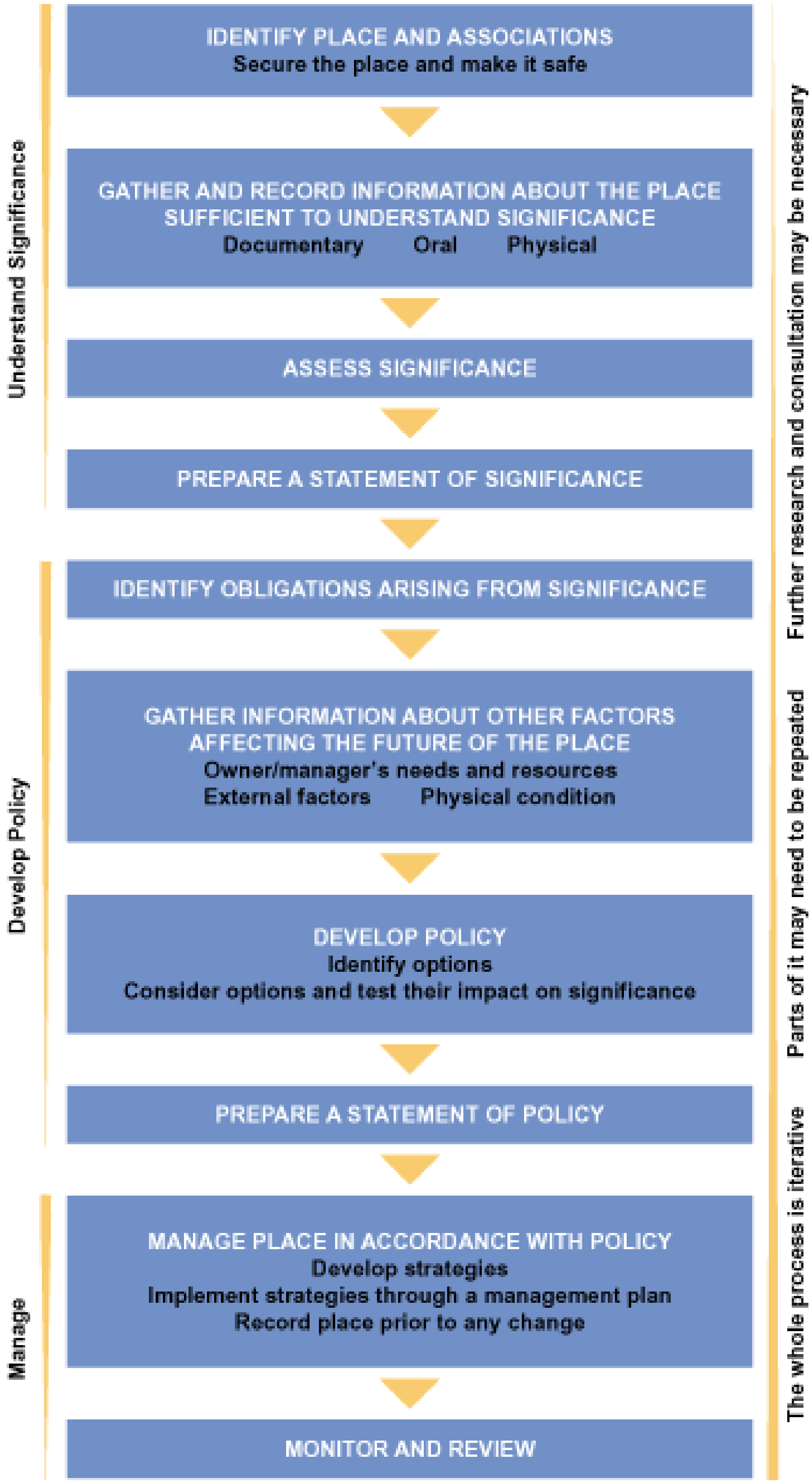


Figure 36 the process that undertaking when developing a project around heritage building (NSW Heritage Office 2002)

Table 13 shows the location and classification of local heritage building around construction area:

Table 13: Heritage Listed Building within the Project Area, all lie within the City of Norwood, Payneham and St Peters (sa.gov.au 2014)

Heritage No.	Address	Details	Class
5832	32 North Terrace KENT TOWN	Former Victorian Bluestone Dwelling	Local
5833	58 North Terrace KENT TOWN	Victorian Masonry Cottage	Local
5834	60 North Terrace KENT TOWN	Victorian Sandstone & Bluestone Villa	Local
6039	2 North Terrace KENT TOWN	Royal Hotel	State
6040	64 North Terrace KENT TOWN	Office (former Parkin College)	State
6394	23 North Terrace HACKNEY	Dwelling ('Singleton')	Local
6395	31 North Terrace HACKNEY	Attached Dwelling	Local
7867	33 North Terrace HACKNEY	Attached Dwelling	Local
6396	37 North Terrace HACKNEY	Row Dwelling	Local
7863	39 North Terrace HACKNEY	Row Dwelling	Local
7864	41 North Terrace HACKNEY	Row Dwelling	Local
5608	North Terrace HACKNEY	Palm House (former Dwelling), St Peter's College	State
5601	1 North Terrace HACKNEY	Former Romilly House	State
6369	85 North Terrace COLLEGE PARK	Row Dwelling	Local
7804	87 North Terrace COLLEGE PARK	Row Dwelling	Local
7805	89 North Terrace COLLEGE PARK	Row Dwelling	Local

2.5.3. Open Spaces and Property Values

Due to the potential of acquiring land for some of the design options, land price is an essential factor requiring analysis. Figure 37 shows the location of large open spaces. Table 14 summarises the land values in the project area, many factors influence the land price including, location, property type, surrounding environment, and quality of buildings. Based on the results of investigation from urban planning team, the prices of land varies from $1000 to $7000 per square metre. Higher values apply to commercial properties, as such the land selection criteria should be based on this information. Within the entire catchment, more properties are residential, but within the project area more commercial properties are present.

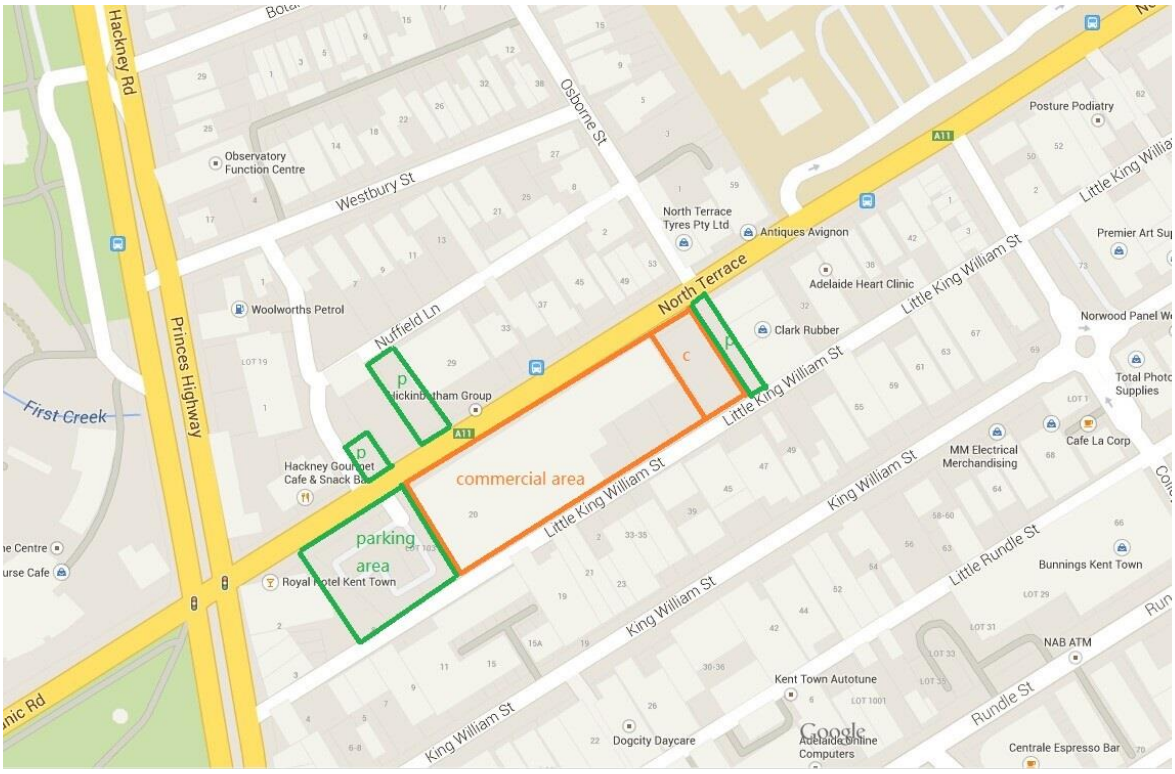


Figure 37 Vacant land within the project area (image source: google maps)(Hydro-Future 2015)

Table 14: Average land price in the project area (onthehouse.com.au, 2015)

Property No.	Property Address	Property Price ($)	Land Size (m^2)	Average Price ($\$/m^2$)
1	17 Edward Street Norwood SA	550,000	505	1,100
2	5 Conigrave Lane Norwood SA	650,000	469	1,400
3	73 Stephen Terrace St Peters SA	650,000	590	1,100
4	47 Aveland Avenue Trinity Gardens SA	820,000	900	1,000
5	5 Pembroke Street College Park SA	2,600,000	1,585	1,700
6	28a Wakefield Street Kent Town SA	1,250,000	291	4,300
7	8 Harrow Road College Park SA	1,200,000	1,084	1,200
8	65 Hackney Road Hackney SA	1,300,000	558	2,400
9	125 Second Avenue Royston Park SA	1,175,000	1,091	1,100
10	58 Seventh Avenue St Peters SA	1,250,000	696	1,800
	Average Price Around Project Area ($\$/m^2$)			1,800

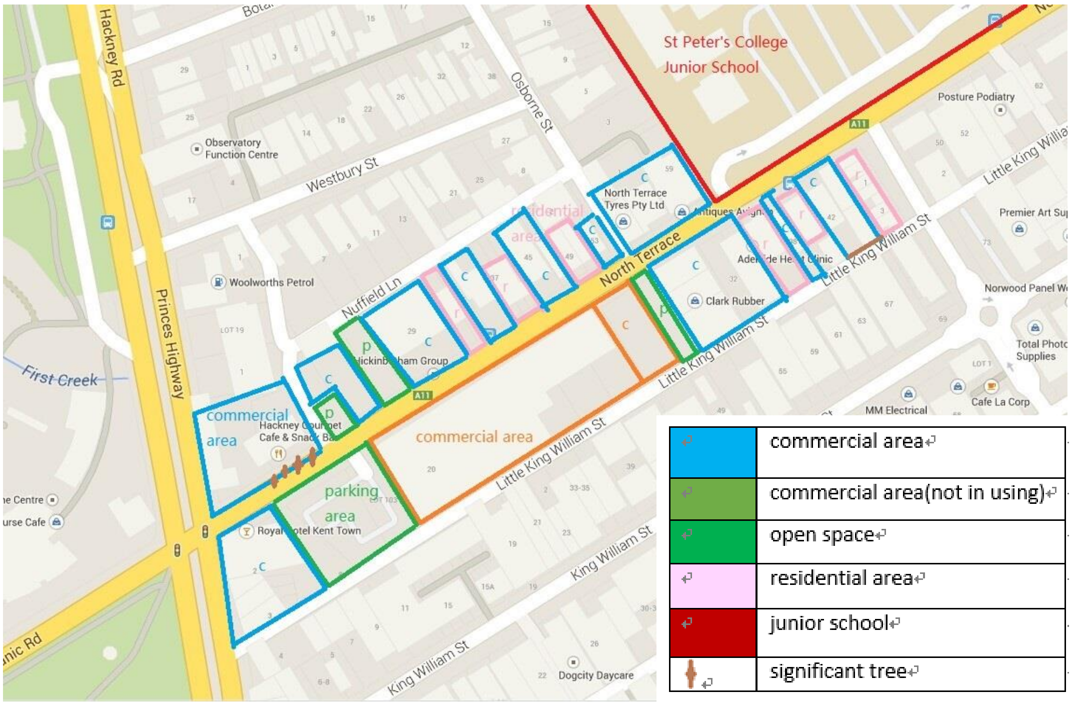


Figure 38: Zoning of properties within the project area (Image Source: Google maps)(Hydro-Future 2015)

2.6. Structural

The structural considerations associated with this project, involve an investigation into viable options on how to connect the proposed stormwater drainage design into First Creek which currently flows beneath North Terrace, Hackney Road and under the residential and domestic footings of the nearby infrastructure through the means of a 150 year old sandstone arch culvert and a modernised concrete box culvert.

Depending on the chosen design option, an investigation and structural analysis of the 150 year old heritage listed sandstone arch culvert or the modernised concrete box culvert may be needed to ensure each piece of infrastructure will not be in danger of collapse. The arch culvert lies beneath Hackney Road and North Terrace whilst the concrete box culvert is used to support the foundations of domestic and residential.

Figure 39 below shows existing stormwater drainage systems underneath North Terrace which had been provided to Hydro-Future from the City of Norwood, Payneham and St Peters. This document illustrates the current construction layout of the arch culvert and the concrete box culvert throughout the area of Norwood, Payneham and St Peters. The arch culvert was constructed using old and somewhat unknown construction methods and materials, which include the use of sandstone blocks, adhered using mortar, whilst the concrete box culvert is of a modern construction using conventional reinforced concrete.

Figure 39 – Current Stormwater Infrastructure (City of Norwood, Payneham & St Peters, n.d.)

2.6.1. Site Investigation

The structural condition of the sandstone arch culvert has been analysed through a preliminary investigation conducted on the 24th of March 2015, by Hydro-Future. Figure 40 shows the sandstone arch culvert at its joint with the concrete box culvert.

Figure 40 - Sandstone Culvert – Box Culvert Connection (Hydro-Future, 2015)

2.6.2. Condition of Sandstone Arch Culvert

As a result of the site investigation, which included several visual observations which have been documented with photography, Hydro-Future was able to gain an appreciation of the current condition of the sandstone culvert. The key structural integrity issues that have been identified during the investigation of the culvert are discussed below.

Sandstone Decay:

The sandstone culvert showed clear exterior damage to the sandstone blocks which had separate at some regions and were left on the creek bed. These fragmented pieces were distributed throughout the box and arch culvert, as shown in Figure 41 and Figure 42. It is apparent that these pieces of sandstone broke off from the culvert walls and roof. The culvert is 150 years old, hence this may have occurred because of the age and decay of the structure, however the culvert is renowned for its extensive vandalism and this indicates that the missing pieces of sandstone may have been intentionally removed.

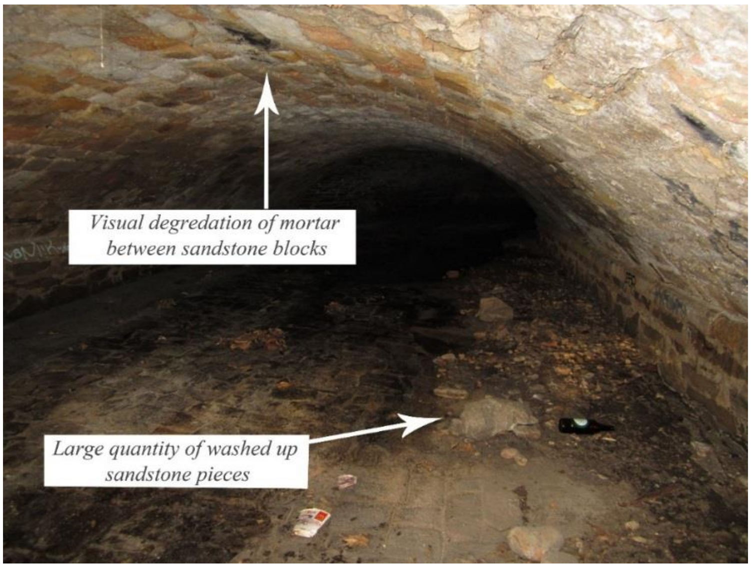


Figure 41: Visual inspect of culvert condition (Hydro-Future, 2015)

Figure 42: Visual inspect of culvert condition (Hydro-Future, 2015)

Mortar Decay:

Further photographs taken during the site visit show the condition of the mortar cement between sandstone block throughout the culvert, as shown in Figure 43 and Figure 44. These figures show signs of mortar deterioration i.e. chucks of mortar missing between sandstone blocks signs of erosion.

Mortar between these Sandstone blocks is vital for the bonding strength of the culvert. Insufficient and degraded mortar prevents the culvert from retaining the sandstone blocks in place, therefore promoting the scattering of sandstone pieces that weaken the structural integrity.

Figure 43: Visual inspect of mortar integrity (Hydro-Future, 2015)

Figure 44: Visual inspect of mortar integrity (Hydro-Future, 2015)

Mortar repairs:

Repairs to the arch culverts degraded mortar are present, as illustrated in Figure 45. Most areas of the culvert ceiling show signs of mortar repair, however, most zones are still fairly degraded and may require further repair of mortar or inclusion of additional support strengthening structures.

Figure 45 – Sandstone Culvert mortar repair (Hydro-Future, 2015)

2.6.3. Dimensions

The preliminary investigation of the culverts was conducted on the 24th of March, 2015. Table 15 illustrates the details of the investigation with respect to the arch culvert, and provides useful preliminary dimensional information.

To ensure accuracy and consistency between the documented dimensions and the measured dimensions, relevant organisations were contacted to ensure original drawings and measurements of the arch and box culvert aligned with the site investigation. Figure 46 shows the council drawing which represents the connection point between the box and arch culvert beneath North Terrace. As the thickness of the sandstone arch culvert could not be measured Hydro-Future will refer to the council drawings for these dimensions and it will be assumed the thickness is uniform throughout the culverts length.

Table 15 – Arch Culvert Dimensions (Hydro-Future, 2015)

Arch Culvert Dimensions		
Aspect	**Original Council Drawings**	**Hydro-Future Measurements**
Bottom Width	4.2 m	4.0m
Middle Height	1.8 m	1.9m
Side Step Height	(N.A. , assume 0.6m)	0.7m
Radius of Arch	N.A.	Approx. 4-5m
Sandstone Wall Thickness	230mm	N.A.

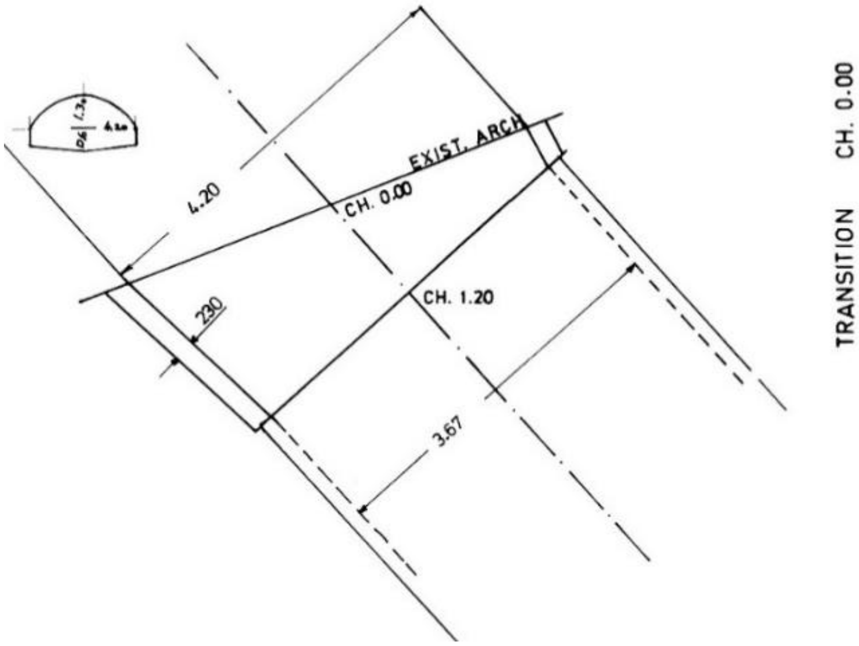


Figure 46 – Arch –Box Culvert Connection (City of Kensington and Norwood, Oct 1993)

The measurements which were taken during the site visit can be seen in a preliminary drawing, Figure 47 below.

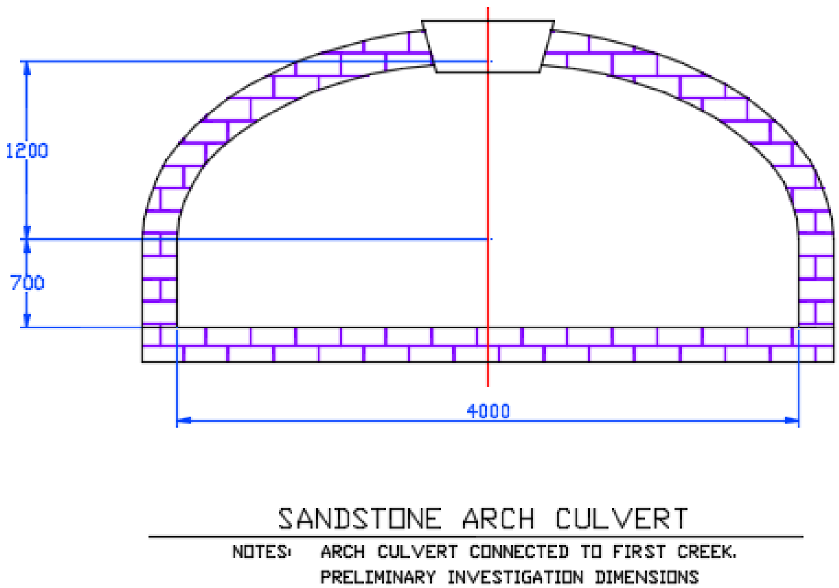


Figure 47: Sandstone Arch Culvert (Hydro-Future, 2015)

As seen below in *Table 16* are the measured and documented dimensions from Hydro-Future and the council drawings respectively.

Table 16 – Box Culvert Dimensions (Hydro-Future, 2015)

Box Culvert Dimensions		
Aspect	**Original Council Drawings**	**Hydro-Future Measurements**
Width	3.67 m	3.6 m
Height	1.8 m	1.9m
Wall Thickness	230mm	N.A.

There is sufficient data for the size of the culvert underneath and between North Terrace and Hackney Road, therefore Table 17 provides a general guideline of the dimensions which may occur along the arch culvert which allows for any site tolerances, similarly Table 18 provides tolerances for the existing box culvert.

Table 17 – Arch Culvert Tolerances (Hydro-Future, 2015)

Sandstone Arch Tolerances	
Aspect	**Tolerances**
Bottom Width	4.0 – 4.2 m
Middle Height	1.8 m
Side Step Height	(N.A. , assume 0.6m)
Radius of Arch	N.A. , assume 4-5 meters
Wall Thickness	Assume 230 mm

Table 18 - Box Culvert Tolerances (Hydro-Future, 2015)

Concrete Box Tolerances	
Aspect	**Tolerances**
Bottom Width	3.67 – 3.6 m
Middle Height	1.8 – 1.9 m
Wall Thickness	Assume 230 mm

3. Water Engineering Design Options

The team at Hydro-Future Consulting have thoroughly investigated the area of concern along North Terrace, this investigation has assisted in the determination of a number of drainage solution options. The feasibility tender document outlined a number of potential options in which will be further explained and expressed within this stage of the Feasibility Study.

All potential drainage solution options within this feasibility study were required to undergo analysis to select the best option(s), based upon an number of factors selected by Hydro-Future, including:

- Environmental factors
- Relative Cost of Materials
- Resources
- Quality
- Effect to stakeholders
- Effect to existing infrastructure

The following components hydrological analysis of the feasibility study explore the potential drainage solution options and their respective preliminary designs.

Option areas include:

- Conventional Stormwater
- Swale Design
- Water Sensitive Urban Design
- Water Harvesting
- Combined Water Sensitive Urban Design Options

3.1. Decision Making Matrix

Each option considered within this feasibility study is required to be assessed through our decision making matrix. The four options listed in Table 19 were at the request of the client. This matrix is created to compare and rank the options based on score and weighting.

Table 19 - Decision Making Factors

Number	Factor	Weighting (%)
1	Cost	35
2	Flood Mitigation	35
3	Quality	20
4	Amenity	10

3.1.1. Cost
Cost is one of the two most important factors as can be seen in *Table 19*, which requires consideration when deciding upon the most feasible option. This is because the project needs to be completed to the specified budget outlined by the client

3.1.2. Flood Mitigation
Flood mitigation is as equally important as cost (*Table 19*) due to the project scope. The goal of this project is to increase drainage of surface water along North Terrace, Kent Town and hence if a solution does not provide substantial flood mitigation, then it in turn does not meet the project scope.

3.1.3. Quality
Quality is paramount to us at Hydro-Future Consulting as it represents our outstanding ability to deliver projects on time and to a high standard. In this project it carries a weighting of 20 percent (*Table 19*), so it is important that we, as a company maintain our focus to deliver quality drainage solutions. This also relates to the quality management of stormwater before, during and after project construction and consists of minimising the risk of pollutants and contaminants entering the surface water through sources such as urban run-off.

3.1.4. Amenity
The final factor included in our decision making matrix is amenity (*Table 19*). At Hydro-Future Consulting we believe that any project that is designed improve and/or eradicate issues, to make the lives easier of respective stakeholders deserves a large percentage of attention in regards to Amenity. Because North Terrace is such a busy, the aesthetic appeal is to be of a high quality. Secondary to this, the solution option needs to not only facilitate the drainage along North Terrace but also avoid negatively impacting the traffic (be it vehicles or pedestrians). The solution should preserve and promote the convenience along North Terrace.

Based on the inadequacies of the current stormwater system, Hydro-Future Consulting have considered a number of potential options and after consultation with the client, five final design options have been decided upon to be further investigated within this feasibility study. A final decision will be made on one of the five design options alongside criteria previously mentioned in Section ?. A preliminary design for each is included to demonstrate their capacity to function.

The following solution options are investigated:

1. **Upgrade Existing Infrastructure** – Involves analysing the existing stormwater infrastructure along North terrace to assess the existing capacity and in-turn determine a number of solutions to upgrade the system.
2. **Swale Design** – Involves designing a swale to be situated in a position on North Terrace, adjacent to the road.
3. **Water Sensitive Urban Design and Infiltration** – Involves assessing the feasibility of a number of WSUD options to be implemented along North Terrace and in surrounding areas such as car parks.
4. **Water Harvesting** – Involves analysing the possibilities to implement one or a number of water harvesting measures to collect and store water for re-use at a later date.
5. **Combined Drainage Option** – This option incorporates the use of multiple drainage options including water sensitive urban design/infiltration, water harvesting and conventional stormwater methods to create an environmentally friendly, efficient and water quality oriented drainage system solution.

3.2. Upgrade Existing Infrastructure

As discussed in Section Water2.1, the existing infrastructure contributing to North Terrace is not functional in the event of major storm occurrences and as a result, North Terrace is receiving a backlog of water. As a result, a number of alterations have been analysed to improve the overall drainage capabilities.

3.2.1. Increase number of pipes and inlets

Increasing the size of the piping network structure in addition to increasing the number and frequency of entry points into the pipe network, will allow the control of flow on the road and the surrounding area. A number of additional entry points have been proposed and a diagram of the additions has been included in Figure 48 (below).

This image displays the existing pipe network with the addition of an extension to existing pipe line on the Northern side of North Terrace as well as an increase in the number of side entry pits and grated pits.

This solution is a simple, cost effective and feasible method to capture the majority of surface water collecting in the low point of the section of North Road closest to Hackney Road and in-turn reduce the backlog of surface water. The absolute minimum slope for all stormwater pipes in this system is to be 1% or 1 in 100 to ensure adequate velocity.

The specifications of both side entry pits and grated pits will be defined in the detailed design stage and include considerations such as the dimensions and spacing of the side entry pits and the type of grated pit (sag pit or an on-grade pit). These will conform to the City of Norwood, Payneham and St Peters standards and the guidelines outlined by SA Water and the EPA's Guideline on Stormwater Pollution Prevention.

Figure 48 - Proposed addition of entry points to existing pipe network

These proposed stormwater infrastructure additions can be seen in Table 20, below.

Table 20 - Proposed stormwater infrastructure on North Terrace

Proposed Stormwater Infrastructure on North Terrace		
	Northern Side	**Southern Side**
Side Entry Pit	6	4
Grated Pit	2	4

3.2.2. Analysis of proposed Stormwater system using drains.

A DRAINS model was created to check the possible solutions in enhancing the conventional stormwater system to prevent the flood condition. For the DRAINS model, the parameters were identified through investigation. The calculated values of sub catchment areas and the IFD data were used. A rational DRAINS model was set up to compare the results with the hand calculated results. As this is the feasibility stage only a conceptual design representing the proposed drainage system was modelled in DRAINS to find out the approximate pipe sizes for the design.

In catchment properties, the time of concentration for the paved areas were assumed to be 15 minutes as the standards roof to gutter flow is 10 minutes (Argue,1986) and another 5 minutes were added considering the gutter to SEP travel time. The gutter to SEP travel time was assumed to be 5 minutes and the minimum travel time for the paved areas considered approximately as 5 minutes (Argue, 1986). The time of concentration for the impervious areas assumed to be 20 minutes. In the pit and pipe properties, Adelaide City Council's 3% cross –fall and 1% grade pit type and reinforced concrete pipes with 1% slope were selected. According to the WSA standards, for the pipes, a cover of 600mm was considered in calculating the invert levels for the DRAINS model.

Using the aforementioned information, a conceptual DRAINS model was designed. The designed model represents a conceptual design of the proposed stormwater system with 6 pits which span along the North Terrace for 700m in length. The existing stormwater system was extended up to the Fullarton Road because of the significant catchment area of 8.9ha. ARI of 20 years was assumed as the major storm as the required standard design for trapped low points within the project area is 1 in 20 year storms. Therefore the stormwater system was checked for 1 in 20 year storms as all the runoff eventually drains to the sag area along North Terrace near the junction with Hackney Road. The obtained results through the model are displayed in Figure 49 as follows:

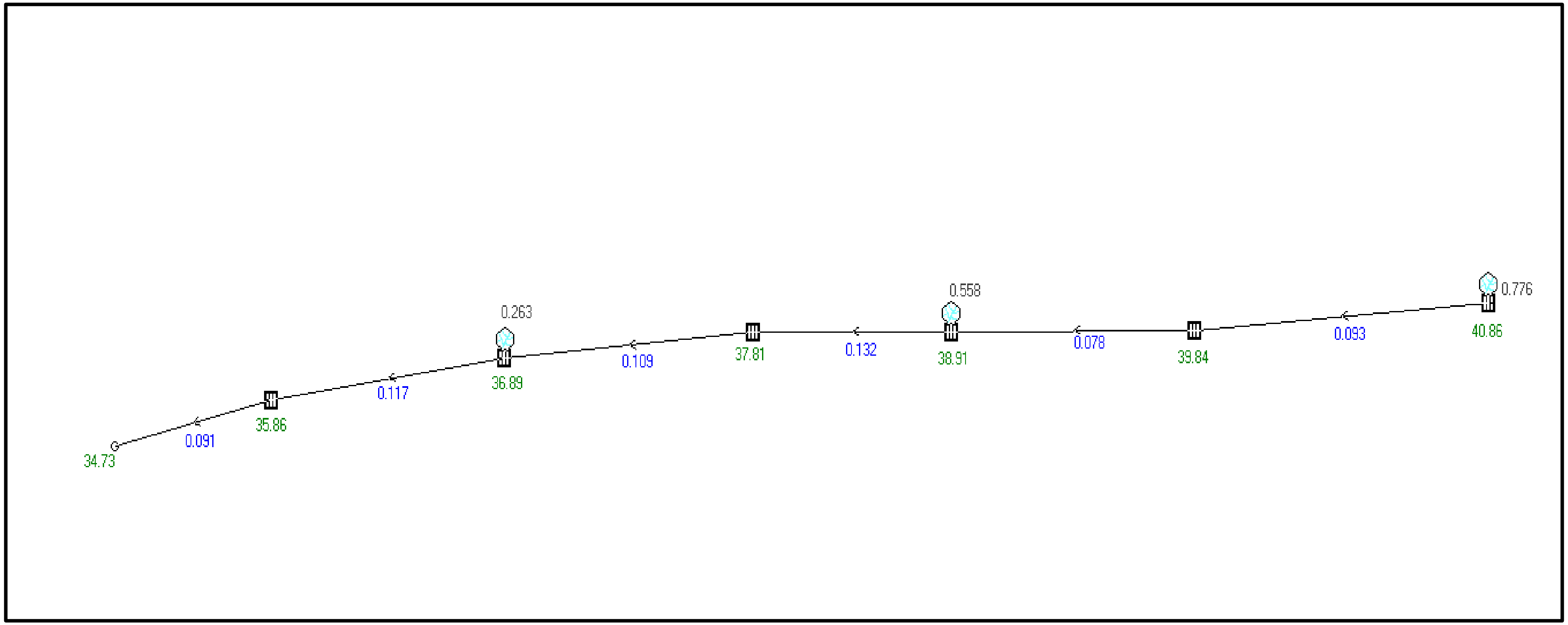


Figure 49 - DRAINS model for conceptual new stormwater infrastructure design (DRAINS, 2015)

Figure 49 represents the results from the DRAINS. The analysis of the results conclude that this particular approach of DRAINS is not suitable to estimate the pipe diameter as the DRAINS gave an error message (see Figure 50) (Ahammed, 2015). The DRAINS results indicate that the designed system is losing water at side entry pits which caused lower flow rates within the system than the total runoff flow. The detailed results of the model is included in the Appendix C3. DRAINS Output for Existing Infrastructure.

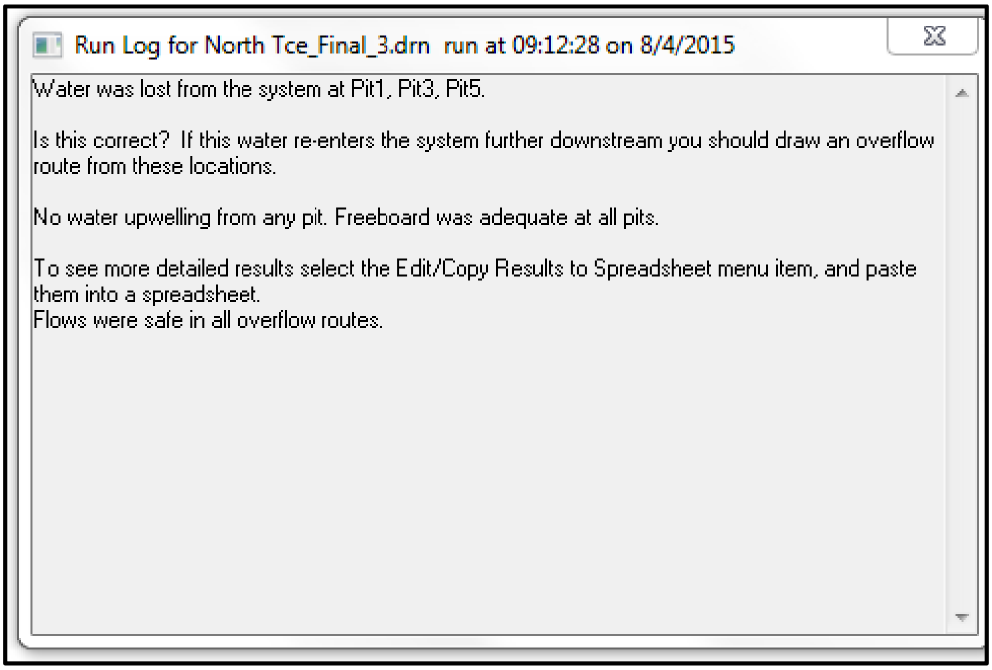


Figure 50 - DRAINS model for conceptual new stormwater infrastructure design – results (DRAINS, 2015)

The DRAINS model results in Figure 50 indicate that the calculated runoff, fail to enter the stormwater infrastructure. Therefore this results conclude that the catchment area has to be divided in to smaller sub - catchments to obtain more accurate model which will be modelled in detail design stage. The obtained long section from the DRAINS is shown in Figure 51.

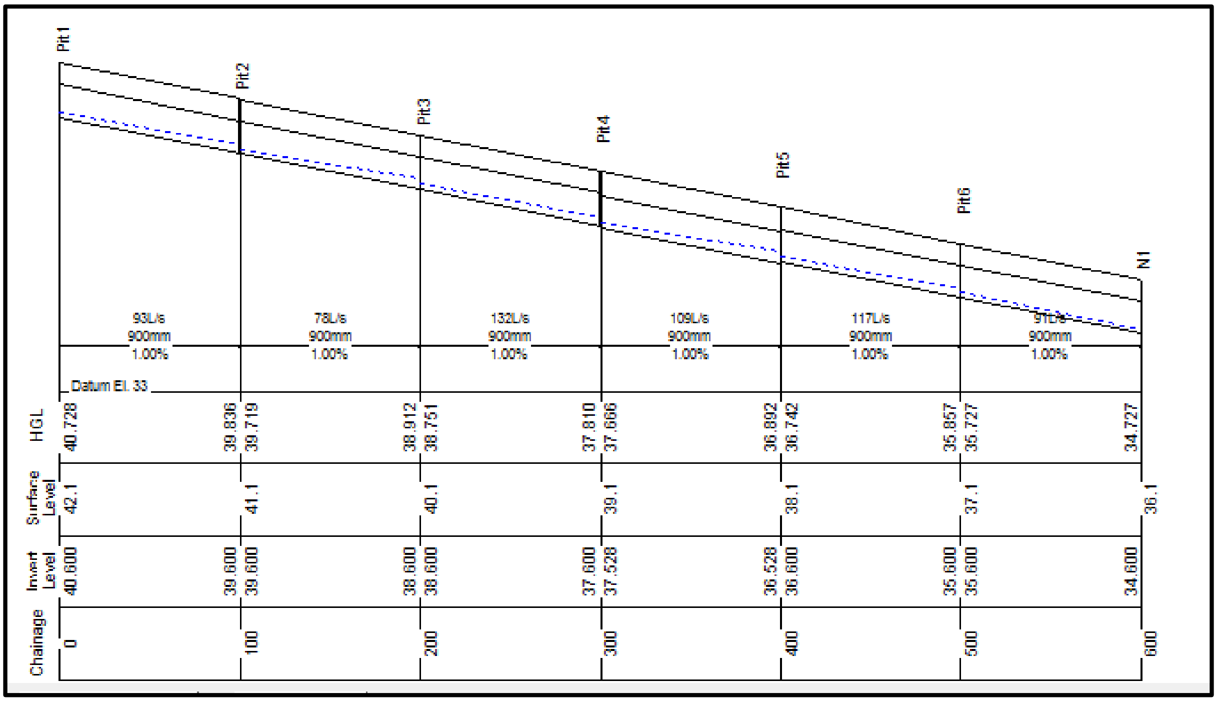


Figure 51 - DRAINS model for conceptual new stormwater infrastructure design – Long Section (DRAINS, 2015)

As the DRAINS model was not the best approach in this conceptual stage a simple hand calculation was done to find the approximate pipe diameter for the total runoff as follows;

$$D = \sqrt[\frac{3}{8}]{\frac{4^{5/3}\, nQ}{\pi S^{1/2}}}$$

$$D = \sqrt[\frac{3}{8}]{\frac{4^{5/3}\,(0.013)(1.36)}{\pi (0.01)^{1/2}}} = 0.808m$$

As the obtained pipe diameter through the hand calculations is 800mm it was assumed that using a 900mm pipe diameter in DRAINS model will be sufficient to manage the flooding in this conceptual design stage. But more investigations will be held in the detailed design considering more pits, more numbers of smaller sub-catchments and accurate calculated travel time to obtain exact pipe diameter and lengths. Based on this conceptual design, the costing will be calculated.

3.2.3. Implementation of a Sedimentation Basin

The implementation of a sedimentation basin within the botanical gardens utilising flows from First Creek is another option to be considered to alleviate flooding along North Terrace.

Positives

This solution is highly desirable as it not only acts as a natural solids filter for the Creek but is an effective way to hold and maintain stormwater discharged into First Creek. Sedimentation Basins improve the quality of the Stormwater but they cannot handle high flows. Sedimentation basins will improve the quality of the water, however, only up to a certain flow capacity.

A typical sedimentation basin is shown in Figure 52 below. This diagram shows the basic layout of a sedimentation basin and outlines its purpose. As the inflow comes in (right) at a high flow rate, it quickly settles inside of the pond and the suspended solids start to separate from the water and settle down to the bottom of the basin. It makes use of native vegetation to reduce microbiological and other organic materials to ensure the water that is discharged through the outflow pipe, which is shown on the left hand side of the figure, is of a higher quality than it was upon entering the basin. The outflow pipe is deliberately limited to ensure the settlement of the solids occurs inside of the basin and are not going back into the creek. There is an overflow weir included to prevent flooding.

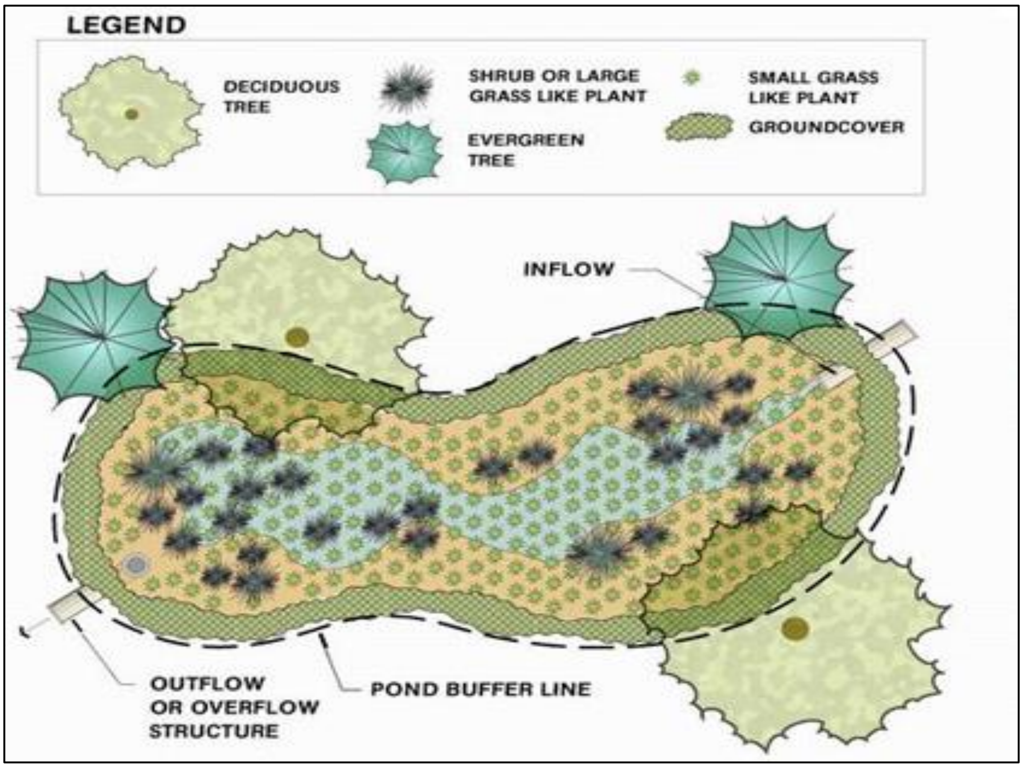


Figure 52 - Example of a Sedimentation Basin (USAF Sustainable Sites Toolkit)

Location of Basin

Due to the large number of residential properties around the First Creek area; an area of vacant land needs to be found. It should be placed with concern to visual amenity, while reducing the risk of people falling in the basin. Figure 53 below shows the proposed location of a sedimentation basin, which could potentially be located behind the Botanic Gardens and adjacent to First Creek.

Figure 53 - Image displaying possible location for a sedimentation basin

Limitations

Some downfalls of using a Sedimentation Basin are that it requires a significant amount of land to be effective, which may cause problems in a suburban area. Secondly, if large flows were to enter the sedimentation basin (1:50 year event) there is no guarantee that the sedimentation basin will remove the solids and other particles, due to the majority of stormwater overflowing.

The small amount of available land area within the Botanic Gardens, reduces the overall effectiveness of the sedimentation basin in absorbing the energy from the water in a major storm event, as well as improving the water quality, but is of still great potential.

Cost Estimate

The cost estimate in Table 21 is made up of the materials required for this option to upgrade the existing stormwater infrastructure.

Table 21 - Conventional Stormwater Material Expenses (Rawlinson's 2014)

Item	Description	Unit	Size	Qty	Rate ea	Cost ($)
1.0	**Stormwater**					
1.1	Side Entry Pits	mm	600^3	6	$930	5,580
1.2	Grate (Lid of Concrete Box)	mm	600^2	4	$231	924
1.3	Precast Concrete Box	mm	600^2	4	$338	1,352
1.4	Reinforced Concrete Pipe	m	0.90	310	$325	136,500
1.5	Reinforced Concrete Pipe	m	0.375	80	$138	11,040
					Total	**155,396**

3.3. Swale Design

The implementation of a swale along North Terrace (parallel to the road) is another potential option to promote drainage whilst also reducing the rate of surface water flow, improving water quality and increasing the aesthetic qualities of North Terrace. A typical swale is displayed in Figure 54 below, showing the shape and general layout.

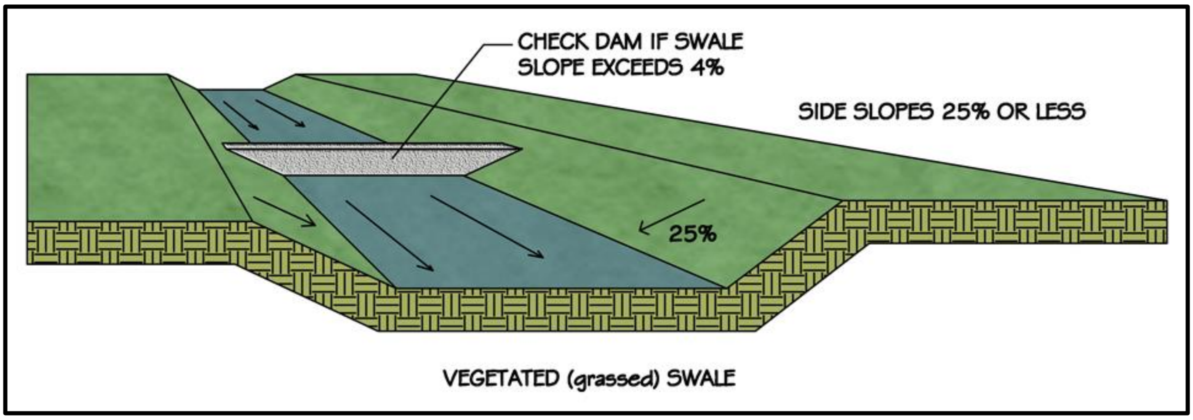


Figure 54 - Diagram of a typical swale (Abbey-Associates 2015)

3.3.1. Swale Types

There are two main types of swales used for stormwater management, a dry swale and a wet swale. Within these two swale types they can be vegetated to further reduce the rate of runoff.

3.3.1.1. Dry Swale

Dry swales are typically trapezoidal shaped open channels created to manage stormwater runoff from a number of sources including residential areas, along the shoulder of paved roads. Implementing a dry swale in a location on North Terrace will be beneficial in major storm events to carry surface water downstream to a discharge point.

Implementing a dry swale along North Terrace would ultimately reduce the rate of surface water runoff, filter the water to reduce contaminants and remove suspended solids. There are a number of potential positive uses of the captured and partially filtered water such as utilization for irrigation and the replenishment of groundwater.

How do dry swales work?

Swales are designed to collect surface water through gravitational flow. Each side of a swale has a sloped surface (as seen in Figure 55), typically grassed, which meets the paved surface at its top. With a dry swale, as the water passes over the sloped surface, it collects at the bottom of the swale, where it begins to infiltrate the surface soil. As the water passes through the soil it is filtered by the soil particles, in-turn removing a number of its contaminants. In major storm events, the water is detained and typically passes through one or a number of perforated or slotted pipes. The advantage of these pipes is it allows excess water (particularly in major storm events) to pass through into the established stormwater system. In a detention situation the quality of the collected water will continue to improve as it works its way through the soil layers. The use of drainage measures in swale channels such as perforated pipes are generally only beneficial when permeable soils are evident in the area of construction as this consistency of soil will allow water to infiltrate.

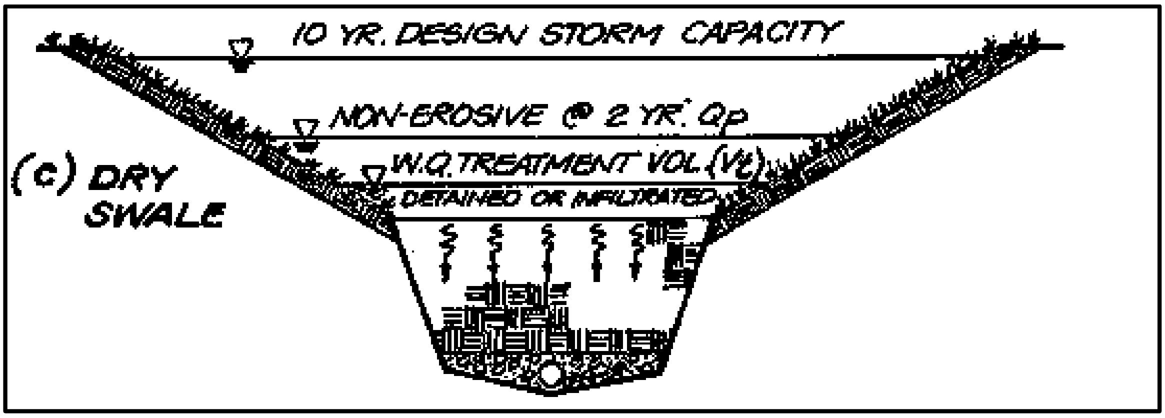


Figure 55 - Diagram of a typical dry swale (Claytor & Schueler, 1996)

Limitations of dry swales:

The limitations in implementing a dry swale include:

- Dry swales can be damaged as a result of off road parking.
- The drainage area leading to a dry swale is smaller than 20,000m^2 to keep flows lower and prevent erosion.
- Using dry swales can result in contamination to the groundwater, especially in industrial and commercial areas.

3.3.1.2. Wet Swale

Wet swales are the same as dry swales in terms of their shape and purpose. The major difference between the two is that the bed of a wet swale does not consist of any underlying material for filtration. Check dams are used to sub-divide the swale and create a number of small wetland areas or marshes inside the swale.

Wet swales have a number of environmental benefits in relation to water quality. Once the water collected in wet swales settles, its heavier nutrients and suspended solids settle to the soil. The roots of plants commonly found in wet swales absorb a margin of the water's nutrients and pollutants, in-turn further improving the quality of the water.

How do wet swales work?

Wet swales work very similar to dry swales in the way that they collect water, except they do not have infiltration layers or perforated pipes to collect and transfer surface water (as seen in Figure 56). The key component of how a swale works is in regards to natural growth and its benefits from biological processes, which ultimately improves water quality before the water is directed downstream to secondary stormwater infrastructure. In the case of a major storm event, swales provide a degree of flood protection through the implementation of flow diversion measures or in extreme cases, allowing the water to pass over banks in a safe manner. These measures of course need to be considered in the design stage and for extreme cases will implement structural measures of control.

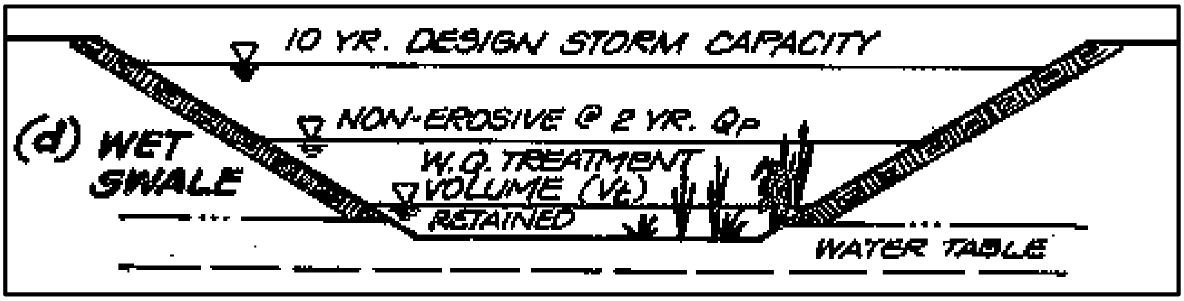


Figure 56 - Diagram of a typical wet swale (Claytor & Schueler, 1996)

Limitations of wet swales

The limitations in implementing a wet swale include:

- Not suitable for very flat locations, as the lack of slope may result in moderate levels of ponding, therefore limiting drainage.
- A wet swale can experience erosion in the event of major stormwater occurrences, carrying large volumes of water at high velocities.
- Wet swales cannot be driven on and therefore cannot be placed adjacent to a road, but rather need to be placed behind the kerb and gutter and associated footpath.

3.3.1.3. Advantages of Swales:

A number of advantages exist in relation to wet and dry swales and these include:

- Controls and captures runoff and in-turn improves drainage
- Reduces stormwater runoff flow rate
- Improves water quality by trapping and removing contaminants and pollutants
- The structure's shape is linear so can run parallel to North Terrace
- Aesthetic qualities
- Ease of construction
- Relatively low cost
- Relatively easy to maintain

3.3.1.4. Disadvantages of Swales

A small number of disadvantages exist in relation to dry swales and these include:

- Requires more maintenance over a traditional curb and gutter setup
- Requires dense vegetation to prevent high flow rates

3.3.1.5. Maintenance Routine

A routine inspection for maintenance is conducted annually and consists of the following:

- Inspect the swale and replace the vegetation if needed.
- Inspect the vegetation and trim vegetation periodically.
- Inspect the swale's functionality and make sure it is clear of any pollutants and repair if required.
- Remove the trash and accumulated pollutants.
- Non routine maintenance may be required at times.

3.3.1.6. Environmental Aspects

Swales improve the quality of stormwater flows. Wet swales work well to remove pollutants and contaminants from surface water. A large percentage of suspended solids found in surface water from urban runoff (typically post-development) is captured and filtered by swales, particularly the soil and roots of plants. Generally speaking, the more vegetated a swale is, the greater its capacity is to remove suspended solids is. Dry swales can also work well to remove contaminants and suspended solids, however they typically require the implementation of an engineered filtration item to enhance the effects.

The following table (Table 22) displays the estimate percentages of total suspended solids (TSS), pollutants and contaminants removed from surface waters as a result of the implementation of swales.

Table 22 -Percentage of Removed Pollutants and Contaminants

Pollutant/Contaminant	Percentage Removed	
	Dry Swale	Wet Swale
Total Suspended Solids (TSS)	80%	80%
Phosphorus	50%	25%
Nitrogen	50%	40%
Heavy Metals	40%	20%

3.3.1.7. Potential locations to implement a swale

Fitting a swale along North Terrace presents a challenge and will require a lot investigating to ensure it is feasible when compared to the other options presented in this Feasibility Study. The swale will need to be constructed along the shoulder of North Terrace and adjacent to the road. The consideration of amenity is high in a project like this as North Terrace is such an important arterial road with thousands of pedestrians using it every day, so keeping the footpath would be beneficial, but due to size, part of the swale may need to sit in-place of the existing footpath along with the kerb and gutter.

3.3.1.8. Design Option

A dry swale would be best suited to this particular application as the water detained in the swale can be transferred through the perforated pipes to the existing stormwater system. Due to the slow rate of infiltration, the perforated pipe would only receive small volumes of water at any given time and thus, a further overflow system would be the best precautionary measure to implement in conjunction with the perforated pipe. This would divert the water directly into the stormwater system via a secondary collection pipe. The dry swale will remain dry in months where no heavy rainfall has occurred, which will reduce potential environmental impacts due to stagnant water.

3.3.1.8.1. Soil Type Considerations

The soil type and consistency along North Terrace ranges between red brown earth (RB5 type) (in the North-Eastern end of North Terrace and alluvial (AL type) in the South-Western end of North Terrace). The alluvial soil makes up around 65% of the project area’s soil and is relative to the location of where a swale would be implemented. The borehole data shows that the top 1250mm of alluvial soil along North Terrace consists of sandy, gravely clay with low plasticity. In reference to Section 3.3.1.9 the depth of the pipe can be seen to be at 1400mm below the existing ground level. This places the pipe in the silty, gravely sand layer with medium plasticity. As previously noted, the top 1250mm of soil is of sandy, gravely clay and will therefore be impermeable to a degree and as a result will limit the amount of water that can pass through into the perforated pipe.

3.3.1.9. Preliminary Design

The implementation of a swale is primarily for the purpose of carry stormwater away from trapped low points along North Terrace as per Section 1 and as a result design flows in these areas will need to be able to overflow into the stormwater system at these particular points of concern.

Swale Requirements:

- 1:5 Year Average Recurrence Interval (ARI)
- $Q_5 = 0.332\ m^3/s$

3.3.1.10. Final Design Dimensions

The preliminary calculations for the required swale dimensions are included in Appendix A.

As the required flowrate can be contained by the sizes assumed in Section 2.1.1, a trapezoidal swale of the following dimensions will be required for the calculated flow rate:

Height (H) = 0.35m

Base Width (B) = 2m

Bed Slope (s) = 2%

Side Width (C) = 1.5m

Overall Width (A) = 5m

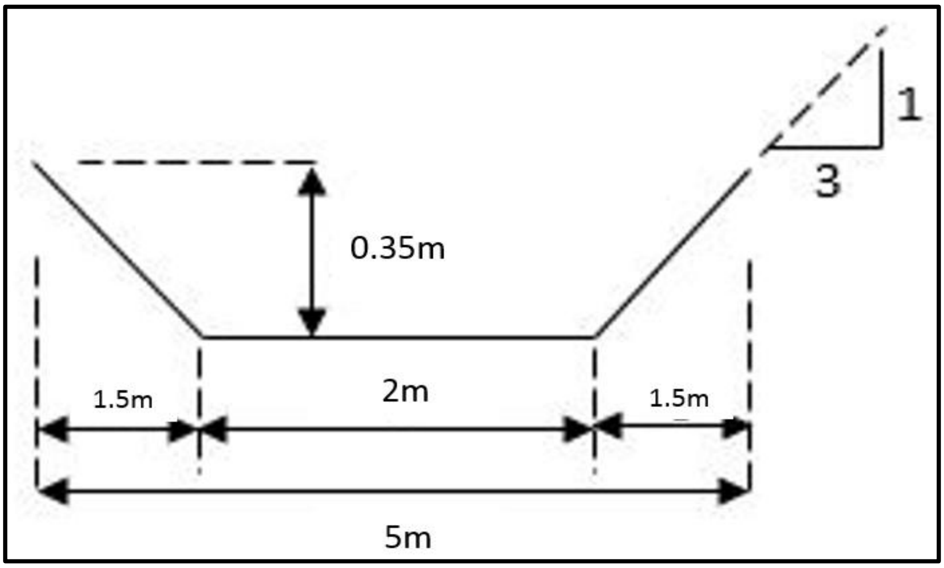


Figure 57 - Diagram showing dimensions of designed swale (Hydro-Future Consulting, 2015)

3.3.1.10.1. Pipe Depth

Based on a trapezoidal dry swale with a 2 metre base width, the layer of permeable soil is to be between 0.6m and 1.8m (Storm Water Manager's Resource Center, 2012) and will sit on a minimum of 150mm of gravel (Storm Water Manager's Resource Center, 2012). This is better illustrated in Figure 58 below.

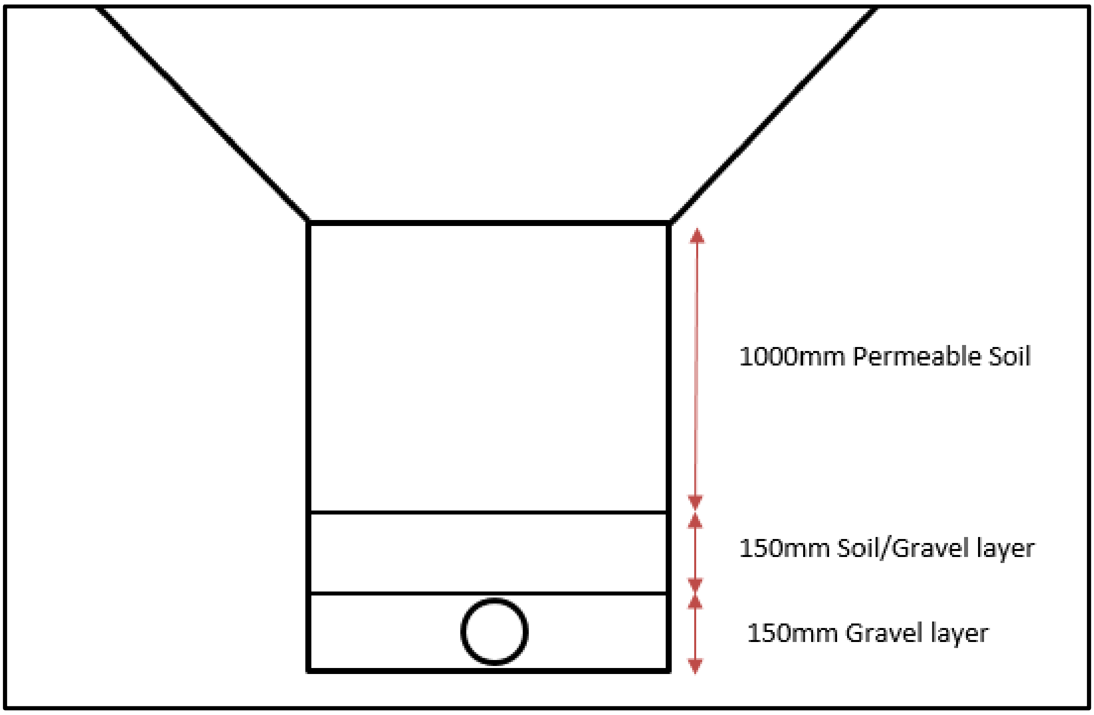


Figure 58 - Diagram showing soil layer thicknesses and pipe depth

Selecting a permeable soil depth layer of 1000mm will result in a pipe depth of 1.15 metres or 1150mm from the channel bed.

3.3.1.11. Placement of Swale

As previously stated in Section 3.3.1.7 locating an area along North Terrace to fit a swale is a challenging task. In a lot of cases where not much shoulder room is present along a road, the footpath and verge, sometimes along with the kerb and gutter is removed to make room for a swale to sit in its place, adjacent to the road, as seen in Figure 59 below.

Figure 59 - Roadside Swale in the place of a footpath and verge (Benvironment, 2014)

The problem with fitting a swale in an area on North Terrace is that not much room is available. The road sits on the outskirts of the CBD and therefore removing the footpath and verge is out of the question as this will impact the large volume of pedestrians using the footpath on a daily basis.

The standard footpath width along North Terrace is around 3.0-3.2 metres, as depicted in Figure 60. As the swale is five metres wide, a minimum of five metres is required behind the footpath and as a result this limits the chances along North Terrace to place the swale.

Unfortunately due to the number of closely spaced buildings, the number of crossovers and general lack of space, between Hackney road and College road, a swale will not fit. Referring to Figure 60, the amount of space required for the swale can be seen clearly. The length here should be neglected as this would be decided in the design stage of this project.

Figure 60 - Arial shot of North Terrace, displaying the room required for a swale (Google Maps, 2015)

3.3.1.12. Cost Estimate

The costs associated with constructing and implementing a swale along North Terrace will not be considered. This predominantly due to the aforementioned lack of space to fit a swale along North Terrace as well as the undesired soil profiles existent with limited permeability. Therefore, the implementation of a swale is deemed impractical and ultimately unfeasible for our design.